COURSE OUTLINE

- Wind Loads
  - MWFRS vs C&C
  - Definitions
  - Design Provisions
  - Example
- Tornado Loads
  - Construction Documents
  - Load Combinations
  - Definitions and Symbols
  - Design Provisions
  - Example
- Resources

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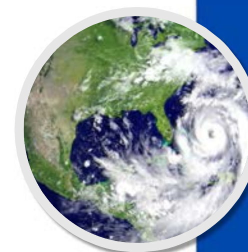
# Wind Loads

Updates, deletions and revisions to various provisions



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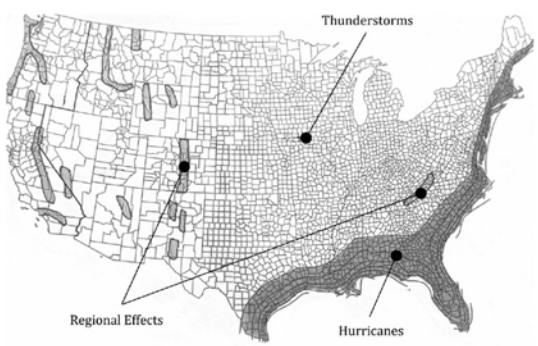
## Extreme Wind Speeds




*Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

15

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## Wind Pressures on a Building

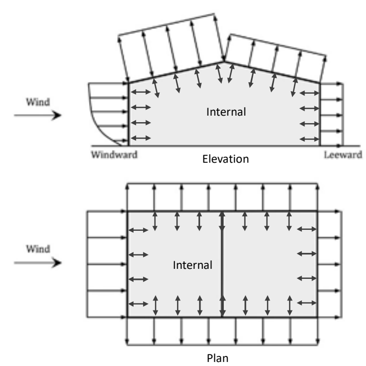


Figure 5.6 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill


youtube.com/shorts/zg7q-lzGBG4?feature=shared

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## MWFRS vs C&C

- MWFRS – main wind-force resisting systems
- C&C – components and cladding

| MWFRS | C&C (not part of MWFRS) |
| --- | --- |
| Diaphragms | Lateral framing (studs and connections) |
| Shear walls | Suction on wall/roof sheathing & cladding |
| Moment frames | Rafters and purlins |
| Braced frames | Curtain walls |
| Roof framing uplift (≥3:12 slope) | Roof framing uplift (<3:12 slope) |

Some elements are both MWFRS and C&C and must be designed for governing conditions

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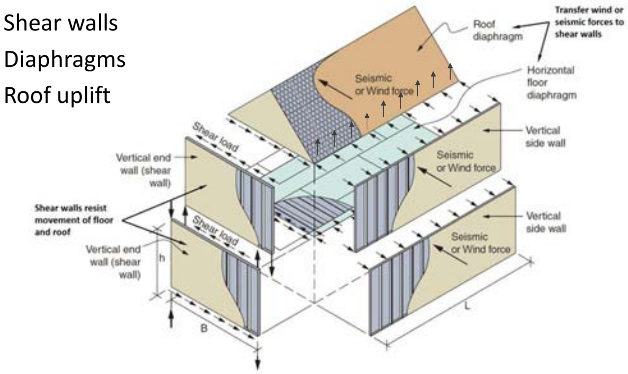
## MWFRS Loads

EXAMPLE

- Shear walls
- Diaphragms
- Roof uplift




18

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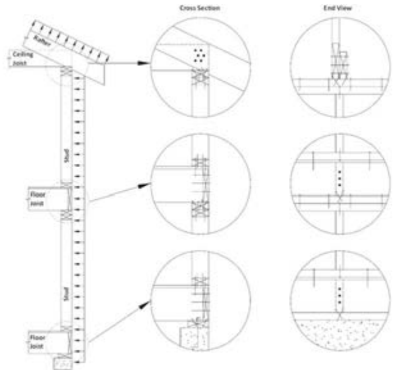
## C&C Loads

EXAMPLE

- Lateral loads on studs
- Fasteners of studs to framing
- Sheathing and its fasteners
- Suction on rafters and purlins
  - Not all roof uplift connectors




Courtesy of American Wood Council

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## Definitions

CODE CHANGE

**<u>BASIC</u> WIND SPEED, V.** ~~Basic design wind speeds.~~ <u>The wind speed used for design, as determined in Chapter 16.</u>

**WINDBORNE DEBRIS REGION.** Areas within hurricane-prone regions located:

1. Within 1 mile of the mean high-water line where an Exposure D condition exists upwind at the waterline and the basic ~~design~~ wind speed, *V*, is 130 mph or greater; or
2. In areas where the basic ~~design~~ wind speed, <u>*V*,</u> is 140 mph or greater.

For Risk Category II buildings and structures and Risk Category III buildings and structures, except health care facilities, the windborne debris region shall be based on Figure ~~1609.3.(1)~~ <u>1609.3.(2)</u>. For <u>Risk Category III health care facilities, and</u> Risk Category IV buildings and structures ~~and Risk Category III health care facilities,~~ the windborne debris region shall be based on Figure ~~1609.3(2)~~ <u>1609.3(3) and Figure 1609.3(4), respectively</u>.

<u>**WIND DESIGN GEODATABASE.** The ASCE database (version 2022-1.0) of geocoded wind speed design data. The ASCE Wind Design Geodatabase of geocoded wind speed design data is available at https://asce7hazardtool.online/.</u>

- "basic ~~design~~ wind speed" changed throughout IBC
- Windborne debris region is not just for coastal areas
- ASCE Wind Design Geodatabase provides values (asce7hazardtool.online)
  - Based on risk category
  - Free resource

§ 202


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## Determination of Wind Loads

CODE CHANGE

§ 1609.1.1

**1609.1.1 Determination of wind loads.** Wind loads on every building or structure shall be determined in accordance with Chapters 26 to 30 of ASCE 7. The type of opening protection required, the basic ~~design~~ wind speed, V, and the exposure category for a site is permitted to be determined in accordance with Section 1609 or ASCE 7. Wind shall be assumed to come from any horizontal direction and wind pressures shall be assumed to act normal to the surface considered.

**Exceptions:**

No changes to exceptions 1-6

7. Temporary structures complying with Section 3103.6.1.2.

- New exception for temporary structures

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## Basic Wind Speed

CODE CHANGE

§ 1609.3

The basic ~~design~~ wind speed, *V*, in mph, for the determination of the wind loads shall be determined by Figures 1609.3(1) through ~~1609.3(12)~~ 1609.3(4). The basic ~~design~~ wind speed, *V*, for use in the design of Risk Category I ~~II~~ buildings and structures shall be obtained from Figure~~s~~ 1609.3(1)~~, 1609.3(5) and 1609.3(6)~~. The basic ~~design~~ wind speed, *V*, for use in the design of Risk Category II ~~III~~ buildings and structures shall be obtained from Figure~~s~~ 1609.3(2)~~, 1609.3(7) and 1609.3(8)~~. The basic ~~design~~ wind speed, *V*, for use in the design of Risk Category III ~~IV~~ buildings and structures shall be obtained from Figure~~s~~ 1609.3(3)~~, 1609.3(9) and 1609.3(10)~~. The basic ~~design~~ wind speed, *V*, for use in the design of Risk Category IV ~~I~~ buildings and structures shall be obtained from Figure~~s~~ 1609.3(4)~~, 1609.3(11) and 1609.3(12)~~. Basic wind speeds for Hawaii, US Virgin Islands, and Puerto Rico shall be determined by using the ASCE Wind Design Geodatabase. The ASCE Wind Design Geodatabase is available at https://asce7hazardtool.online, or an approved equivalent.

The basic ~~design~~ wind speed, *V*, for the special wind regions indicated near mountainous terrain and near gorges shall be in accordance with local jurisdiction requirements. The basic ~~design~~ wind speeds, *V*, determined by the local jurisdiction shall be in accordance with Chapter 26 of ASCE 7. In nonhurricane-prone regions, when the basic ~~design~~ wind speed, *V*, is estimated from regional climatic data, the basic design wind speed, *V*, shall be determined in accordance with Chapter 26 of ASCE 7.

- Related technical and editorial changes throughout IBC
- Consistent with ASCE 7-22
- Hawaii, US Virgin Islands, and Puerto Rico per ASCE Wind Design Geodatabase

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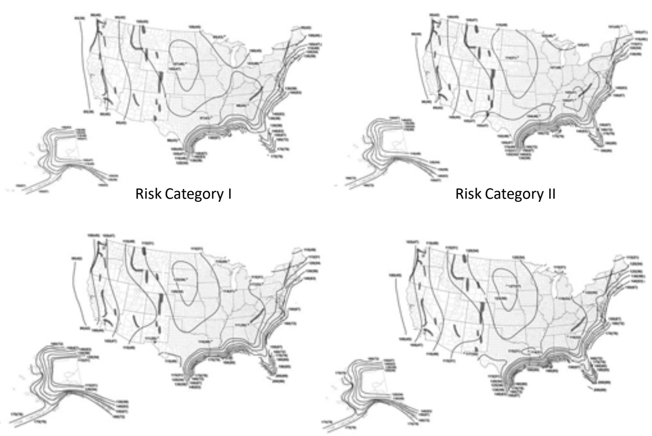
## Basic Wind Speed Maps

Figs 1609.3(1-4)



Risk Category I

Risk Category II

Risk Category III

Risk Category IV

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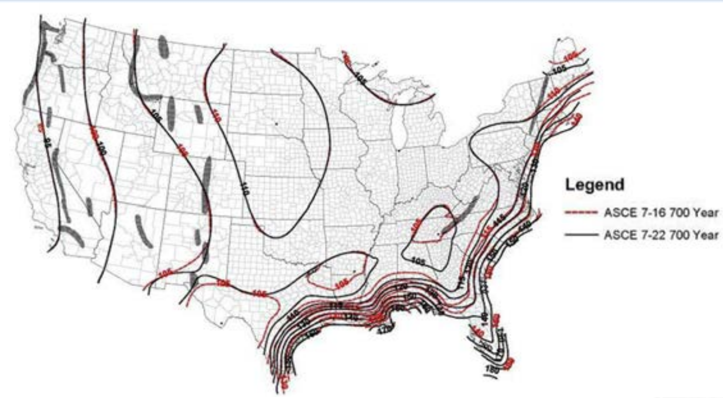
## Wind Speed Comparison – RC II

CODE CHANGE

§ 1609.3

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EXAMPLE

§ 1609.3

## Coastal Wind Speeds

| Excerpt of ASCE 7-22 Table C26.5-3 – Basic Wind Speeds (mph) | | | |
|---|---|---|---|
| Location | Risk Cat. II (700-year) | Risk Cat. III (1,700-year) | Risk Cat. IV (3,000-year) |
| Bar Harbor, Maine | 109 | 119 | 121 |
| Hampton Beach, New Hampshire | 113 | 124 | 125 |
| Boston, Massachusetts | 116 | 125 | 129 |
| Hyannis, Massachusetts | 123 | 139 | 141 |
| Newport, Rhode Island | 124 | 139 | 139 |
| New Haven, Connecticut | 120 | 129 | 133 |
| Southampton, New York | 129 | 138 | 140 |
| Manhattan, New York | 116 | 127 | 130 |
| Atlantic City, New Jersey | 126 | 135 | 138 |
| Rehoboth Beach, Delaware | 122 | 131 | 136 |
| Ocean City, Maryland | 128 | 136 | 139 |
| Virginia Beach, Virginia | 125 | 132 | 138 |
| Wrightsville Beach, North Carolina | 146 | 156 | 160 |
| Folly Beach, South Carolina | 149 | 158 | 165 |
| Sea Island, Georgia | 131 | 145 | 153 |
| Jacksonville Beach, Florida | 129 | 140 | 149 |
| Melbourne Beach, Florida | 152 | 162 | 172 |
| Miami Beach, Florida | 171 | 183 | 191 |
| Key West, Florida | 176 | 200 | 200 |
| Clearwater, Florida | 146 | 154 | 160 |
| Panama City Beach, Florida | 141 | 146 | 162 |
| Gulf Shores, Alabama | 159 | 172 | 181 |
| Biloxi, Mississippi | 157 | 176 | 177 |
| Slidell, Louisiana | 138 | 152 | 155 |
| Cameron, Louisiana | 141 | 154 | 157 |
| Galveston, Texas | 151 | 159 | 166 |
| Port Aransas, Texas | 159 | 157 | 174 |

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CODE CHANGE

§ 1609.7

## Elevators, Escalators, and Other Conveying Systems

Elevators, escalators, and other conveying systems and their components exposed to outdoor environments shall satisfy the wind design requirements of ASCE 7.

- Similar provisions for flood and seismic loads

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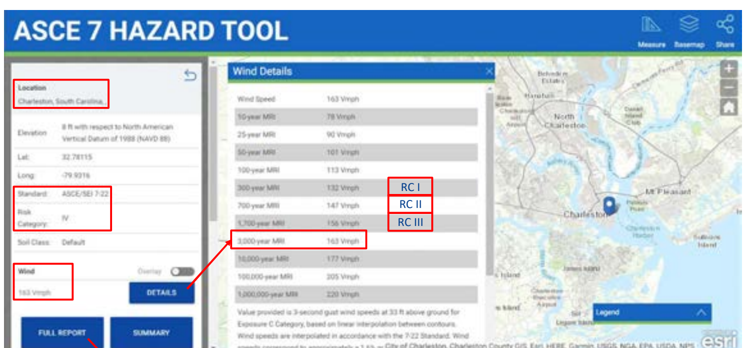
EXAMPLE

## asce7hazardtool.online



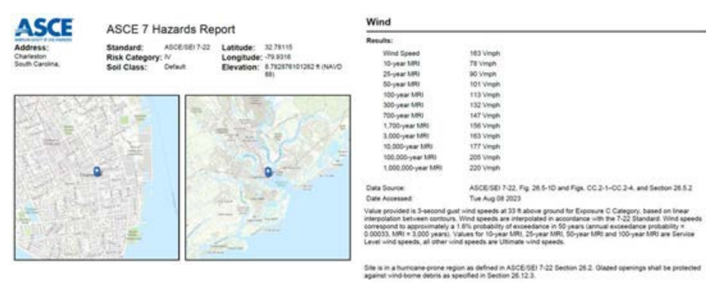

Generate PDF...

28

28

EXAMPLE

## asce7hazardtool.online




Full Report PDF

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## Summary of ASCE 7-22 Wind Procedures

| System | ASCE 7 Chapter | Description |
|---|---|---|
| MWFRS | 27 | Directional procedure for buildings of all heights |
| | 28 | Envelope procedure for low-rise buildings |
| | 29 | Directional procedure for building appurtenances (rooftop structures and rooftop equipment) and other structures |
| | 31 | Wind tunnel procedure for any building or other structure |
| C&C | 30 | • Envelope procedure in Part 1; or<br>• Directional procedure in Parts 2 and 3<br>• Building appurtenances (roof overhangs and parapets) in Part 4; and<br>• Nonbuilding structures in Part 5 |
| | 31 | Wind tunnel procedure for any building or other structure |

- Will not cover Chapter 31
- Will not cover nonbuilding structures
  - Except ground-mounted PV systems

Table 5.1 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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CODE CHANGE

## Summary of ASCE 7-22 Major Wind Changes

- Basic wind speeds , $V$, updated (already discussed)
- “Simple Diaphragm Building” deleted
  - 2 simplified methods also deleted
- Wind directionality factor, $K_d$, moved
- Topographic factor, $K_{zt}$, updated
- Velocity pressure exposure coefficient , $K_z$, revised
- New provisions for “Elevated Buildings” added
- New provisions for ground-mounted fixed-tilt solar panel systems added
- C&C wind load provisions revised

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CODE CHANGE

§ 26.2

### Definitions

~~**BUILDING, SIMPLE DIAPHRAGM:** A building in which both windward and leeward wind loads are transmitted by roof and vertically spanning wall assemblies, through continuous floor and roof diaphragms, to the MWFRS.~~

- All simple diaphragm building provisions deleted
  - ~~CHAPTER 28 PART 2: ENCLOSED SIMPLE DIAPHRAGM LOW-RISE BUILDINGS~~
  - ~~CHAPTER 30 PART 2: LOW-RISE BUILDINGS (SIMPLIFIED)~~
  - ~~CHAPTER 27 PART 2: ENCLOSED SIMPLE DIAPHRAGM BUILDINGS WITH h ≤ 160 ft~~
  - ~~CHAPTER 30 PART 4: BUILDINGS WITH 60 ft < h ≤ 160 ft (SIMPLIFIED)~~

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CODE CHANGE

§ 26.2

### Definitions

**BUILDING, ELEVATED:** A building supported on structural elements where wind can pass beneath the building.

- New provisions included in Chapters 27 (MWFRS) and 30 (C&C)

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CODE CHANGE

§ 26.2

## Definitions

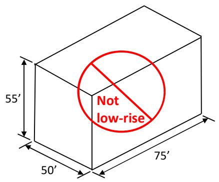
**BUILDING, LOW-RISE:** An enclosed, ~~or~~ partially enclosed, or partially open building that complies with the following conditions:

**1.** Mean roof height $h$ less than or equal to 60 ft, and

**2.** Mean roof height $h$ does not exceed the least horizontal dimension.

**OPENINGS:** ~~Apertures or~~ holes ~~in the building envelope~~ that allow air to flow through the building envelope ~~and that are designed as "open" during design winds as defined by these provisions~~ during a design wind event.

- Low-rise buildings
  - "Partially open" buildings included
- "Openings" clarified

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## Wind Loads – General Requirements

Ch 26

| References* | Wind Loads: General Requirements (for both MWFRS and C&C) [ASCE 7 Ch 26] |
|---|---|
| ASCE 7 F26.5-1A-D or asce7hazardtool.online | Basic wind speed, $V$ |
| ASCE 7 §26.6 | Directionality factor, $K_d$ |
| ASCE 7 §26.7 | Exposure category: B, C or D |
| ASCE 7 §26.8 | Topographic factor, $K_{zt}$ |
| ASCE 7 §26.9 | Ground elevation factor, $K_e$ |
| ASCE 7 §26.10.1 | Velocity pressure exposure coefficient, $K_z$ or $K_h$ |
| ASCE 7 §26.11 | Gust Effect Factor, $G$ and $G_f$ |
| ASCE 7 §26.12 | Enclosure classification |
| ASCE 7 §26.13 | Internal pressure coefficient, $(GC_{pi})$ |

*IBC 2024 and ASCE 7-22

pressure coefficients sometimes grouped with gust factor – shown inside parenthesis

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CODE CHANGE

§ 26.10

## Wind Directionality Factor, $K_d$

- Moved from velocity pressure calculations to wind pressure calculations
  - Chapters 27-30
- $K_d$ = 0.85 for buildings' MWFRS and C&C

$q_z = 0.00256\, K_z\, K_{zt}\, \cancel{K_d}\, K_e\, V^2$ (26.10-1)

$p = q\, K_d\, GC_p - q_i\, K_d\, (GC_{pi})$ (27.3-1)

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## Exposure Category

§ 32.5.2

- Surface roughness
- 80% all buildings Exposure B (NAHB)


Exposure B


Exposure C


Exposure D

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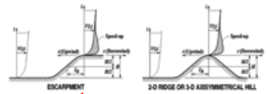
CODE CHANGE

§ 26.8

## Topographic Factor, $K_{zt}$

- Considered even in presence of upwind topographic features of similar height

**Diagrams**




**Topographic Multipliers**[a,b,c,d]

| $H/L_h$ | $K_1$ Multiplier: 2D Ridge | $K_1$ Multiplier: 2D Escarpment | $K_1$ Multiplier: 3D Axisym-metrical Hill | $x/L_h$ | $K_2$ Multiplier: 2D Escarpment | $K_2$ Multiplier: All Other Cases | $z/L_h$ | $K_3$ Multiplier: 2D Ridge | $K_3$ Multiplier: 2D Escarpment | $K_3$ Multiplier: 3D Axisym-metrical 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 |
| | | | | | | | 0.50 | 0.01 | 0.02 | 0.00 |
| | | | | | | | 2.00 | 0.00 | 0.00 | 0.00 |

[a]For values of $H/L_h$, $x/L_h$, and $z/L_h$ other than those shown, linear interpolation is permitted.

[b]For $H/L_h > 0.5$, assume that $H/L_h = 0.5$ for evaluating $K_1$ and substitute $2H$ for $L_h$ for evaluating $K_2$ and $K_3$.

[c]Multipliers are based on the assumption that wind approaches the hill or escarpment along th direction of max

[d]Multipliers shall be used for any exposure.

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CODE CHANGE

§ 26.10.1

## Velocity Pressure Exposure Coefficient, $K_z$

$$K_z = \begin{cases} 2.41\left(\dfrac{15}{z_g}\right)^{2/\alpha} & \text{for } z < 15 \text{ ft} \\ 2.41\left(\dfrac{z}{z_g}\right)^{2/\alpha} & \text{for } 15 \text{ ft} \le z \le z_g \\ 2.41 & \text{for } z_g < z \le 3{,}280 \text{ ft} \end{cases}$$

| Exposure | α | $z_g$ (ft) |
|---|---|---|
| B | ~~7.0~~ 7.5 | ~~1200~~ 3280 |
| C | ~~9.5~~ 9.8 | ~~900~~ 2460 |
| D | 11.5 | ~~700~~ 1935 |

| Ht above Ground z or h | Exposure Category | | |
|---|---|---|---|
| ft | B | C | D |
| 0–15 | 0.57 (0.70)* | 0.85 | 1.03 |
| 20 | 0.62 (0.70)* | 0.90 | 1.08 |
| 25 | 0.66 (0.70)* | 0.94 | 1.12 |
| 30 | 0.70 | 0.98 | 1.16 |
| 40 | ~~0.76~~0.74 | 1.04 | 1.22 |
| 50 | ~~0.81~~0.79 | 1.09 | 1.27 |
| 60 | ~~0.85~~0.83 | 1.13 | 1.31 |
| 70 | ~~0.89~~0.86 | 1.17 | 1.34 |
| 80 | ~~0.93~~0.90 | 1.21 | 1.38 |
| 90 | ~~0.96~~0.92 | 1.24 | 1.40 |
| 100 | ~~0.99~~0.95 | 1.26 | 1.43 |
| 120 | ~~1.04~~1.00 | 1.31 | 1.48 |
| 140 | ~~1.09~~1.04 | ~~1.36~~1.34 | 1.52 |
| 160 | ~~1.13~~1.08 | 1.39 | 1.55 |
| 180 | ~~1.17~~1.11 | ~~1.43~~1.41 | 1.58 |
| 200 | ~~1.20~~1.14 | ~~1.46~~1.44 | 1.61 |
| 250 | ~~1.28~~1.21 | ~~1.53~~1.51 | 1.68 |
| 300 | ~~1.35~~1.27 | ~~1.59~~1.57 | 1.73 |
| 350 | ~~1.41~~1.33 | ~~1.64~~1.62 | 1.78 |
| 400 | ~~1.47~~1.38 | ~~1.69~~1.66 | 1.82 |
| 450 | ~~1.52~~1.42 | ~~1.73~~1.70 | 1.86 |
| 500 | ~~1.56~~1.46 | ~~1.77~~1.74 | 1.89 |

* Use 0.70 in Chapter 28, Exposure B, when $z < 30$ ft

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CODE CHANGE

§ 26.12.1

**Enclosure Classification – General**

For the purpose of determining internal pressure coefficients, ~~all~~ buildings and other structures for which internal pressure coefficients ($GC_{pi}$) apply shall be classified as enclosed, partially enclosed, partially open, or open, as defined in Section 26.2. If a building or other structure satisfies both the "open" and "partially enclosed" enclosure classification definitions, it shall be classified as a "partially open" building or other structure.

**Table 26.13-1 Main Wind Force Resisting System and Components and Cladding (All Heights): Internal Pressure Coefficient, ($GC_{pi}$), for Enclosed, Partially Enclosed, Partially Open, and Open Buildings (Walls and Roof)**

| Enclosure Classification | Criteria for Enclosure Classification | Internal Pressure | Internal Pressure Coefficient, ($GC_{pi}$) |
|---|---|---|---|
| Enclosed buildings | $A_o$ is less than the smaller of $0.01A_g$ or 4 sq ft (0.37 m) and $A_{oi}/A_{gi} \le 0.2$ | Moderate | +0.18<br>−0.18 |
| Partially enclosed buildings | $A_o > 1.1A_{oi}$ and $A_o >$ the lesser of $0.01A_g$ or 4 sq ft (0.37 m) and $A_{oi}/A_{gi} \le 0.2$ | High | +0.55<br>−0.55 |
| Partially open buildings | A building that does not comply with Enclosed, Partially Enclosed, or Open classifications | Moderate | +0.18<br>−0.18 |
| Open buildings | Each wall is at least 80% open | Negligible | 0.00 |

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EXAMPLE

§ 26.12.1

## Enclosure Classification – General

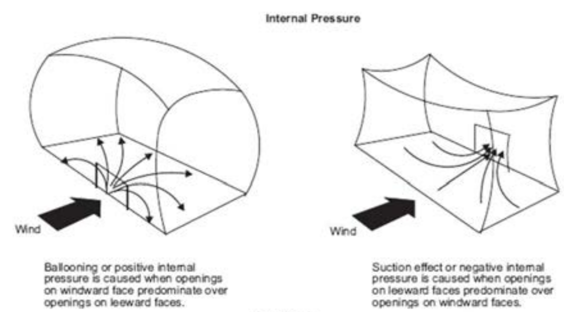


**Figure 15**
**Effects of Openings Source: Wind Design Overview Code Master**

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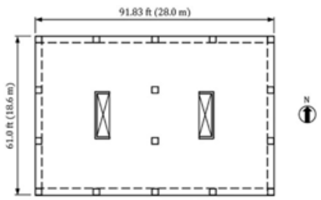
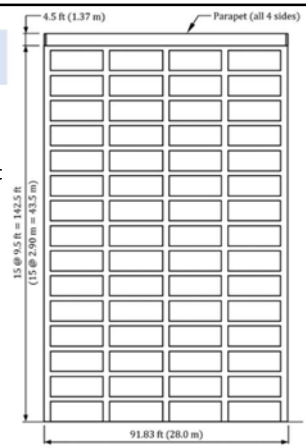
EXAMPLE

## Wind Load Parameters

- Basic wind speed: V = 105 mph
- Surface roughness: B
- Topography: Not on hill, ridge or escarpment
- Risk category: II
- Enclosed and flexible building



Plan



Elevation

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Example 5.12 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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EXAMPLE

## Wind Load Supplemental Data Request

| References* | Wind Load Parameters for MWFRS and C&C (ASCE 7 Ch 26) | Data |
|---|---|---|
| IBC §1604.5 | Building Risk Category: I, II, III, IV | II |
| IBC §1609.3 | Basic wind speed, $V$ (mph) [maps or asce7hazardtool.online] | 105 |
| ASCE 7 §26.6 | Wind directionality factor, $K_d$ | 0.85 |
| ASCE 7 §26.7 | Exposure category: B, C, D | B |
| ASCE 7 §26.8 | Topographic factor, $K_{zt}$ | 1.0 |
| ASCE 7 §26.9 | Ground elevation factor, $K_e$ | 1.0 |
| ASCE 7 §26.10.1 | Ht. above ground, z or h, (ft) and velocity pressure exposure coefficient, $K_z$ or $K_h$ | See table |
| ASCE 7 Eq 26.10-1 | Velocity pressure, $q_z$ or $q_h$ (psf) | See table |
| ASCE 7 §26.11 | Gust Effect Factor, $G$ or $G_f$ | 0.93 N-S<br>0.96 E-W |
| ASCE 7 §26.12 | Enclosure classification: enclosed, partially enclosed, partially open, open | Enclosed |
| ASCE 7 §26.13 | Internal pressure coefficient, $(GC_{pi})$ | ±0.18 |

*IBC 2024 and ASCE 7-22

$$q_z = 0.00256\, K_z\, K_{zt}\, K_e\, V^2 \quad (26.10\text{-}1)$$

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Example 5.12 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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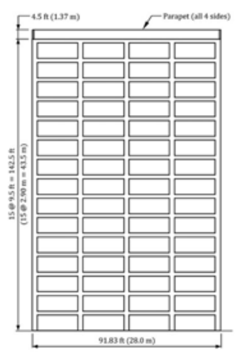
EXAMPLE

## Wind Load Supplemental Data Request



Elevation

| Height, z (ft) | $K_z$ | $q_z$ (psf) |
|---|---|---|
| 142.5 | 1.04 | 29.4 |
| 133.0 | 1.03 | 29.1 |
| 123.5 | 1.01 | 28.5 |
| 114.0 | 0.98 | 27.7 |
| 104.5 | 0.96 | 27.1 |
| 95.0 | 0.94 | 26.5 |
| 85.5 | 0.91 | 25.7 |
| 76.0 | 0.88 | 24.8 |
| 66.5 | 0.85 | 24.0 |
| 57.0 | 0.82 | 23.1 |
| 47.5 | 0.78 | 22.0 |
| 38.0 | 0.73 | 20.6 |
| 28.5 | 0.68 | 19.2 |
| 19.0 | 0.61 | 17.2 |
| 9.5 | 0.57 | 16.1 |

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Example 5.12 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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## ASCE 7 General Wind Provisions Change Summary

1. Basic wind speeds, $V$, updated
2. “Simple Diaphragm Building” and 2 simplified methods deleted
3. Wind directionality factor, $K_d$, moved from “q” to “p” calcs
4. Topographic factor, $K_{zt}$, updated
5. Velocity pressure exposure coefficient, $K_z$, revised

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## Wind Load Provisions – MWFRS Directional

| References* | Wind Load Parameters for MWFRS Loads – Buildings of all Heights [ASCE 7 Ch 27 – Directional] |
|---|---|
| ASCE 7 §26.6 | Directionality factor, $K_d$ |
| ASCE 7 §27.3.1 | External pressure coefficients, $C_p$ (enclosed, partially enclosed, and partially open buildings of all heights) |
| ASCE 7 §27.3.2 | External pressure coefficients, $C_N$ (open buildings with monoslope, pitched, or troughed free roofs) |
| ASCE 7 §27.3.3 | Roof overhangs |
| ASCE 7 §27.3.4 | Combined net pressure coefficient for parapets, $(GC_{pn})$ |
| ASCE 7 §27.3.5 | Controlling wind load Cases: 1, 2, 3, 4 |

*IBC 2024 and ASCE 7-22

$p = q\,K_d\,GC_p - q_i\,K_d\,(GC_{pi})$ (27.3-1) Not open

$p = q\,K_d\,GC_N$ (27.3-2) Open

$p_p = q_p\,K_d\,(GC_{pn})$ (27.3-3) Parapets

Ch 27-28

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CODE CHANGE

§ 27.3.1.1

## MWFRS: Elevated Buildings (Ch 27 – Directional)

The MWFRS loads for rigid or flexible elevated buildings meeting both of the following geometric limitations, for any principal wind direction, shall be determined in accordance with this section for that principal direction.

*(Additional text not shown)*

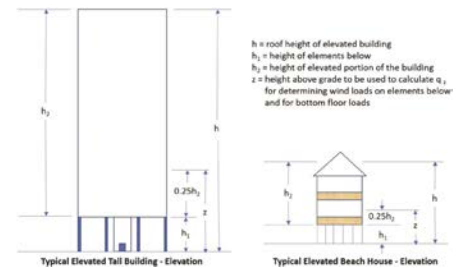


Figure C27.3-1 from *ASCE/SEI 7-22* – ASCE

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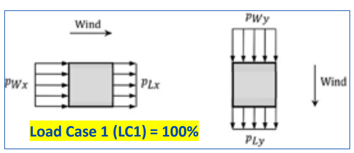
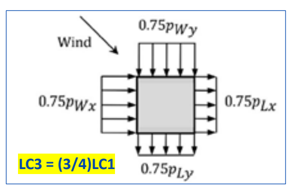
CODE CHANGE

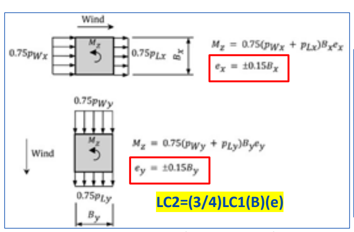
§ 27.3.5

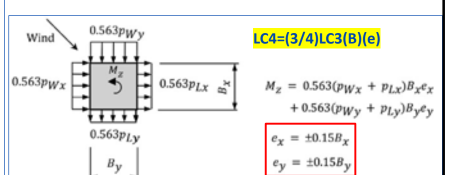
## MWFRS: Load Cases (Ch 27 – Directional)




- Includes roof pressures
- All heights




Excerpted from Figure 5.16 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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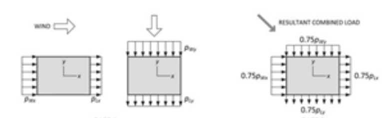
§ 27.3.5

## MWFRS: Load Cases (Ch 27 – Directional)

- Exceptions: buildings only requiring Case 1 & 3 (no torsion)
  - One-story with MRH ≤ 30 ft
  - ≤ 2 stories of light-frame
  - ≤ 2 stories with flexible diaphragms
  - Seismic controlled
  - Torsionally regular for wind
  - With flexible diaphragms designed ≥ 1.5 x Cases 1 and 3




Excerpt of Figure 27.3-8 from *ASCE/SEI 7-22* – ASCE

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## Wind Load Supplemental Data Request

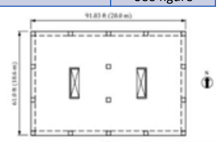
EXAMPLE

**N-S Direction** **(E-W not shown)**

| References* | Wind Load Parameters for MWFRS Loads (ASCE 7 Ch 27 – Directional) | Data |
|---|---|---|
| ASCE 7 §27.3 | Flat, gable, hip, monoslope and mansard roofs – not open (Fig 27.3-1) | |
| ASCE 7 F27.3-1 | Horizontal building dimension perp to wind, $B$ (ft) | 91.83 |
| | Horizontal building dimension parallel to wind, $L$ (ft) | 61.0 |
| | Roof slope, θ (degrees) | 0 |
| | Mean roof height, $h$ (ft) [eave height for $\theta \le 10°$] | 142.5 |
| | Height above ground, $z$ (ft) | See table |
| | Velocity pressure at respective height, $q_z$, $q_h$, or $q_p$ | See table |
| | External pressure coefficients, $C_p$ | F27.3-1 |
| | Combined net pressure coefficient for parapet, $(GC_{pn})$ | W=+1.5<br>L=-1.0 |
| ASCE 7 §27.3.5 | Wind load Cases: 1, 2, 3, 4 | See figure |

*IBC 2024 and ASCE 7-22

$$p = q\,K_d\,GC_p - q_i\,K_d\,(GC_{pi}) \qquad (27.3\text{-}1)$$




Example 5.12 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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## Wind Load Supplemental Data Request

EXAMPLE

**N-S Direction** **(E-W not shown)**

$$p = q\,K_d\,GC_p - q_i\,K_d\,(GC_{pi}) \quad (27.3\text{-}1)$$

| Building Surface | Height above Ground Level, z, ft (m) | q, lb/ft² (kN/m²) | External Pressure $qK_d\,GC_p$, lb/ft² (kN/m²) | Internal Pressure $q_iK_d\,(GC_{pi})$, lb/ft² (kN/m²) | Net Pressure, p, lb/ft² (kN/m²) | |
|---|---|---|---|---|---|---|
| | | | | | +(GC_pi) | −(GC_pi) |
| Windward wall | 142.5 (43.5) | 29.4 (1.41) | 18.6 (0.89) | ±4.5 (±0.22) | 14.1 (0.67) | 23.1 (1.11) |
| | 133.0 (40.6) | 29.1 (1.40) | 18.4 (0.88) | ±4.5 (±0.22) | 13.9 (0.66) | 22.9 (1.10) |
| | 123.5 (37.7) | 28.5 (1.37) | 18.0 (0.87) | ±4.5 (±0.22) | 13.5 (0.65) | 22.5 (1.09) |
| | 114.0 (34.8) | 27.7 (1.33) | 17.5 (0.84) | ±4.5 (±0.22) | 13.0 (0.62) | 22.0 (1.06) |
| | 104.5 (31.9) | 27.1 (1.30) | 17.1 (0.82) | ±4.5 (±0.22) | 12.6 (0.60) | 21.6 (1.04) |
| | 95.0 (29.0) | 26.5 (1.27) | 16.8 (0.80) | ±4.5 (±0.22) | 12.3 (0.58) | 21.3 (1.02) |
| | 85.5 (26.1) | 25.7 (1.23) | 16.3 (0.78) | ±4.5 (±0.22) | 11.8 (0.56) | 20.8 (1.00) |
| | 76.0 (23.2) | 24.8 (1.19) | 15.7 (0.75) | ±4.5 (±0.22) | 11.2 (0.53) | 20.2 (0.97) |
| | 66.5 (20.3) | 24.0 (1.15) | 15.2 (0.73) | ±4.5 (±0.22) | 10.7 (0.51) | 19.7 (0.95) |
| | 57.0 (17.4) | 23.1 (1.11) | 14.6 (0.70) | ±4.5 (±0.22) | 10.1 (0.48) | 19.1 (0.92) |
| | 47.5 (14.5) | 22.0 (1.06) | 13.9 (0.67) | ±4.5 (±0.22) | 9.4 (0.45) | 18.4 (0.89) |
| | 38.0 (11.6) | 20.6 (0.99) | 13.0 (0.63) | ±4.5 (±0.22) | 8.5 (0.41) | 17.5 (0.85) |
| | 28.5 (8.70) | 19.2 (0.92) | 12.1 (0.58) | ±4.5 (±0.22) | 7.6 (0.36) | 16.6 (0.80) |
| | 19.0 (5.80) | 17.2 (0.83) | 10.9 (0.52) | ±4.5 (±0.22) | 6.4 (0.30) | 15.4 (0.74) |
| | 9.5 (2.90) | 16.1 (0.77) | 10.2 (0.49) | ±4.5 (±0.22) | 5.7 (0.27) | 14.7 (0.71) |
| Leeward wall | All | 29.4 (1.41) | −11.6 (−0.56) | ±4.5 (±0.22) | −16.1 (−0.78) | −7.1 (−0.34) |
| Side walls | All | 29.4 (1.41) | −16.3 (−0.78) | ±4.5 (±0.22) | −20.8 (−1.00) | −11.8 (−0.56) |
| Roof | —* | 29.4 (1.41) | −24.2 (−1.16) | ±4.5 (±0.22) | −28.7 (−1.38) | −19.7 (−0.94) |

*From windward edge of roof to 61.0 ft (18.6 m).

Example 5.12 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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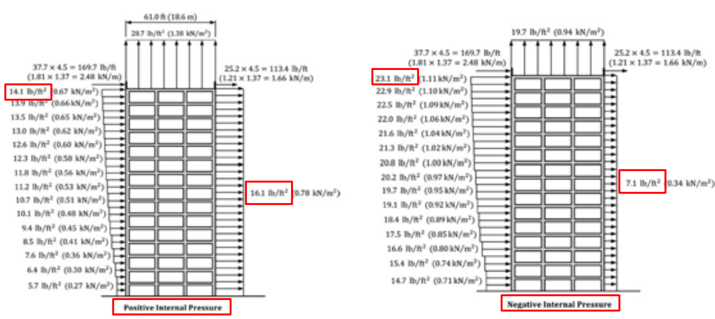
## Wind Load Supplemental Data Request

EXAMPLE

**N-S Direction** **(E-W not shown)**




Example 5.12 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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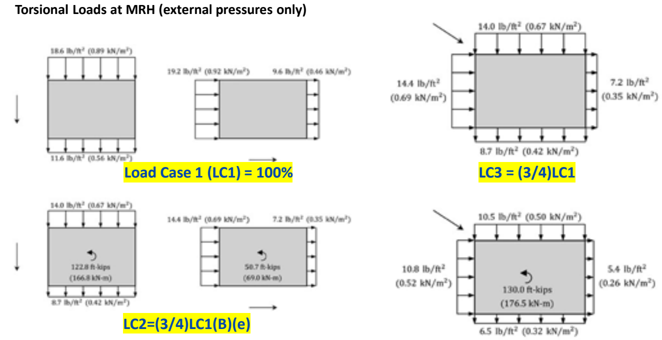
## Wind Load Supplemental Data Request

EXAMPLE

**Torsional Loads at MRH (external pressures only)**




Example 5.12 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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## Wind Load Provisions – MWFRS Envelope

| References* | Wind Load Parameters for MWFRS Loads – Low-Rise ($h \leq 60$ ft & $h \leq B$ & $h \leq L$) [ASCE 7 Ch 28 – Envelope] |
|---|---|
| ASCE 7 §26.6 | Directionality factor, $K_d$ |
| ASCE 7 §28.3.1 | External pressure coefficients, $(GC_{pf})$ (enclosed, partially enclosed, and partially open buildings) |
| ASCE 7 §28.3.7 | External pressure coefficients, $(GC_{pf})_{windward}$ & $(GC_{pf})_{leeward}$ (open or partially enclosed building with transverse frames and pitched roof (θ < 45°) |
| ASCE 7 §28.3.5 | Roof overhangs |
| ASCE 7 §28.3.4 | Combined net pressure coefficient for parapets, $(GC_{pn})$ |
| ASCE 7 §28.3.2 | Controlling wind load Cases: 1, 2, 3, 4 |

*IBC 2024 and ASCE 7-22

Ch 28

$p = q_h K_d [(GC_{pf}) - (GC_{pi})]$ (28.3-1) Not open

$p_p = q_p K_d (GC_{pn})$ (28.3-2) Parapets

$p = q_h K_d [(GC_{pf})_{windward} - (GC_{pf})_{leeward}] K_B K_S$ (28.3-3) Transverse frames

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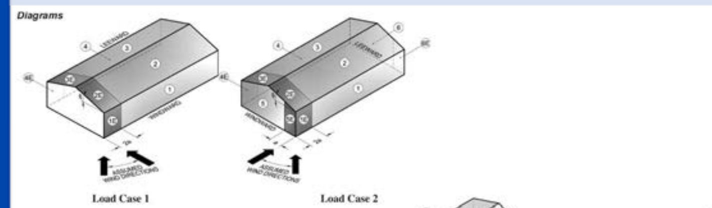
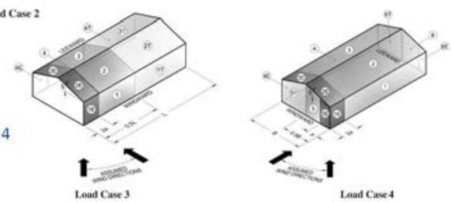
## MWFRS: Load Cases (Ch 28 – Low-rise)

CODE CHANGE

§ 28.3.2




- 8 graphics consolidated to 2
- Torsional loads are now Cases 3 & 4
- No changes to coefficients




Excerpted from Figure 5.17 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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## MWFRS: Ground-Mounted Fixed-Tilt Solar Panel Systems

CODE CHANGE

§ 29.4.5

Wind loads shall be calculated in accordance with Sections 29.4.5.2 and 29.4.5.3 for ground-mounted fixed-tilt solar photovoltaic (PV) panel systems installed in rows satisfying the following geometric limitations…

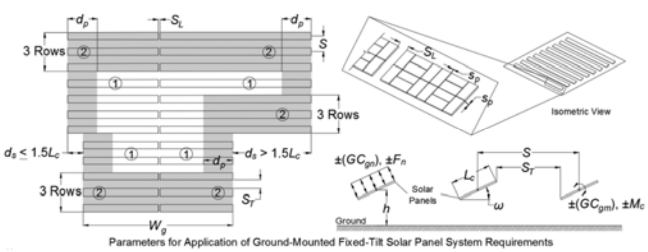


Figure 29.4-9 from *ASCE/SEI 7-22* – ASCE

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## Wind Load Supplemental Data Request

| References* | Wind Load Parameters for MWFRS Loads (ASCE 7 Ch 29 – Appurtenances) | Data |
|---|---|---|
| ASCE 7 §29.4.5 | Max loads on ground-mounted fixed-tilt solar PV systems | |
| | Effective wind area, $A$ (ft$^2$) | |
| ASCE 7 §29.4.5.1** | Panel chord length: 6 ft $\leq L_c \leq$ 14 ft \| Y N | |
| ASCE 7 §29.4.5.1** | Shortest row length in an array, $W_g$: $W_g / L_c \geq 7$ \| Y N | |
| ASCE 7 §29.4.5.1** | Angle between solar panel and ground, $\omega$: $0° \leq \omega \leq 60°$ \| Y N | |
| ASCE 7 §29.4.5.1** | Mean height of panel, $h$: $0.5 \leq h / L_c \leq 0.8$ \| Y N | |
| ASCE 7 §29.4.5.1** | Center-to-center row spacing, $S$: $0.2 \leq L_c / S \leq 0.6$ \| Y N | |
| ASCE 7 §29.4.5.1** | Gap between adjacent panels in both directions, $s_p$: $s_p \leq 0.014L_c$ \| Y N | |
| ASCE 7 §29.4.5.1** | Horizontal longitudinal distance of open area within a single row, $S_L$: $S_L \leq 0.25L_c$ \| Y N | |
| ASCE 7 §29.4.5.1** | Horizontal transverse distance of open area between adjacent rows, $S_T$: $S_T \leq 2S$ \| Y N | |
| ASCE 7 Eq 29.4-10 | Combined static and dynamic net pressure coefficient, $(GC_{gn})$ | |
| ASCE 7 Eq 29.4-11 | Combined static and dynamic net pressure moment coefficient, $(GC_{gm})$ | |
| ASCE 7 §29.4.5.2 | Design wind force, $F_n$ (lb) per Eq. 29.4-8 | |
| ASCE 7 §29.4.5.2 | Design moment, $M_c$ (plf) per Eq. 29.4-9 | |

*IBC 2024 and ASCE 7-22
**Must be Yes to proceed

$F_n = q_h K_d [\pm(GC_{gn})] A$ (29.4-8)

$M_c = q_h K_d [\pm(GC_{gm})] A L_c$ (29.4-9)

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## ASCE 7 MWFRS Provisions Change Summary

1. New provisions for "Elevated Buildings" added
2. Load cases clarified for directional procedure (Ch 27)
3. Load cases clarified for low-rise procedure (Ch 28)
4. New provisions for ground-mounted fixed-tilt solar panel systems added

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## Wind Load Provisions – C&C for Buildings

| References* | Tornado Load Parameters for C&C Loads (ASCE 7 Ch 30) |
|---|---|
| ASCE 7 §26.6 | Directionality factor, $K_d$ |
| | Effective wind area, $A$ |
| | External pressure coefficients and design pressure, $p$ |
| ASCE 7 §30.3 (Part 1) | Buildings h ≤ 60 ft or low-rise ($GC_p$) |
| ASCE 7 §30.4.2 (Part 2) | Buildings h > 60 ft ($GC_p$) |
| ASCE 7 §30.5.2 (Part 3) | Open buildings ($C_N$) |
| ASCE 7 §30.6 (Part 4) | Parapets ($GC_p$) |
| ASCE 7 §30.7 (Part 5) | Overhangs ($GC_p$) |
| ASCE 7 §30.12 | Pavers ($GC_{Lnet}$) |

*IBC 2024 and ASCE 7-22

$p = q_h \underline{K_d} [(GC_p) - (GC_{pi})]$ (30.3-1) h ≤ 60 ft or low-rise

$p = q \underline{K_d} (GC_p) - q_i \underline{K_d} (GC_{pi})$ (30.4-1) h > 60 ft

$p = q_h \underline{K_d} G C_N$ (30.5-1) Open buildings

$p = q_p \underline{K_d} [(GC_p) - (GC_{pi})]$ (30.6-1) Parapets

$p = q_h \underline{K_d} [(GC_p) - (GC_{pi})]$ (30.7-1) Overhangs

$p = q_h \underline{K_d} GC_{Lnet}$ (30.12-1) Pavers

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## C&C: Building Types

CODE CHANGE

§ 30.1.1

| Part | Applicability: Building/Element Type | | Applicability: Ht. Limit | Conditions |
|---|---|---|---|---|
| 1 | Enclosed<br>Enclosed, low-rise<br>Partially enclosed<br>Partially enclosed, low-rise<br>Partially open<br>Partially open, low-rise | | $h \leq 60$ ft | • Building has flat, gable, multispan gable, hip, monoslope, stepped, or sawtooth roof<br>• Wind pressures are determined from a wind pressure equation |
| 2 | Enclosed<br>Partially enclosed<br>Partially open | | $h > 60$ ft | • Building has a flat, pitched, gable, hip, mansard, arched or domed roof<br>• Wind pressures are determined from a wind pressure equation |
| 3 | Open | | None | Building has a pitched, monoslope, or troughed free roof |
| 4 | Building appurtenances and rooftop structures and equipment | | None | See ASCE/SEI 30.8 through 30.11 for additional conditions for various element types |
| 5 | Nonbuilding structures | Circular bins, silos, and tanks | $h < 120$ ft | — |
| 5 | Nonbuilding structures | Rooftop solar panels | None | Buildings with flat roofs or with gable or hip roofs with roof slopes ≤ 7º |
| 5 | Nonbuilding structures | Rooftop pavers | None | Buildings with roof slopes ≤ 7 degrees |

- Simplified approaches eliminated (previously Parts 2 & 4)

Adapted from Table 5.17 of *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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## C&C Wind Load Changes

CODE CHANGE

§ 30.3

Part 1: $h \leq 60$ ft or low-rise buildings

| Figures | Change | Table Description |
|---|---|---|
| 30.3-1A | Added | Bottom horizontal surface of elevated buildings |
| 30.3-2B through 30.3-2G | Significantly revised | Gable and hip roofs (θ > 7°) |
| 30.3-2H and 30.3-2I | Removed | Hip roofs (27° < θ ≤ 45°) Roof and Overhang |
| 30.3-3 | Significantly revised | Stepped roofs (θ ≤ 7°) |
| 30.3-4 through 30.3-7 | Added "partially open buildings" | Multispan gable, monoslope, sawtooth, domed roofs |
| 30.3-8 | Added | Arched roofs |

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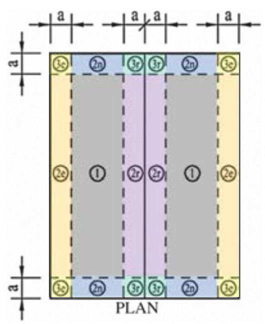
CODE CHANGE

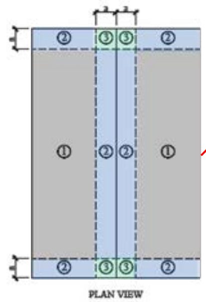
§ 30.3 – P1

## C&C Wind Load Simplifications

**ASCE 7-16**




**ASCE 7-22**




Overhangs addressed separately

Gable Roof 7° < θ < 20° (Figure 30.3-2B)

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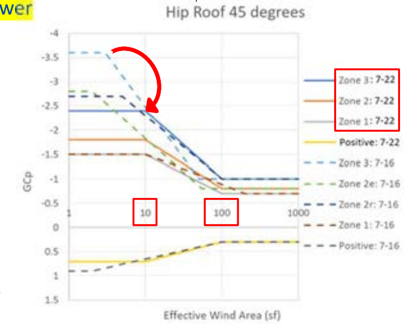
CODE CHANGE

§ 30.3 – P1

## C&C Wind Load Simplifications

- Pressures generally the same or lower
- Only 3 zones on gables and hips
- Lower EWA $\geq$ 10 ft²
- Upper EWA
  - 100 ft², 200 ft² or 300 ft²
- Eliminated *h/B* ratios for hip roofs
  - *h/B* = roof height / least horizontal dimension
- Eliminated $(GC_p)$ equations for hip roofs of 27° to 45°
  - New graph for 45° only (Fig 30.3-2G)
  - Interpolation equations still available
    - Updated in the ASCE 7-22 commentary




Adapted from *STRUCTURE* Magazine – Oct 2022: "ASCE 7-22 Changes to Component and Cladding Wind Provisions"

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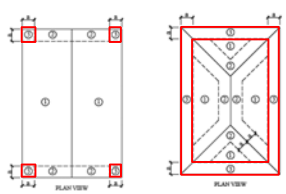
EXAMPLE

## C&C Roof Pressure Comparison

- Location: Kansas City, MO
- V = 110 mph
- Exposure B
- Mean roof height = 20 feet (low-rise)
- Partially open building
- Elevation = 897 feet
- Velocity pressure = 15.8 psf




| Roof Shape | Slope (degrees) | Zone | EWA (ft²) | ASCE 7-16 $(GC_p)$ | ASCE 7-22 $(GC_p)$ | ASCE 7-16 C&C Pressure (psf) | ASCE 7-22 C&C Pressure (psf) | 7-22/ 7-16 |
|---|---|---|---|---|---|---|---|---|
| Gable | 25 | 3 | 4 | -3.6 | -3.0 | -59.7 | -50.2 | 0.85 |
| Gable | 40 | 3 | 100 | -1.5 | -1.3 | -26.5 | -23.4 | 0.88 |
| Hip | 25 | 3 | 100 | -1.23 | -1.0 | -22.3 | -18.6 | 0.83 |
| Hip | 35 | 3 | 20 | -2.53 | -1.82 | -42.8 | -31.6 | 0.74 |
| Hip | 45 | 3 | 3 | -3.6 | -2.4 | -59.7 | -40.8 | 0.68 |

*STRUCTURE* Magazine – Oct 2022: "ASCE 7-22 Changes to Component and Cladding Wind Provisions"

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CODE CHANGE

§ 30.3.2.1

**C&C: Bottom Horizontal Surface of Elevated Buildings (h ≤ 60 ft)**

Design wind pressures for C&C elements on the bottom flat horizontal surface of elevated buildings shall be determined using the roof pressure coefficients...

*(Additional text not shown)*

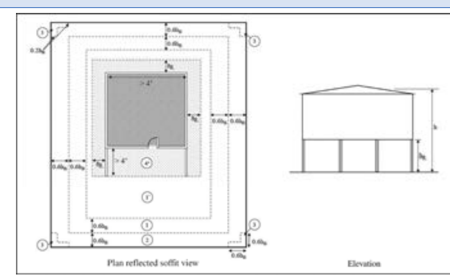


Figure 30.3-1A from *ASCE/SEI 7-22* – ASCE

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CODE CHANGE

§ 30.4.2.1

**C&C: Bottom Horizontal Surface of Elevated Buildings (h > 60 ft)**

Design wind pressures for C&C elements on the bottom flat horizontal surface of elevated buildings shall be determined using the roof pressure coefficients…

*(Additional text not shown)*

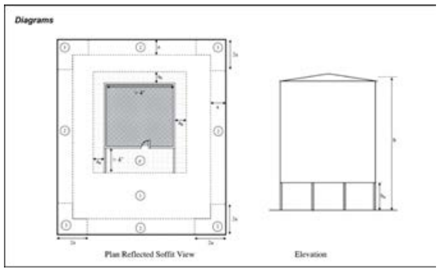


Figure 30.4-1A from *ASCE/SEI 7-22* – ASCE

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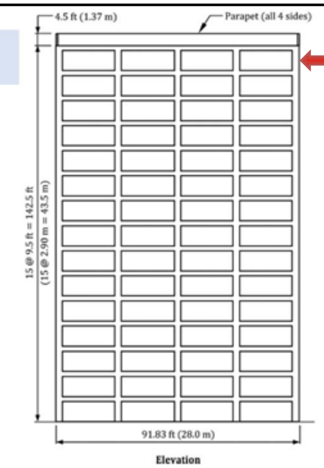
EXAMPLE

## C&C Wind Pressures

- Cladding on 15th floor
- Mullions
  - 9.5 ft span
  - 5 ft spacing




Example 5.21 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

75

75

EXAMPLE

## Wind Load Supplemental Data Request

| References* | Wind Load Parameters for C&C Loads (ASCE 7 §30.4 – Part 2: h > 60 ft) | Data | | | | | |
|---|---|---|---|---|---|---|---|
| ASCE 7 §30.4.2 | External pressure coefficients and design pressures (psf) | Zone | | | | 4 | 5 |
| ASCE 7 F30.4-1 | Walls and Roofs ≤ 7°: Zones 1, 2, 3, 4, 5 | $(GC_p)$ | | | | +0.82<br>-0.85 | +0.82<br>-1.59 |
| | | $p$ | | | | 25.0<br>25.0 | -25.8<br>-44.3 |
| ASCE 7 F30.4-1A and F30.4-1 | Bottom horizontal surface of elevated buildings: Zones 1, 2, 3 | $(GC_p)$ | | | | | |
| | | $p$ | | | | | |
| ASCE 7 F30.3-2A to 2I, 30.3-5A & 30.3-5B | Roofs > 7° and other roof geometries: Zones 1, 2, 3 | $(GC_p)$ | | | | | |
| | | $p$ | | | | | |
| ASCE 7 F30.3-7 | Domed roofs: Negative and Positive | $(GC_p)$ | | | | | |
| | | $p$ | | | | | |
| ASCE 7 F30.3-8 | Arched roofs: Zones A, B, C | $(GC_p)$ | | | | | |
| | | $p$ | | | | | |

*IBC 2024 and ASCE 7-22

$$p = q\,K_d\,(GC_p) - q_i\,K_d\,(GC_{pi}) \qquad (30.4\text{-}1)$$

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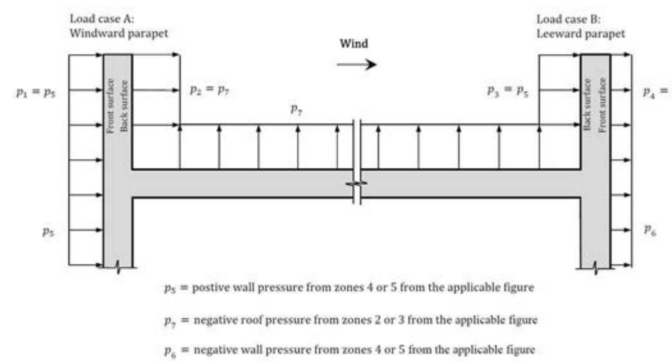
## C&C: Parapets

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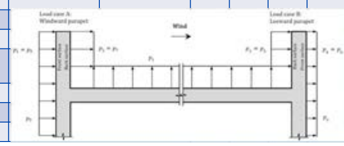
EXAMPLE

## Wind Load Supplemental Data Request

| References* | Wind Load Parameters for C&C Loads (ASCE 7 §30.6 – Part 4: Parapets) | Data | | | | | |
|---|---|---|---|---|---|---|---|
| ASCE 7 §30.6 | External pressure coefficients and design pressures (psf) | *($GC_p$)* | | $p_1$ | $p_2$ | $p_3$ | $p_4$ |
| ASCE 7 F30.3-1 | Walls with h ≤ 60’: Zones 4 & 5 | | | | | | |
| ASCE 7 F30.4-1 | Walls with h > 60’: Zones 2, 3, 4 & 5 | A: -3.2<br>B: -1.8 | A: 0.9<br>B: 0.9 | 18.2<br>27.2 | -85.1<br>-76.1 | 18.2<br>27.2 | -49.9<br>-40.9 |
| ASCE 7 F30.3-2A to F30.3-2C | Flat, Gable and Hip Roofs: Zones 2, 3 | | | | | | |
| ASCE 7 F30.3-3 | Stepped Roofs: Zones 2, 3 | | | | | | |
| ASCE 7 F30.3-4 | Multispan gable roofs: Zones 2, 3 | | | | | | |
| ASCE 7 F30.3-5A and F30.3-5B | Monoslope Roofs: Zones 2, 2’, 3 | | | | | | |
| ASCE 7 F30.3-6 | Sawtooth roofs: Zones 2, 3A, 3 (BCD) | | | | | | |
| ASCE 7 F30.3-7 | Domed roofs – all heights: negative and positive | | | | | | |
| ASCE 7 F30.3-8 | Arched roofs: Zone C | | | | | | |



*IBC 2024 and ASCE 7-22

$$p = q_p K_d [(GC_p) - (GC_{pi})] \qquad (30.6\text{-}1)$$

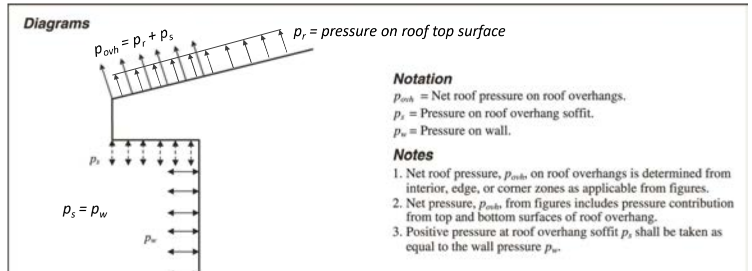
80

80

CODE CHANGE

§ 30.7

## C&C: Roof Overhangs




Adapted from Figure 30.7-1 from *ASCE/SEI 7-22* – ASCE

- Eliminated graphs for overhangs on gable roofs with slopes > 7°

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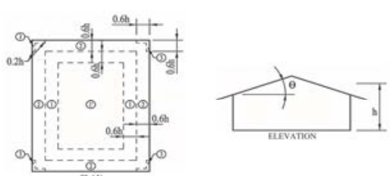
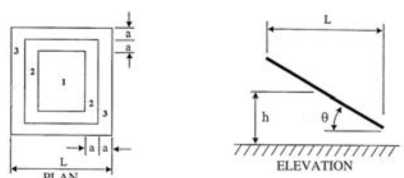
CODE CHANGE

§ 30.12

## Rooftop Pavers for All Heights with Roof <7°




h ≤ 60’




h > 60’

$$p = q_h K_d GC_{Lnet} \qquad (30.12\text{-}1)$$

83

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## ASCE 7 C&C Provisions Change Summary

1. “Partially open” buildings added for low-rise and all heights
2. *($GC_p$)* coefficient graphs simplified for h ≤ 60’ and low-rise (Part 1)
3. New provisions for “Elevated Buildings” added for all heights
4. Eliminated graphs for overhangs on gable roofs with slopes > 7°
5. Added procedures for rooftop pavers for roofs < 7°

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# Tornado Loads

Risk Category III and IV structures

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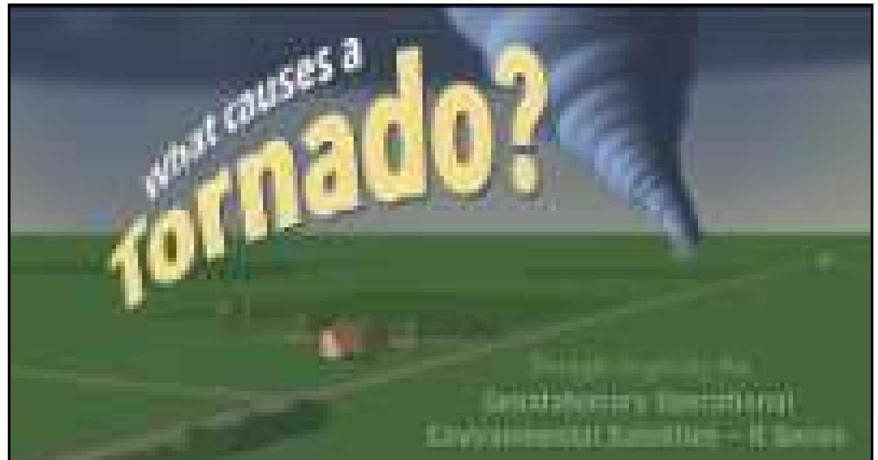


https://www.youtube.com/watch?v=LT7yRMLAkCY

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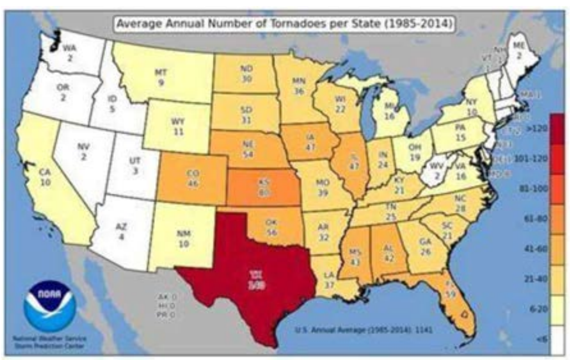
## Why is this important?




88

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## Why is this important?

Reported tornadoes from 1995 to 2016

| Scale | Wind Speed (mph) | Damage |
|---|---|---|
| EF0 | 65–85 | Light damage |
| EF1 | 86–110 | Moderate damage |
| EF2 | 111–135 | Considerable damage |
| EF3 | 136–165 | Severe damage |
| EF4 | 166–200 | Devastating damage |
| EF5 | >200 | Incredible damage |

89%

97%

89

89



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CODE CHANGE

§ 1609.5

### Tornado Loads

The design and construction of Risk Category III and IV buildings and other structures located in the tornado-prone region as shown in Figure 1609.5 shall be in accordance with Chapter 32 of ASCE 7, except as modified by this code.

- Risk Category III and IV buildings
- Tornado-prone region
- ASCE 7 Chapter 32

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CODE CHANGE

§ 1603.1.4

### Construction Documents

**Wind and tornado design data.** The following information related to wind loads and where required by Section 1609.5 tornado loads shall be shown, regardless of whether wind or tornado loads govern the design of the lateral force-resisting system of the structure:

1. Basic ~~design wind~~ wind speed, V (mph), **tornado speed, $V_T$ (mph)**, ~~miles per hour~~ and allowable stress design wind speed, $V_{asd}$ (mph), as determined in accordance with Section 1609.3.1.
2. Risk category.
3. **Effective plan area, $A_e$, for tornado design** in accordance with Chapter 32 of ASCE 7.

~~3~~ 4. Wind exposure. Applicable wind direction if more than one wind exposure is utilized.

~~4~~ 5. Applicable internal pressure ~~coefficient~~ coefficients, and **applicable tornado internal pressure coefficients**.

~~5~~ 6. Design wind pressures and their applicable zones with dimensions to be used for exterior component and cladding materials not specifically designed by the registered design professional responsible for the design of the structure, pounds per square foot. **Where design for tornado loads is required, the design pressures shown shall be the maximum of wind or tornado pressures.**

- Tornado design data added

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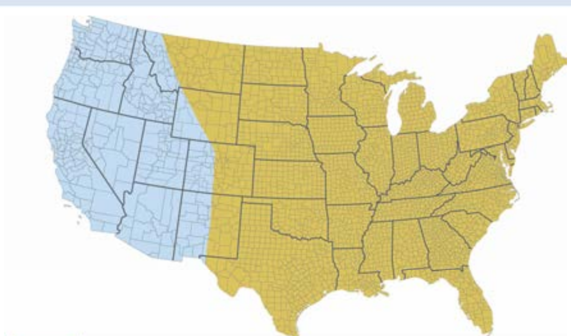
CODE CHANGE

§ 1609.5

## Tornado-prone Region



Tornado-prone region

Areas outside tornado-prone region

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CODE CHANGE

§ 32.1.1

### Tornado Loads – Scope

Buildings and other structures classified as Risk Category III or IV and located in the tornado-prone region as shown in Figure 32.1-1, including the main wind force resisting system (MWFRS) and all components and cladding (C&C) thereof, shall be designed and constructed to resist the greater of the tornado loads determined in accordance with the provisions of this chapter or the wind loads determined in accordance with Chapters 26 through 31, using the load combinations provided in Chapter 2.

- MWFRS and C&C
  - Design for greater of tornado loads or wind loads

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§ 2.3.3 & 2.4.3

## Load Combinations

**Strength Design**

1.2D + 1.0(W or $W_T$) + L + (0.5$L_r$ ~~or 0.3S~~ or 0.5R) (Eq 4a)

0.9D + 1.0 (W or $W_T$) (Eq 5a)

**ASD**

D + 0.6(W or $W_T$) (Eq 5a)

D + 0.75L + 0.75(0.6(W or $W_T$)) + 0.75($L_r$ ~~or 0.7S~~ or R) (Eq 6a)

0.6D + 0.6(W or $W_T$) (Eq 7a)

$W_T$ = load due to tornado pressure

Snow ignored if $W_T$ controls – tornados are warm-weather events

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CODE CHANGE

§ 32.1.2

## Design Procedures for Tornado Loads

| Element | Permitted Procedure | Modified by |
|---|---|---|
| MWFRS | Directional procedure – buildings of all heights (Ch 27) | § 32.15 |
| | Directional procedure – appurtenances (Ch 29) | § 32.16 |
| | Wind tunnel (Ch 31) | § 32.18 |
| C&C | Parts 1 through 5 (Ch 30) | § 32.17 |
| | Wind tunnel (Ch 31) | § 32.18 |

- Envelope (low-rise) procedure (Ch 28) not permitted
  - Tornado gusts are likely much smaller than standard atmospheric boundary layer gusts

95

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CODE CHANGE

§ 32.1.1

## Not for Storm Shelter Design

… A building or other structure designed for tornado loads determined exclusively in accordance with **Chapter 32 cannot be designated as a storm shelter without meeting additional critical requirements** provided in the applicable building code and ICC 500, the ICC/NSSA *Standard for the Design and Construction of Storm Shelters*…


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EXAMPLE

§ 423.4

## Storm Shelters

- V = 250 mph (red zone)
- Critical services → RC IV
  - 911 call stations
  - Emergency ops centers
  - Fire, rescue, ambulance and police stations
- Storm shelter per ICC 500


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§ 32.5.1

## Tornado Speed, *V*

Example: Risk Category IV for 4 million ft² area

- Risk Category III
  - 8 maps
- Risk Category IV
  - 8 maps
- Based on effective plan area, $A_e$
  - 1 ft² to 4 million ft²

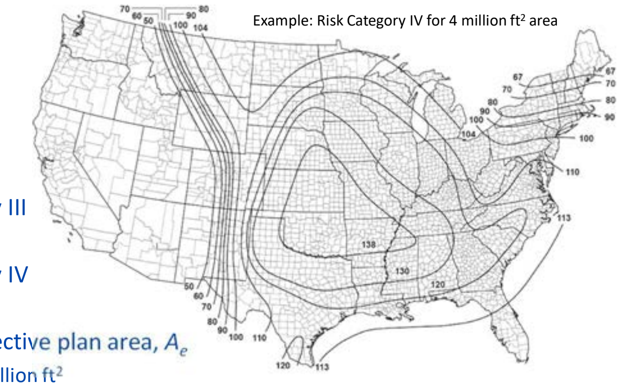

Figure 32.5-2H from *ASCE/SEI 7-22* – ASCE

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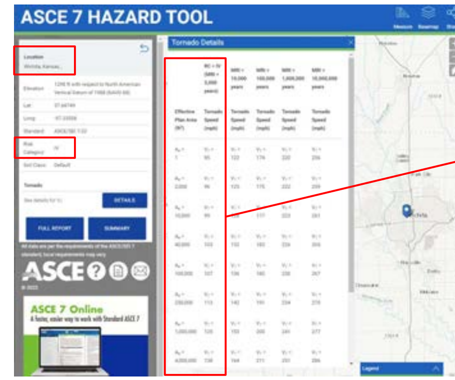
EXAMPLE

## asce7hazardtool.online



$A_e$ = 1 $V_T$ = 95

$A_e$ = 2,000 $V_T$ = 96

$A_e$ = 10,000 $V_T$ = 99

$A_e$ = 40,000 $V_T$ = 103

$A_e$ = 100,000 $V_T$ = 107

$A_e$ = 250,000 $V_T$ = 113

$A_e$ = 1,000,000 $V_T$ = 125

$A_e$ = 4,000,000 $V_T$ = 138

100

100

CODE CHANGE

§ 32.5.2

## Design for Tornado Loads Not Required

$V_T$ < 60 mph

Where $V_T \geq 60$ mph but less than
- Exposure B: $V_T < 0.50V$
- Exposure C: $V_T < 0.60V$
- Exposure D: $V_T < 0.67V$

| $V_T$ (mph) | V (mph) Exposure | | |
|---|---|---|---|
| | B | C | D |
| **60** | 120 | 100 | 90 |
| **70** | 140 | 117 | 104 |
| **80** | 160 | 133 | 119 |
| **90** | 180 | 150 | 134 |
| **100** | 200 | 167 | 149 |
| **110** | - | 183 | 164 |
| **120** | - | 200 | 179 |
| **130** | - | - | 194 |
| **140** | - | - | - |

- $V_T$ = tornado speed (Chapter 32)
- $V$ = basic wind speed (Chapter 26)
- Exposure category (Chapter 26)
  - Only used to determine applicability – not to determine loads

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Ch 32

## Tornado Loads – General Requirements

| References* | Tornado Loads: General Requirements (for both MWFRS and C&C) [ASCE 7 Ch 32] |
|---|---|
| ASCE 7 §32.5.1 or asce7hazardtool.online | Tornado speed, $V_T$ |
| ASCE 7 §32.5.4 | Effective plan area, $A_e$ |
| ASCE 7 §32.6 | Directionality factor, $K_{dT}$ |
| ASCE 7 §32.9 | Ground elevation factor, $K_e$ |
| ASCE 7 §32.10 | Tornado velocity pressure exposure coefficient, $K_{zTor}$ or $K_{hTor}$ |
| ASCE 7 §32.11 | Tornado Gust Effect Factor, $G_T$ |
| ASCE 7 §32.12 | Enclosure classification |
| ASCE 7 §32.13 | Tornado Internal pressure coefficient, $(GC_{piT})$ |
| ASCE 7 §32.14 | Tornado pressure coefficient adjustment factor, $K_{vT}$ |

- Exposure category and topographic factor not used

*IBC 2024 and ASCE 7-22

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§ 32.5.4

## Effective Plan Areas, $A_e$

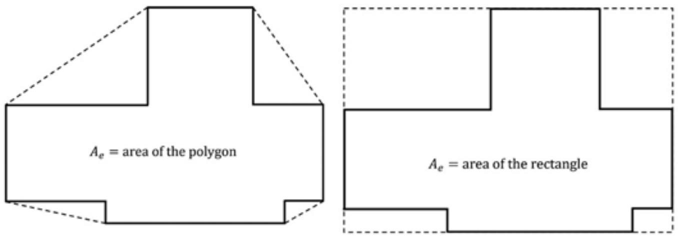


Figure 5.23 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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CODE CHANGE

§ 32.5.4

## Effective Plan Areas, $A_e$

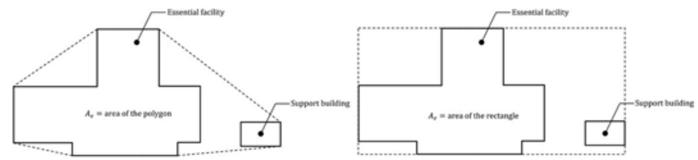


Figure 5.24 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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CODE CHANGE

§ 32.6

## Tornado Directionality Factor, $K_{dT}$

| Structure Type | Tornado Directionality Factor, $K_{dT}$ | |
|---|---|---|
| **Buildings** | | |
| Main wind force resisting system | 0.80 | |
| Components and cladding | | |
| For Essential Facilities and for buildings and other structures required to maintain the functionality of Essential Facilities | 1.0 | Risk Category IV |
| Roof Zone 1' as shown in Figure 30.3-2A | 0.9 | Risk Category III |
| All other cases | 0.74 | Risk Category III |
| **Arched roofs, circular domes, and all other structures** | ASCE 7 Table 26.6-1 | |

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CODE CHANGE

T 32.10-1

## Tornado Velocity Pressure Coefficient, $K_{zTor}$

$$K_{zTor} = \begin{cases} 1.0 & \text{for } 0 < z \le 200 \text{ ft} \\ [(2{,}820 - z)/2{,}620]^2 & \text{for } 200 \text{ ft} < z \le 328 \text{ ft} \\ 0.90 & \text{for } z > 328 \text{ ft} \end{cases}$$

| Ht above Ground *z* or *h* | $K_z$ (Table 26.10-1) Exposure Category | | $K_{zTor}$ |
|---|---|---|---|
| ft | B | C | n/a |
| **0–15** | 0.57 (0.70)* | 0.85 | 1.00 |
| **20** | 0.62 (0.70)* | 0.90 | 1.00 |
| **25** | 0.66 (0.70)* | 0.94 | 1.00 |
| **30** | 0.70 | 0.98 | 1.00 |
| **40** | 0.74 | 1.04 | 1.00 |
| **50** | 0.79 | 1.09 | 1.00 |
| **60** | 0.83 | 1.13 | 1.00 |
| **70** | 0.86 | 1.17 | 1.00 |
| **80** | 0.90 | 1.21 | 1.00 |
| **90** | 0.92 | 1.24 | 1.00 |
| **100** | 0.95 | 1.26 | 1.00 |
| **120** | 1.00 | 1.31 | 1.00 |
| **140** | 1.04 | 1.34 | 1.00 |
| **160** | 1.08 | 1.39 | 1.00 |
| **180** | 1.11 | 1.41 | 1.00 |
| **200** | 1.14 | 1.44 | 1.00 |
| **250** | 1.21 | 1.51 | 0.96 |
| **300** | 1.27 | 1.57 | 0.92 |
| **350** | 1.33 | 1.62 | 0.90 |

**Only applies to buildings ≥ 250 ft** $\Big\{$

* Use 0.70 in Chapter 28, Exposure B, when $z$ < 30 ft

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T 32.13-1

## Tornado Internal Pressure Coefficients ($GC_{piT}$)

Chapter 26 for wind loads ↓

| Enclosure Classification | Internal Pressure Coefficients ($GC_{pi}$) | Tornado Internal Pressure Coefficients ($GC_{piT}$) |
|---|---|---|
| Sealed other structures | n/a | +1.0 |
| Enclosed buildings and other structures | +0.18 / −0.18 | +0.55 / −0.18 |
| Partially enclosed buildings and other structures | +0.55 / −0.55 | +0.55 / −0.55 |
| Partially open buildings and other structures | +0.18 / −0.18 | +0.18 / −0.18 |
| Open buildings and other structures | 0.00 | 0.00 |

For MWFRS and C&C (all heights)

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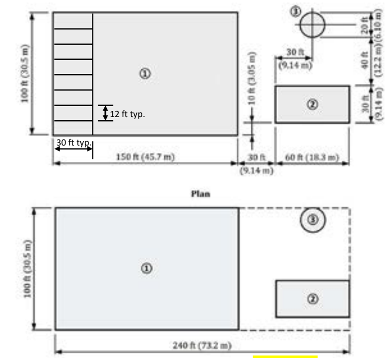
EXAMPLE

## Tornado Design

- Location: Springfield, MO
- Surface roughness: B
- Topography: Not situated on a hill, ridge, or escarpment
- 5-story hospital ①
  - Flat roof
  - 10-ft story height
  - No parapets or overhangs
- 1-story emergency power backup building ②
- Tank for hospital function ③




$A_e$ = (150 + 30 + 60) x 100 = 24,000 ft²

Example 5.26 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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EXAMPLE

## Is Tornado Design Required?

- ✓ Risk Category IV
- ✓ Tornado-prone region
- ✓ $V_T$ = 103 mph > 60 mph
- ✓ $V_T$ > ½ V
  - V = 120 mph Exposure B

| $V_T$ (mph) | V (mph) Exposure B | C | D |
|---|---|---|---|
| 60 | 120 | 100 | 90 |
| 70 | 140 | 117 | 104 |
| 80 | 160 | 133 | 119 |
| 90 | 180 | 150 | 134 |
| 100 | 200 | 167 | 149 |
| 110 | - | 183 | 164 |
| 120 | - | 200 | 179 |
| 130 | - | - | 194 |
| 140 | - | - | - |

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EXAMPLE

## Tornado Load Supplemental Data Request

| References* | Tornado Load Parameters for MWFRS and C&C (ASCE 7 Ch 32) | Data |
|---|---|---|
| IBC §1604.5 | Building Risk Category: I, II, III, IV | IV |
| ASCE 7 §32.5.4.1 | Effective plan area, $A_e$ (ft²) | 24,000 |
| ASCE 7 §32.5 | Tornado speed, $V_T$ (mph) [maps or asce7hazardtool.online] | 103 |
| ASCE 7 §26.9 | Ground elevation factor, $K_e$ | 1.0 |
| ASCE 7 §32.10.1 | Height z or h (ft) and tornado velocity pressure exposure coefficient, $K_{zTor}$ or $K_{hTor}$ | 1.0 |
| ASCE 7 Eq 32.10-1 | Tornado velocity pressure, $q_{zT}$ or $q_{hT}$ (psf) | 27.2 |
| ASCE 7 §32.11.1 | Tornado gust effect factor, $G_T$ = 0.85 (or per Eq 26.11-6 using Exp C) | 0.85 |
| ASCE 7 §32.12.2 | Enclosure classification: enclosed, partially enclosed, partially open, open, sealed | Enclosed |
| ASCE 7 §32.13 | Internal pressure coefficient, $GC_{piT}$ | +0.55<br>-0.18 |

*IBC 2024 and ASCE 7-22

$$q_{zT} = 0.00256\, K_{zTor}\, K_e\, V_T^2 \qquad (32.10\text{-}1)$$

Example 5.26 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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T 32.14-1

## Tornado Pressure Coefficient Adjustment Factor for Vertical Winds, $K_{vT}$

| STRUCTURE TYPE | $K_{vT}$ |
|---|---|
| **Buildings** | |
| **Negative (Uplift) Pressures on Roofs** | |
| MWFRS | 1.1 |
| C&C | |
| Roof slope ≤ 7° | |
| Zone 1 | 1.2 |
| Zone 2 | 1.05 |
| Zone 3 | 1.05 |
| Roof slope > 7° | |
| Zone 1 | 1.2 |
| Zone 2 | 1.2 |
| Zone 3 | 1.3 |
| **Positive Pressures (Downward Acting) on Roofs** | 1.0 |
| **Wall Pressures** | 1.0 |
| **All Other Cases** | 1.0 |
| **Other Structures** | |
| **Negative (Uplift) Pressures on Rooftop Structures and Equipment and Rooftop Solar Panels Parallel to the Roof Surface** | |
| MWFRS | 1.1 |
| C&C | Use values for building C&C |
| **Negative (Uplift) Pressures on Roofs of Bins, Silos, and Tanks** | |
| MWFRS | 1.1 |
| C&C | See Section 32.17.5 |
| **All Other Cases** | 1.0 |

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CODE CHANGE

Ch 32

## Tornado Load Provisions – MWFRS

| References* | Tornado Load Parameters for MWFRS Loads (ASCE 7 Ch 32 – Directional) |
|---|---|
| ASCE 7 T32.6-1 | Directionality factor, $K_{dT}$ |
| ASCE 7 T 32.14-1 | Pressure coefficient adjustment factor, $K_{vt}$ |
| ASCE 7 §32.15.1 & 27.3.1 | External pressure coefficients, $C_p$ (enclosed, partially enclosed, and partially open buildings of all heights) |
| ASCE 7 §32.15.2 & 27.3.2 | External pressure coefficients, $C_N$ (open buildings with monoslope, pitched, or troughed free roofs) |
| ASCE 7 §32.15.3 & 27.3.3 | Roof overhangs |
| ASCE 7 §32.15.4 & 27.3.4 | Combined net pressure coefficient for parapets, $(GC_{pn})$ |
| ASCE 7 §32.15.5 & 27.3.5 | Controlling wind load Cases: 1, 2, 3, 4 |

*IBC 2024 and ASCE 7-22

$p_T = q_T\, G_T\, K_{dT}\, K_{vT}\, C_p - q_i\,(GC_{piT})$ (32.15-1) All heights

$p_T = q_T\, G_T\, K_{dT}\, C_N$ (32.15-2) Open

$p_{pT} = q_{pt}\, K_{dT}\,(GC_{pn})$ (32.15-3) Parapets

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EXAMPLE

## Tornado Load Supplemental Data Request

**Hospital Building ① N-S Direction** (E-W not shown)

| References* | Wind Load Parameters for MWFRS Loads (ASCE 7 Ch 32 – Directional) | Data |
|---|---|---|
| ASCE 7 T32.6-1 | Directionality factor, $K_{dT}$ | 0.80 |
| ASCE 7 T 32.14-1 | Pressure coefficient adjustment factor, $K_{vt}$ | Roof (-): 1.1<br>Roof (+): 1.0<br>Walls: 1.0 |
| ASCE 7 §32.15.1 & 27.3.1 | Flat, gable, hip, monoslope and mansard roofs – not open (Fig 27.3-1) (enclosed, partially enclosed, and partially open buildings of all heights) | |
| ASCE 7 F27.3-1 | Horizontal building dimension perp to wind, $B$ (ft) | 150 |
| | Horizontal building dimension parallel to wind, $L$ (ft) | 100 |
| | Roof slope, θ (degrees) | 0 |
| | Mean roof height, $h$ (ft) [eave height for $\theta \leq 10°$] | 50 |
| | Height above ground, $z$ (ft) | n/a |
| | Velocity pressure at respective height, $q_z$, $q_h$, or $q_p$ | See table |
| | External pressure coefficients, $C_p$ | F27.3-1 |
| ASCE 7 §32.15.4 & 27.3.4 | Combined net pressure coefficient for parapet, $(GC_{pn})$ | n/a |
| ASCE 7 §32.15.5 & 27.3.5 | Wind load Cases: 1, 2, 3, 4 | See figure |

*IBC 2024 and ASCE 7-22

$p_T = q_T\, G_T\, K_{dT}\, K_{vT}\, C_p - q_i\,(GC_{piT})$ (32.15-1)

Example 5.26 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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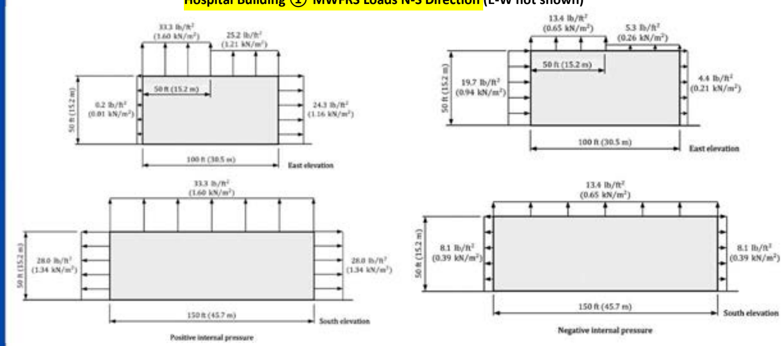
EXAMPLE

## Tornado Load Supplemental Data Request

**Hospital Building ① MWFRS Loads N-S Direction** (E-W not shown)




Example 5.26 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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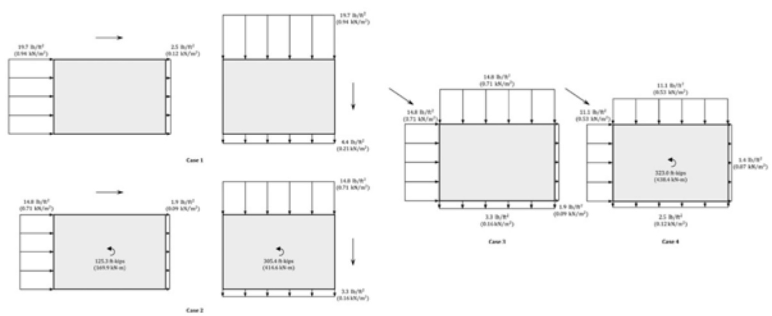
EXAMPLE

## Tornado Load Supplemental Data Request

**Hospital Building ① MWFRS Torsional Loads** at Roof Level



Example 5.26 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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EXAMPLE

## Comparison of Tornado to Wind Loads

**MWFRS Loads N-S Direction with Negative Internal Pressure (E-W not shown)**

| Height (ft) | Windward Wall | | Leeward Wall | |
|---|---|---|---|---|
| | p (psf) | $p_T$ (psf) | p (psf) | $p_T$ (psf) |
| 50 | 21.3 | 19.7 | -6.1 | -4.4 |
| 40 | 20.3 | 19.7 | -6.1 | -4.4 |
| 30 | 19.1 | 19.7 | -6.1 | -4.4 |
| 20 | 17.6 | 19.7 | -6.1 | -4.4 |
| 10 | 16.7 | 19.7 | -6.1 | -4.4 |

Use whichever controls

Use whichever controls

Example 5.26 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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CODE CHANGE

## Tornado Load Provisions – C&C

| References* | Tornado Load Parameters for C&C Loads (ASCE 7 Ch 32 – Directional) |
|---|---|
| ASCE 7 T32.6-1 | Directionality factor, $K_{dT}$ |
| ASCE 7 T 32.14-1 | Pressure coefficient adjustment factor, $K_{vt}$ |
| | Effective wind area, *EWA* |
| | External pressure coefficients and design tornado pressure, $p_T$ |
| ASCE 7 §32.17.1 & 30.3 | Buildings h ≤ 60 ft & low-rise ($GC_p$) |
| ASCE 7 §32.17.2 & 30.4.2 | Buildings h > 60 ft ($GC_p$) |
| ASCE 7 §32.17.3 & 30.5.2 | Open buildings ($C_N$) |
| ASCE 7 §32.17.4.1 & 30.6 | Parapets ($GC_p$) |
| ASCE 7 §32.17.4.2 & 30.7 | Overhangs ($GC_p$) |

*IBC 2024 and ASCE 7-22

$p_T = q_{hT}\,[K_{dT}\,K_{vT}\,(GC_p) - (GC_{piT})]$ (32.17-1) h ≤ 60 ft & low-rise

$p_T = qK_{dT}\,K_{vT}\,(GC_p) - q_i(GC_{piT})$ (32.17-2) h > 60 ft

$p_T = q_{hT}\,G_T\,K_{dT}\,C_N$ (32.17-3) Open buildings

$p_T = q_{pT}\,[K_{dT}\,(GC_p) - (GC_{piT})]$ (32.17-4) Parapets

$p_T = q_{hT}\,[K_{dT}\,K_{vT}\,(GC_p) - (GC_{piT})]$ (32.17-5) Overhangs

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EXAMPLE

## Tornado Load Supplemental Data Request

| References* | Tornado Load Parameters for C&C Loads: h ≤ 60 ft or low-rise | Data | | | | | |
|---|---|---|---|---|---|---|---|
| ASCE 7 T32.6-1 | Directionality factor, $K_{dT}$ | 1.0 | | | | | |
| | | Zone | 1′ (-) | 1 (-) | 2 (-) | 3 (-) | 3 (+) |
| ASCE 7 T 32.14-1 | Pressure coefficient adjustment factor, $K_{vt}$ | | 1.2 | 1.2 | 1.05 | 1.05 | 1.0 |
| ASCE 7 §32.17.1 & 30.3 | External pressure coefficients and design pressures (psf) | Zone | 1′ (-) | 1 (-) | 2 (-) | 3 (-) | |
| ASCE 7 F30.3-1 | Walls: Zones 4 & 5 | $(GC_p)$ | | | | | |
| | | $p_T$ | | | | | |
| ASCE 7 F30.3-1A, §32.17.1.1 & F30.3-2A | Bottom horizontal surface of elevated buildings ($K_{vt}$ = 1.0): Zones 1′, 1, 2, 3 | $(GC_p)$ | | | | | |
| | | $p_T$ | | | | | |
| ASCE 7 F30.3-2A | Gable (flat) Roofs ≤ 7°: Zones 1′, 1, 2, 3 | $(GC_p)$ | 0.20<br>-0.61 | 0.20<br>-1.05 | 0.20<br>-1.47 | 0.20<br>-1.53 | |
| | | $p_T$ | See table | | | | |
| ASCE 7 F30.3-2B to F30.3-2G | Gable or Hip Roofs > 7°: Zones 1, 2, 3 | $(GC_p)$ | | | | | |
| | | $p_T$ | | | | | |

*IBC 2024 and ASCE 7-22

$p_T = q_{hT}\,[K_{dT}\,K_{vT}\,(GC_p) - (GC_{piT})]$ (32.17-1)

Example 5.27 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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EXAMPLE

## Comparison of Tornado to Wind Loads

**C&C Loads on the Roof**

| Zone | Positive | | Negative | |
|---|---|---|---|---|
| | p (psf) | $p_T$ (psf) | p (psf) | $p_T$ (psf) |
| 1' | 9.4 | 10.3 | -19.6 | -34.9 |
| 1 | 9.4 | 10.3 | -30.5 | -49.2 |
| 2 | 9.4 | 10.3 | -40.7 | -56.9 |
| 3 | 9.4 | 10.3 | -42.4 | -58.7 |

Use whichever controls

Use whichever controls

Example 5.27 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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EXAMPLE

## Comparison of Tornado to Wind Loads

**Controlling Load on Hospital (N-S Direction)**

| Height (ft) | Windward Wall | Leeward Wall | Roof |
|---|---|---|---|
| 50 | Wind | Wind | |
| 40 | Wind | Wind | |
| 30 | Tornado | Wind | |
| 20 | Tornado | Wind | |
| 10 | Tornado | Wind | |
| Roof | - | - | Tornado |

Example 5.26 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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## ASCE 7 Tornado Provisions Summary

1. Apply to Risk Category III and IV structures
2. Apply in tornado-prone regions east of the continental divide
3. MWFRS and C&C loads are calculated for both wind and tornado and governing loads apply
4. Does not constitute storm shelter loads
5. Effective plan area encloses both the Essential Facility and all other structures that maintain its functionality

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## Resources

Available for use or that can be used for support or help

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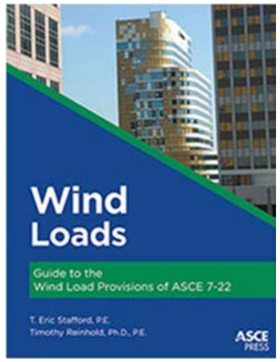
## Wind Loads Guide

- 19 worked examples
- ASCE 7-22 wind load revisions covered
  - Simplifications to external C&C roof pressure coefficients for steep slope roofs
  - Design wind speed maps that include terrain speedup effects
  - Changes to Wind-borne Debris Region definition
  - Deletion of Simplified Procedures
- ASCE 7-22 wind load revisions not covered
  - Ground-mounted solar arrays
  - Elevated buildings
    - Example in ASCE 7-22 commentary
  - New tornado loads
    - Separate ASCE guide

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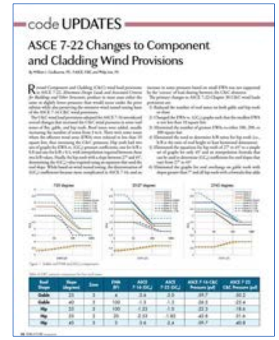
## *STRUCTURE* Magazine Articles


code UPDATES

ASCE 7-22 Changes to Component and Cladding Wind Provisions

structuremag.org – Oct 2022

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## COURSE OUTLINE

- Earthquake Provisions
  - Seismic Design Category
  - Load Combinations
  - Definitions and Symbols
  - Seismic Force-resisting Systems
  - Design Provisions
  - Examples
- Resources

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## Earthquake Loads

Updates, deletions and revisions to various provisions

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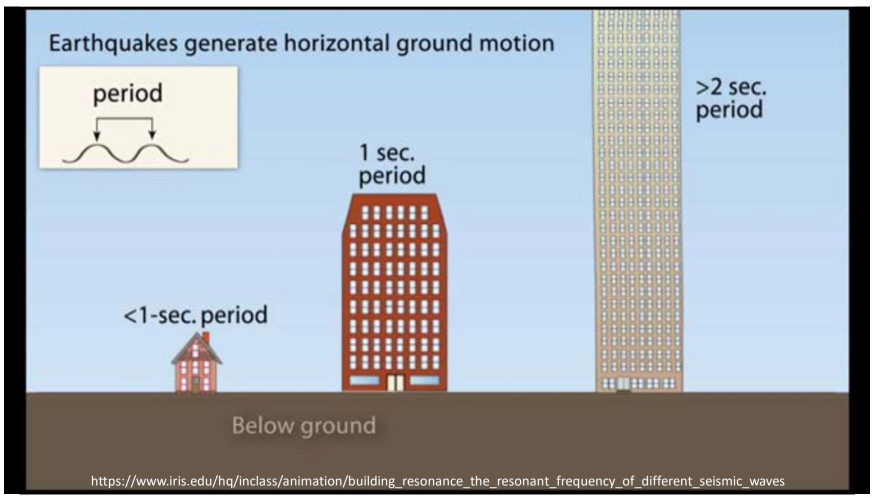


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## CODE CHANGE

### Earthquake Loads – Scope

Every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with Chapters 11, 12, 13, 15, 17 and 18 of ASCE 7, as applicable. The *seismic design category* for a structure is permitted to be determined in accordance with Section 1613 or ASCE 7.

**Exceptions:**

1. Detached one- and two-family dwellings, assigned to *Seismic Design Category* A, B or C. ~~, or located where the mapped short-period spectral response acceleration,~~ ~~$S_S$, is less than 0.4 g.~~

(no changes to exceptions 2-5)

6. *Temporary structures* complying with Section 3103.6.1.4.

- Application based on SDC alone not $S_S$
- New exception for temporary structures

§ 1613.1

24 IBC

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CODE CHANGE

§ 1613.2

## Determination of Seismic Design Category

~~Seismic ground motion values~~ **Determination of Seismic Design Category**. ~~Seismic ground motion values shall be determined in accordance with this section.~~ Structures shall be assigned to a Seismic Design Category based on one of the following methods unless the authority having jurisdiction or geotechnical data determines that Site Class DE, E or F soils are present at the site.

1. Based on the structure risk category using Figures 1613.2(1) through 1613.2(7).
2. Determined in accordance with ASCE 7.

Where Site Class DE, E or F soils are present, the Seismic Design Category shall be determined in accordance with ASCE 7.

(Sections 1613.2.1 through 1613.2.5 deleted without substitution)

- Site classes A, B, BC, C, CD, D
- SDC determined by
  - 1 of 7 IBC maps
  - ASCE 7 maps
  - asce7hazardtool.online
- All other site classes per ASCE 7

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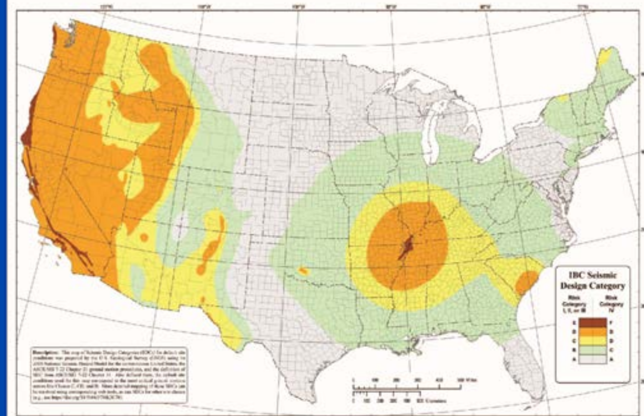
## Seismic Design Categories

Figs 1613.2.1(1-7)




Figures 1613.2.1(1) & (2) Western and Eastern U.S.

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EXAMPLE

## asce7hazardtool.online


Generate PDF...

17

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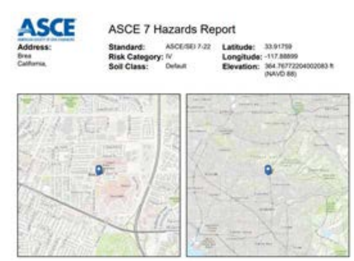
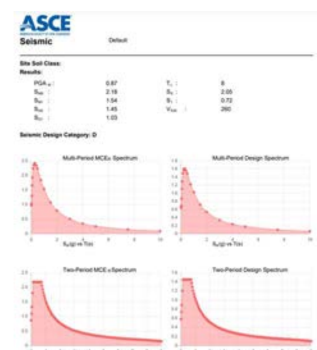
EXAMPLE

## asce7hazardtool.online




Full Report PDF

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§ 1613.4

## Ballasted Photovoltaic Panel Systems

Ballasted, roof-mounted photovoltaic panel systems need not be rigidly attached to the roof or supporting structure. ~~Ballasted non-penetrating systems~~ Ballasted, unattached PV panel systems shall be designed and installed only on roofs with slopes not more than one unit vertical in 12 units horizontal. ~~Ballasted nonpenetrating systems~~ Ballasted, unattached PV panel systems shall be designed to ~~resist~~ accommodate sliding ~~and uplift~~ in accordance with ASCE 7 Chapter 13. ~~resulting from lateral and vertical forces as required by Section 1605, using a coefficient of friction determined by acceptable engineering principles. In structures assigned to Seismic Design Category C, D, E or F, ballasted nonpenetrating systems shall be designed to accommodate seismic displacement determined by nonlinear response-history or other approved analysis or shake-table testing, using input motions consistent with ASCE 7 lateral and vertical seismic forces for nonstructural components on roofs.~~

- Ballasted, unattached PV systems
  - Roofs ≤ 1:12
  - Designed to accommodate sliding per ASCE 7
  - Simplified provisions

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CODE CHANGE

§ 1613.5

## Elevators, Escalators, and Other Conveying Systems

Elevators, escalators, and other conveying systems and their components shall satisfy the seismic requirements of ASCE 7 and ASME A17.1/CSA B44 as applicable.

- Similar provisions for flood and wind loads
- ASME A17.1-2022/CSA B44-22: *Safety Code for Elevators and Escalators*

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CODE CHANGE

§ 1613.6

## Automatic Sprinkler Systems

Where required, automatic sprinkler systems, including anchorage and bracing, shall comply with ASCE 7 and Section 903.3.1.1.

- Nonstructural components
- Consistent with ASCE 7-22
  - Clearances for sprinkler drops and sprigs
- Consistent with NFPA 13
  - *Standard for the Installation of Sprinkler Systems*

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CODE CHANGE

§ 1605.1

## Load Combinations

**General.** Buildings and *other structures* and portions thereof shall be designed to resist the strength load combinations specified in ASCE 7, Section 2.3, the *allowable stress design* load combinations specified in ASCE 7, Section 2.4, or the alternative *allowable stress design* load combinations of Section 1605.2.

**Exceptions:**

1. The modifications to load combinations of ASCE 7 Section 2.3, ASCE 7 Section 2.4, and Section 1605.2 specified in ASCE 7 Chapters 18 and 19 shall apply.
2. Where the allowable stress design load combinations of ASCE 7 Section 2.4 are used, flat roof snow loads of ~~30~~ 45 pounds per square foot and roof live loads of 30 pounds per square foot or less need not be combined with seismic load. Where flat roof snow loads exceed ~~30~~ 45 pounds per square foot, ~~20~~ 15 percent shall be combined with seismic loads.

- Strength Design per ASCE 7 Section 2.3
- ASD per ASCE 7 Section 2.4
- Alternative ASD per 1605.2
- ASD snow load values adjusted for new risk-based approach

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§ 2.3.6

## Strength Design Seismic Load Combinations

**Strength Design**

$1.2D + E_v + E_h + L + \cancel{0.2}\underline{0.15}S$ (Eq 6)

$0.9D - E_v + Eh$ (Eq 7)

**Overstrength**

6. $1.2D + E_v + E_{mh} + L + \cancel{0.2}\underline{0.15}S$ (Eq 6)

7. $0.9D - E_v + E_{mh}$ (Eq 7)

- The most unfavorable effects from seismic loads shall be investigated where appropriate, but they need not be considered to act simultaneously with wind or tornado loads.

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CODE CHANGE

§ 2.4.1 & 2.4.5

## ASD Seismic Load Combinations

**Allowable Stress Design (ASD)**

$1.0D + 0.7E_v + 0.7E_h$ (Eq 8)

$1.0D + 0.525E_v + 0.525E_h + 0.75L + \cancel{0.75}\underline{0.1}S$ (Eq 9)

$0.6D - 0.7E_v + 0.7Eh$ (Eq 10)

**Overstrength**

$1.0D + 0.7E_v + 0.7E_{mh}$ (Eq 8)

$1.0D + 0.525E_v + 0.525E_{mh} + 0.75L + \cancel{0.75}\underline{0.1}S$ (Eq 9)

$0.6D - 0.7E_v + 0.7E_{mh}$ (Eq 10)

- …Seismic load effects shall be combined with other loads, in accordance with Section 2.4.5. ~~Wind and seismic loads~~ The most unfavorable effects from wind loads, tornado loads, and earthquake loads shall be considered, where appropriate, but they need not be ~~considered~~ assumed to act simultaneously…When a structure is subject to seismic load effects, the following load combinations shall be considered in addition to the basic combinations and associated exceptions detailed in Section 2.4.1.

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CODE CHANGE

§ 1605.2

## Load Combinations – Alternative ASD

D + L + ($L_r$ or 0.7S or R) (Eq 16-1)

D + L + 0.6W (Eq 16-2)

D + L + 0.6W + 0.7S/2 (Eq 16-3)

D + L + 0.7S + 0.6W/2 (Eq 16-4)

D + L + 0.7S + E/1.4 (Eq 16-5)

0.9D + E/1.4 (Eq 16-6)

**Exception 2.** Flat roof snow *loads* of ~~30~~ 45 pounds per square foot or less and *roof live loads* of 30 pounds per square foot or less need not be combined with seismic loads. Where flat roof snow loads exceed ~~30~~ 45 pounds per square foot, ~~20~~ 15 percent shall be combined with seismic loads.

- Allowable stresses can be increased, or load combinations reduced if permitted by the material chapter or referenced standards
- For proportioning foundations for seismic loads, the vertical seismic load effect, $E_v$, in Equation 12.4-4 of ASCE 7 is permitted to equal zero

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## Summary of ASCE 7-22 Seismic Procedures

**Table 6.1 Chapters in ASCE/SEI 7 Referenced by the IBC for Seismic Load Provisions**

| Chapter | Title |
|---|---|
| 11 | Seismic Design Criteria |
| 12 | Seismic Design Requirements for Building Structures |
| 13 | Seismic Design Requirements for Nonstructural Components |
| 15 | Seismic Design Requirements for Nonbuilding Structures |
| 20 | Site Classification Procedure for Seismic Design |
| 21 | Site-Specific Ground Motion Procedures for Seismic Design |
| 22 | Seismic Ground Motion and Long-Period Transition Maps |

*Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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CODE CHANGE

Ch 11

## Summary of ASCE 7-22 Major Seismic Changes

- Chapter 11 – Seismic Design Criteria
  - Definitions
  - Symbols
  - Site Class
  - $MCE_R$ Parameters
  - Design Response Spectrum
  - Vertical Ground Motion

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CODE CHANGE

§ 11.4.2

## Site Class

~~Based on the site soil properties,~~ the site shall be classified as Site Class A, B, BC, C, CD, D, DE, E, or F in accordance with Chapter 20.

**11.4.2.1 Default Site Conditions** Where the soil properties are not known in sufficient detail to determine the site class, ~~Site Class D, subject to the requirements of Section 11.4.4, shall be used~~ risk-targeted maximum considered earthquake ($MCE_R$) spectral response accelerations shall be based on the most critical spectral response acceleration at each period of Site Class C, Site Class CD, and Site Class D, unless the Authority Having Jurisdiction or ~~geotechnical data~~ determines, based on geotechnical data, that Site Class DE, E, or F soils are present at the site.

~~For situations in which site investigations, performed in accordance with Chapter 20, reveal rock conditions consistent with Site Class B, but site-specific velocity measurements are not made, the site coefficients $F_a$, $F_v$, and $F_{PGA}$ shall be taken as unity (1.0).~~

- Three new soil site classes: BC, CD, DE
- C, CD and D are default

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CODE CHANGE

T 20.2-1

## Site Classification

| Site Class | $v_s$ (ft/s) |
|---|---|
| A. Hard rock | >5,000 |
| B. Medium hard rock | ~~2,500~~>3,000 to 5,000 |
| BC. Soft rock | >2,100 to 3,000 |
| C. Very dense ~~soil and soft rock~~ sand or hard clay | ~~1,200~~>1,450 to ~~2,500~~2,100 |
| CD. Dense sand or very stiff clay | >1,000 to 1,450 |
| D. ~~Stiff soil~~ Medium dense sand or stiff clay | ~~600~~>700 to ~~1,200~~1,000 |
| DE. Loose sand or medium stiff clay | >500 to 700 |
| E. ~~Soft clay soil~~ Very loose sand or soft clay | ~~<600~~≥500 |
| F. Soils requiring a site response analysis in accordance with ASCE/ SEI 21.1 | See Section 20.2.1 |

$\left.\right\}$ Based on shear wave velocity tests only

$\left.\right\}$ Use if soil properties unknown

Average shear wave velocity parameter, $v_s$, is derived from the measured shear wave velocity profile from the ground surface to a depth of 100 ft.

Adapted from ASCE 7-22 Table 20.2-1

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§ 11.4.3

## $MCE_R$ Parameters

Risk-targeted maximum considered earthquake ($MCE_R$) spectral response acceleration parameters $S_S$, $S_1$, $S_{MS}$, and $S_{M1}$ shall be obtained from the USGS Seismic Design Geodatabase for the applicable site class.

**EXCEPTION:** Where a site-specific ground motion analysis is performed in accordance with Section 11.4.7, risk-targeted maximum considered earthquake ($MCE_R$) spectral response acceleration parameters $S_{MS}$ and $S_{M1}$ shall be determined in accordance with Section 21.4 and risk-targeted maximum considered earthquake ($MCE_R$) spectral response acceleration parameters $S_S$ and $S_1$ shall be either:

(1) determined from the site-specific $MCE_R$ response spectrum calculated in accordance with the requirements of Section 21.2.3 assuming Site Class BC site condition or

(2) obtained from the USGS Seismic Design Geodatabase.

- Seismic parameters $S_{MS}$ and $S_{M1}$ (and $S_{DS}$ and $S_{D1}$) incorporate site effects
- Online tools provide all parameters
- Site factor tables for $F_a$ and $F_v$ eliminated

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§ 11.4.5

## Design Response Spectrum

Where a design response spectrum is required by this standard ~~and site-specific ground motion procedures are not used~~, the design response spectrum ~~curve~~ shall be ~~developed as indicated in Fig. 11.4-1 and as follows:~~ determined in accordance with the requirements of Section 11.4.5.1.

**EXCEPTIONS:**

**1.** Where a site-specific ground motion analysis is performed in accordance with Section 11.4.7, the design response spectrum shall be determined in accordance with Section 21.3.

**2.** Where values of the multi-period 5%-damped $MCE_R$ response spectrum are not available from the USGS Seismic Design Geodatabase, the design response spectrum shall be permitted to be determined in accordance with Section 11.4.5.2.

- 11.4.5.1 Multi-period design response spectrum – more accurate
- 11.4.5.2 Two-period design response spectrum – permitted

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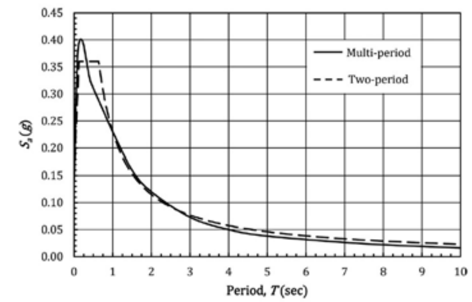
EXAMPLE

§ 11.4.5

## Design Response Spectrum

- Nashville, TN
- Risk Category II
- Site Class D




Example 6.1 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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ASCE 7 Seismic Design Criteria Change Summary

1. Simplified maps for determining SDC
2. Three new soil site classes: BC, CD, DE
3. Seismic parameters $S_{MS}$, $S_{M1}$, $S_{DS}$, and $S_{D1}$ incorporate site effects eliminating need for site factors $F_a$ and $F_v$
4. Multi-period design response spectrum is preferred as more accurate over the 2-period approach

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APPLICATION

**4. Seismic Design Categories can be determined by which of the following?**

a) IBC maps
b) ASCE 7 maps
c) asce7hazardtool.online
d) All of the above

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Ch 12

## Summary of ASCE 7-22 Major Seismic Changes

- Chapter 12 – Seismic Design Requirements for Buildings
  - Seismic Force-Resisting Systems – three new systems
  - Diaphragm in Hillside Light-Frame Structures
  - Torsionally Irregular Structures
  - Redundancy Factor
  - Direction of Loading – SDC D through F
  - Modeling Criteria – Effective Seismic Weight
  - Equivalent Lateral Force (ELF) Procedure
  - Diaphragms, Chords, and Collectors
  - Alternative Diaphragm Design Provisions – RWFD

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§ 12.2.1

## Seismic Force-Resisting Systems

Table 12.2-1 (excerpt). Design Coefficients and Factors for Seismic Force-Resisting Systems (SFRS)

| SFRS | ASCE 7 Detailing | R | $\Omega_0$ | $C_d$ | Height Limits (ft) SDC | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | B | C | D | E | F |
| A. BEARING WALL SYSTEMS | | | | | | | | | |
| 2. Reinforced concrete ductile coupled walls[q] | 14.2 | 8 | 2½ | 8 | NL | NL | 160 | 160 | 100 |
| 20. Cross-laminated timber shear walls | 14.5 | 3 | 3 | 3 | 65 | 65 | 65 | 65 | 65 |
| 21. Cross-laminated timber shear walls with shear resistance provided by high-aspect-ratio panels only | 14.5 | 4 | 3 | 4 | 65 | 65 | 65 | 65 | 65 |
| B. BUILDING FRAME SYSTEMS | | | | | | | | | |
| 5. Reinforced concrete ductile coupled walls[q] | 14.2 | 8 | 2½ | 8 | NL | NL | 160 | 160 | 100 |
| 28. Steel and concrete coupled composite plate shear walls | 14.3 | 8 | 2½ | 5½ | NL | NL | 160 | 160 | 100 |
| D. DUAL SYSTEMS WITH SPECIAL MOMENT FRAMES | | | | | | | | | |
| 15. Steel and concrete coupled composite plate shear walls | 14.3 | 8 | 2½ | 5½ | NL | NL | NL | NL | NL |

q. Structural height, $h_n$, shall not be less than 60 ft.

- Three new SFRSs
  - Concrete ductile coupled walls
  - Cross-Laminated Timber (CLT) shear walls
  - Steel and concrete coupled composite plate shear walls (SpeedCore)

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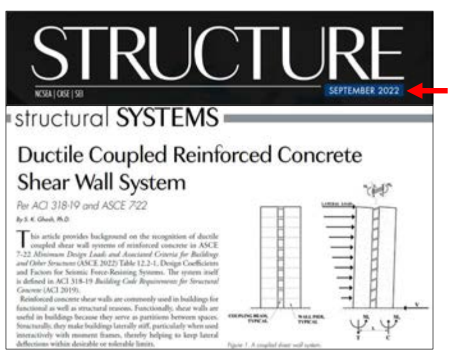
CODE CHANGE

§ 12.2.1

## Concrete Ductile Coupled Walls

- ACI 318-19 new system



STRUCTURE

SEPTEMBER 2022

structural SYSTEMS

Ductile Coupled Reinforced Concrete Shear Wall System

Per ACI 318-19 and ASCE 7-22

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CODE CHANGE

§ 12.2.1

## CLT Shear Walls

2021 SDPWS CLT Shear Wall Capacities* (ASD)

| Panel Length (ft) | Panel Height (ft) | No. of Angles* | Seismic Shear (plf) | Seismic Hold-down (lbs) | Wind Shear (plf) | Wind Hold-down (lbs) |
|---|---|---|---|---|---|---|
| 4 | 8 | 2 | 465 | 7443 | 651 | 7815 |
| 4 | 8 | 3 | 698 | 11164 | 977 | 11723 |
| 4 | 8 | 4 | 930 | 14886 | 1303 | 15630 |
| 4 | 8 | 5 | 1163 | 18607 | 1628 | 19538 |

*Capacity derived from 2021 SDPWS based on angles along bottom of one panel face and G = 0.42.


§ 4.6 & App. B

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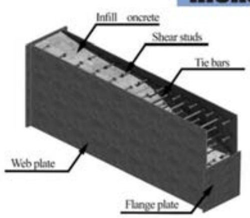
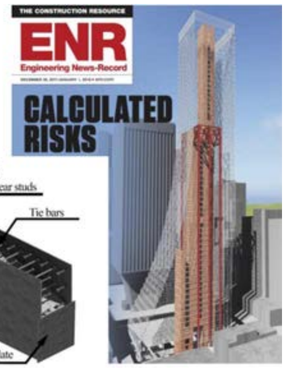
§ 12.2.1

## Steel & Concrete Coupled Composite Plate Shear Walls

- AISC *Design Guide 38 – SpeedCore Systems for Steel Structures*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§ 12.2.5.4

## Increased Structural Height

**Increased Structural Height Limit for Steel Eccentrically Braced Frames, Steel Special Concentrically Braced Frames, Steel Buckling-Restrained Braced Frames, Steel Special Plate Shear Walls, Steel and Concrete Coupled Composite Plate Shear Walls, Reinforced Concrete Ductile Coupled Walls, and Special Reinforced Concrete Shear Walls**

The limits on structural height, $h_n$, in Table 12.2-1 are permitted to be increased from 160 ft to 240 ft for structures assigned to Seismic Design ~~Categories~~ Category D or E and from 100 ft to 160 ft for structures assigned to Seismic Design Category F, provided that the seismic force-resisting systems are limited to steel eccentrically braced frames, steel special concentrically braced frames, steel buckling-restrained braced frames, steel special plate shear walls, steel and concrete coupled composite plate shear walls, reinforced concrete ductile coupled walls, or special reinforced concrete cast-in-place shear walls and both of the following requirements are met:

**1.** The structure shall not have ~~an extreme torsional~~ a Type 1 horizontal irregularity ~~as defined in Table 12.3-1 (horizontal structural irregularity Type 1b).~~ with a TIR > 1.4, and

**2.** The steel eccentrically braced frames, steel special concentrically braced frames, steel buckling-restrained braced frames, steel special plate shear walls, steel and concrete coupled composite plate shear walls, reinforced concrete ductile coupled walls, or special reinforced cast-in-place concrete shear walls in any one plane shall resist no more than 60% of the total seismic forces in each direction, neglecting accidental torsional effects.

- SDC D or E: 160 ft → 240 ft
- SDC F: 100 ft → 160 ft
- New SFRSs: RC ductile coupled walls and SpeedCore systems
- New Item 2 similar to redundancy requirements

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CODE CHANGE

§ 12.3.1.4

## Diaphragms in Hillside Light-Frame Structures

Diaphragms in hillside structures of light-frame construction shall be modeled as rigid or semirigid when they are seismically braced on one or more sides directly by the foundation or foundation stem wall or by a framed wall with a clear height of 2 ft or less, and are also seismically braced on other sides by framed walls with any wall having a clear height of more than 7 ft. Other diaphragms shall be idealized as flexible or rigid in accordance with Sections 12.3.1.1, 12.3.1.2, or 12.3.1.3, or shall be modeled as semirigid.


ASCE 7-22 Figure C12.3-1a

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## Irregular Buildings

Seismic Design Category (SDC) "triggers" consideration of structural irregularities

| | | Structural Irregularity Type | | | | |
|---|---|---|---|---|---|---|
| **SDC** | | **1** | **2** | **3** | **4** | **5** |
| **B or C** | **Horizontal** | x | | | x | x |
| | **Vertical** | | | x | x | n/a |
| **D, E or F** | **Horizontal** | x | x | x | x | x |
| | **Vertical** | x | x | x | x | n/a |

ASCE 7 § 12.3.2

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§ 12.3.2.1.1

## Torsional Irregularity Ratio

A Torsional Irregularity Ratio (*TIR*) shall be calculated for each story and for each accidental torsion case:

$$TIR = \Delta_{max} / \Delta_{avg} \qquad (12.3\text{-}2)$$

where $\Delta_{max}$ is the maximum story drift at the building's edge subjected to lateral forces using the equivalent lateral force procedure per Section 12.8 with the application of accidental torsion per Section 12.8.4 and $A_x = 1.0$; and $\Delta_{avg}$ is the average of the story drifts at the two opposing edges of the building determined using the same loading and diaphragm rigidity as applied for the determination of $\Delta_{max}$.

For the purpose of computing $\Delta_{max}$ and $\Delta_{avg}$, it shall be permitted to assume the diaphragm is rigid for structures with rigid or semirigid diaphragms. The *TIR* for the building is the maximum value from the values computed for each story and each direction. The *TIR* shall not apply to structures with flexible diaphragms.

- Likelihood of excessive torsional response used to trigger design penalties where TIR > 1.2
- Not applicable to flexible diaphragms

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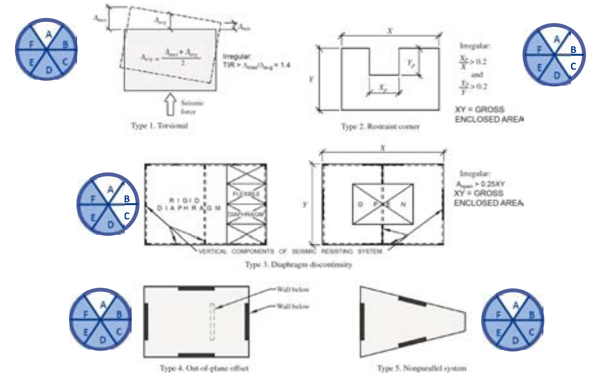
§ 12.3.2.1

## Horizontal Irregularities




Adapted from ASCE 7-22 Figure C12.3-1b

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CODE CHANGE

§ 12.3.2.1

## Horizontal Irregularities

**Table 12.3-1. Horizontal Structural Irregularities**

| Type | Description | Ref. | SDC |
|---|---|---|---|
| 1. | **Torsional Irregularity:** Torsional irregularity, defined to exist where either:<br>• More than 75% of any story's lateral strength below the diaphragm is provided at or on one side of the center of mass, or<br>• The Torsional Irregularity Ratio (TIR) exceeds 1.2.<br>The story lateral strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration. | Table 12.3-1a, 12.3.3.5<br>12.5.1.2<br>12.7.3<br>12.8.4.3<br>12.8.6<br>16.3.4 | B, C, D, E, F<br>C, D, E, F<br>B, C, D, E, F<br>C, D, E, F<br>C, D, E, F<br>B, C, D, E, F |
| 2. | **Reentrant Corner Irregularity:** Reentrant corner irregularity, defined to exist where both plan projections of the structure beyond a reentrant corner are greater than ~~15%~~ 20% of the plan dimension of the structure in the given direction. | 12.3.3.5 | D, E, F |
| 3. | **Diaphragm Discontinuity Irregularity:** Diaphragm discontinuity irregularity, defined to exist where there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one that has a cutout or open area greater than ~~50%~~ 25% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next. | 12.3.3.5 | D, E, F |

- Not all strikethrough text shown
- Items 4 and 5 are unchanged and not shown

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CODE CHANGE

§ 12.3.2.1

## Horizontal Irregularity Type 1

**Table 12.3-1a. Reference Sections for Type 1 Horizontal Torsional Irregularity (TIR)**

| TIR | SDC | Ref. | Description [a] |
|---|---|---|---|
| >1.2 [b] | B, C, D, E, F | 15.4.1 | Inclusion of accidental torsion in nonbuilding structures not similar to buildings |
| | | 16.3.4 | Mass offset in Nonlinear Response History analysis |
| | | 17.2.2 | Irregularities in Seismically Isolated Buildings |
| | | 18.2.3.2 | Applicability of Equivalent Lateral Force Procedure for structures with damping systems |
| | C, D, E, F | 12.7.3 | Use of 3D analytical model |
| | D, E, F | 12.5.4 | Use of orthogonal load combinations |
| | | 12.8.4.2 and 12.8.4.3 | Inclusion of amplified accidental torsion |
| | | 12.8.6 | Story drift for purpose of comparing to drift limits, computing P-Delta, and checking deformation compatibility must be computed at the edge with the largest drift. |
| | D, E, F | 12.3.3.4 | Amplification of design force for collectors, their connections, and connections of diaphragms to collectors or vertical elements of the seismic force-resisting system (SFRS) |
| >1.4 | B, C, D, E, F | 18.2.1.1 | Minimum seismic base shear for structures with damping systems |
| | D, E, F | 12.2.5.4 | Increased structural height limits not allowed for certain systems |
| | D, E, F | 12.5.4 | Use of orthogonal load combinations |
| | | 12.8.4.2 | Include accidental torsion |
| >1.4 both directions | D, E, F | 12.3.4.2.1 | Criteria for setting redundancy factor = 1.3 |
| >1.6 | D, E, F | 12.9.1.5 | Restrictions on use mass offset for accidental torsion in Modal Response Spectrum analysis |

a. Refer to the reference section for the precise requirement.
b. The requirements for TIR > 1.2 also apply if TIR ≤ 1.2 and if more than 75% of the story's lateral strength below the diaphragm is provided at or on one side of the center of mass per Table 12.3-1.
Notes: The requirements in this table are cumulative; thus for TIR > 1.4 or 1.6, all the requirements for TIR > 1.2 also apply.

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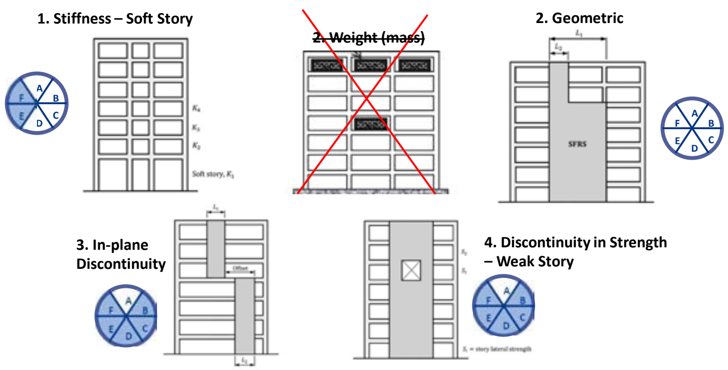
## Vertical Irregularities




§ 12.3.2.2

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Figures 6.20 to 6.23 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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## Vertical Irregularities

CODE CHANGE

**Table 12.3-2. Vertical Structural Irregularities**

| Type | Description | Ref. | SDC |
|---|---|---|---|
| 1a | **Stiffness–Soft Story Irregularity** is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or, where there are at least three stories above, less than 80% of the average stiffness of the three stories above. | ~~Table 12.6-1~~ | ~~D, E, F~~ |
| 1b | **Stiffness–Extreme Soft Story Irregularity** is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or, where there are at least three stories above, less than 70% of the average stiffness of the three stories above. | 12.3.3.1<br>~~Table 12.6-1~~ | E, F<br>~~D, E, F~~ |
| ~~2~~ | ~~Weight (Mass) Irregularity: Weight (mass) irregularity is defined to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.~~ | ~~Table 12.6-1~~ | ~~D, E, F~~ |
| 2 | **Vertical Geometric Irregularity** is defined to exist where the horizontal dimension of the seismic force-resisting system in any story is more than 130% of that in an adjacent story. | ~~Table 12.6-1~~ | ~~D, E, F~~ |
| 3 | **In-Plane Discontinuity** in Vertical Lateral Force-Resisting Element Irregularity is defined to exist where there is an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on supporting structural elements. | ~~12.3.3.3~~12.3.3.4<br>~~12.3.3.4~~12.3.3.5<br>~~Table 12.6-1~~ | B, C, D, E, F<br>D, E, F<br>~~D, E, F~~ |
| 4a | **Discontinuity in Lateral Strength–Weak Story Irregularity** is defined to exist where the story lateral strength is less than ~~80% of~~ that in the story above. The story lateral strength is the total lateral strength of all seismic force-resisting system elements ~~sharing~~ resisting the story shear for the direction under consideration. | 12.3.3.1<br>~~Table 12.6-1~~ | E, F<br>~~D, E, F~~ |
| 4b | **Discontinuity in Lateral Strength–Extreme Weak Story Irregularity** is defined to exist where the story lateral strength is less than 65% of that in the story above. The story lateral strength is the total lateral strength of all seismic force-resisting system elements ~~sharing~~ resisting the story shear for the direction under consideration. | 12.3.3.1<br>12.3.3.2<br>12.3.3.3<br>~~Table 12.6-1~~ | ~~D,~~ E, F<br>~~B, C~~ D<br>B, C<br>~~D, E, F~~ |

* See Section 12.8.1.3 for requirement for any structure with an irregularity listed in this table.

- Type 2 deleted

§ 12.3.2.2

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CODE CHANGE

### Redundancy Factor, $\rho$ = 1.0

The value of $\rho$ is permitted to equal 1.0 for the following:

**7.** Diaphragm ~~loads~~ seismic design forces determined using Equation (12.10-1), including the limits imposed by Equations (12.10-2) and (12.10-3);

**8.** Diaphragm seismic design forces determined in accordance with Section 12.10.4;

- Higher degree of redundancy exists where multiple paths resist lateral forces
- Reduces R for less redundant structures → increases seismic demand
- Items 1-6 unchanged
- Items 9 and 10 renumbered
- Item 8 refers to new provisions for rigid wall flexible diaphragm systems

§ 12.3.4.1

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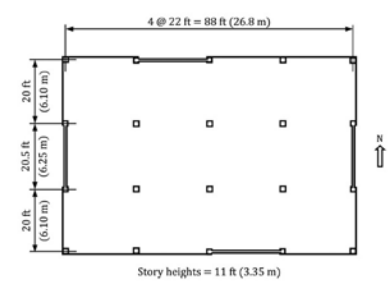
EXAMPLE

## Redundancy Factor, $\rho$

- 10-story residential building
- No irregularities
- SDC D

- §12.3.4.2(1)
- N-S: $h_w / \ell_w$ = 11.0/20.5 = 0.54
- E-W: $h_w / \ell_w$ = 11.0/22.0 = 0.50
- $\rho$ = 1.0 both directions

- §12.3.4.2(2)
- E-W: #bays = 22.0/11.0 = 2.0
  - $\rho$ = 1.0
- N-S: #bays = 20.5/11.0 = 1.9 < 2.0
  - $\rho$ = 1.3




Example 6.9 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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§ 12.8.1.1

## Equivalent Lateral Force (ELF) Procedure

**Calculation of Seismic Response Coefficient** Where the design spectral acceleration parameter $S_a$ determined in accordance with either Section 11.4.5.1 or Chapter 21 is available, either Method 1 or Method 2 is permitted to determine the seismic response coefficient, $C_s$. Where Exception 2 of Section 11.4.5 applies, Method 1 shall not be used. The lower bound for the seismic response coefficient, $C_s$, provided in Item 3 shall be applicable for both Method 1 and Method 2.

**Method 1:** The seismic response coefficient, $C_s$, shall be determined in accordance with Equation (12.8-2).

$$C_s = S_a / (R/I_e) \qquad \textbf{(12.8-2)}$$

where

$\cancel{S_{DS}}$ $S_a$ = Design spectral response acceleration parameter defined in Section 11.4.5.1 and determined for the period $T$ defined in Section 12.8.2;

$R$ = Response modification factor in Table 12.2-1; and

$I_e$ = Importance Factor determined in accordance with Section 11.5.1.

Where Equation (12.8-2) is used and the period $T$ is less than the period at which $S_a$ is maximum, the maximum value of $S_a$ shall be used.

- Method 1: new and based on $S_a$
- Method 2: unchanged and based on $S_{DS}$ and $S_1$

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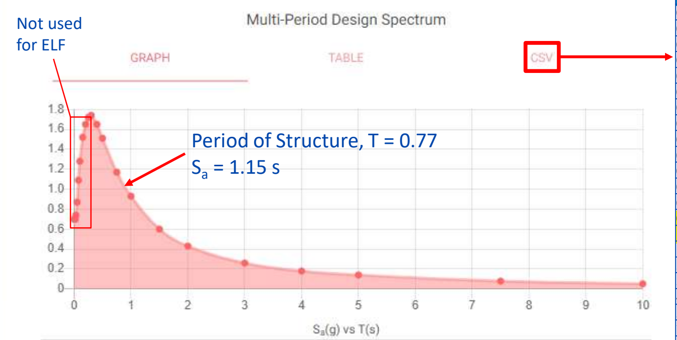
EXAMPLE

§ 12.8.1.1

## ELF – Method 1




| T(s) | $S_a$(g) |
|---|---|
| 0 | 0.7 |
| 0.01 | 0.7 |
| 0.02 | 0.7 |
| 0.03 | 0.74 |
| 0.05 | 0.87 |
| 0.075 | 1.09 |
| 0.1 | 1.28 |
| 0.15 | 1.52 |
| 0.2 | 1.65 |
| 0.25 | 1.72 |
| 0.3 | 1.74 |
| 0.4 | 1.65 |
| 0.5 | 1.51 |
| 0.75 | 1.17 |
| 1 | 0.93 |
| 1.5 | 0.6 |
| 2 | 0.43 |
| 3 | 0.26 |
| 4 | 0.18 |
| 5 | 0.14 |
| 7.5 | 0.079 |
| 10 | 0.052 |

Example 6.10 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill & asce7hazardtool.online

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§ 12.8.1.1

## ELF – Method 2

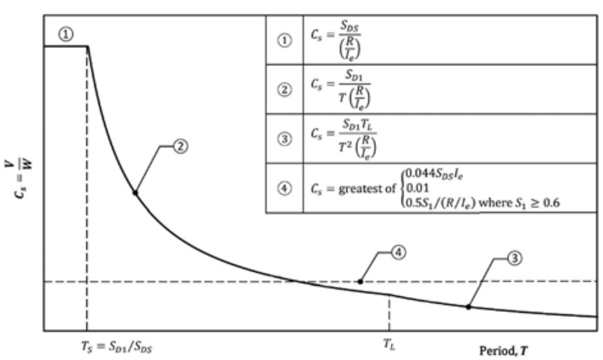


Figure 6.29 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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CODE CHANGE

§ 12.8.1.3

## ELF Procedure – Max $S_{DS}$ in Determination of $C_s$ and $E_v$

The values of $C_s$ and $E_v$ are permitted to be calculated using a value of $S_{DS}$ equal to 1.0, but not less than 70% of the value of $S_{DS}$, as defined in Section 11.4.5, provided that all of the following criteria are met:

*[Items 1-6 unchanged]*

7. The response modification coefficient, $R$, as defined in Table 12.2-1, is 3 or greater.

- No irregularities
- $\leq$ 5 stories
- T $\leq$ 0.5 s
- ρ = 1.0
- Not site class E or F
- Risk Category I or II
- R $\geq$ 3 New

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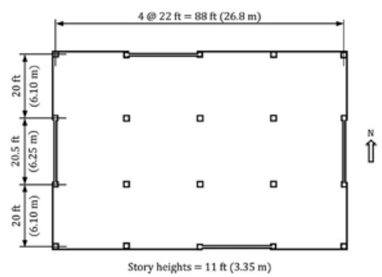
EXAMPLE

## Equivalent Lateral Force (ELF) Procedure

- 10-story residential <300 OL
- Lat: 34.045° Long: −118.258° (Los Angeles)
- Site class CD
- CIP reinforced concrete
- SFRS: Building frame with special reinforced concrete shear walls (system B4)

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Example 6.10 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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EXAMPLE

## Equivalent Lateral Force (ELF) Procedure


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EXAMPLE

## Seismic Load Supplemental Data Request

| References* | Equivalent Lateral Force (ELF) Procedure (ASCE 7 Ch 12) | Data | |
|---|---|---|---|
| IBC §1604.5 | Building Risk Category: I, II, III, IV | II | |
| IBC §1613.2 | Seismic Design Category [maps or asce7hazardtool.online]: | D | |
| ASCE 7 T12.2-1 | Seismic response coefficient, $R$: | 6 | |
| ASCE 7 T1.5-2 | Importance factor, $I_e$: | 1.0 | |
| ASCE 7 §12.8.2 | Fundamental period of the structure, $T$ (s): | 0.77 | |
| ASCE 7 §12.8.2.1 | Approximate fundamental period, $T_a$ (s): | 0.68 | |
| ASCE 7 §11.4.4 | Design spectral acceleration parameters, $S_{DS}$ and $S_{D1}$ [or by §11.4.7]: | 1.57 | 0.93 |
| ASCE 7 T12.8-1 | Upper limit coefficient on calculated period, $C_u$: | 1.4 | |
| ASCE 7 §12.8.2 | Determine if $T > C_u T_a$ | 0.95 > T use T | |
| ASCE 7 hazard tool | Long-period transition period(s), $T_L$: | 8 | |
| ASCE 7 §12.8.1.1 | Seismic response coefficient, $C_s$: | 0.192 | |
| ASCE 7 §12.7.2 | Portion of effective seismic weight at each level, $w_x$: | See table | |
| ASCE 7 Eq 12.8-1 | Seismic base shear, $V$: | See table | |
| ASCE 7 §12.8.3 | Exponent related to the structure period, $k$: | 1.14 | |
| ASCE 7 Eq 12.8-13 | Vertical distribution factor, $C_{vx}$: | See table | |
| ASCE 7 Eq 12.8-12 | Lateral seismic force, $F_x$: | See table | |

*IBC 2024 and ASCE 7-22

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Example 6.10 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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EXAMPLE

## Equivalent Lateral Force (ELF) Procedure

**Seismic Forces and Seismic Story Shears – N-S Direction**

| Level | Story Weight, $w_x$ (kips) | Height, $h_x$ (ft) | $w_x h_x^k$ | Lateral Force, $F_x$ (kips) | Story Shear, $V_x$ (kips) |
|---|---|---|---|---|---|
| R | 708 | 110.0 | 150,391 | 265.7 | 265.7 |
| 10 | 812 | 99.0 | 152,961 | 270.3 | 536.0 |
| 9 | 812 | 88.0 | 133,741 | 236.3 | 772.3 |
| 8 | 812 | 77.0 | 114,856 | 203.0 | 975.3 |
| 7 | 812 | 66.0 | 96,347 | 170.2 | 1,145.5 |
| 6 | 812 | 55.0 | 78,265 | 138.3 | 1,283.8 |
| 5 | 812 | 44.0 | 60,687 | 107.2 | 1,391.0 |
| 4 | 812 | 33.0 | 43,718 | 77.3 | 1,468.3 |
| 3 | 812 | 22.0 | 27,537 | 48.7 | 1,517.0 |
| 2 | 812 | 11.0 | 12,495 | 22.1 | 1,539.1 |
| Σ | 8,016 | | 870,998 | 1,539.1 | |

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Table 6.46 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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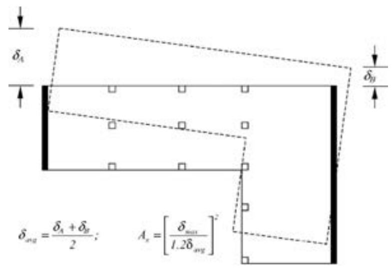
## Amplification of Accidental Torsional Moment

CODE CHANGE

§ 12.8.4.3

- ELF Procedure
- SDC C, D, E, or F
- Type 1 Horizontal Irregularity
- $1.0 \leq A_x \leq 3.0$
- For $\delta_{max}$ and $\delta_{avg}$ can assume rigid diaphragm




ASCE 7-22 Figure 12.8-1

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## ELF – Displacement and Drift

CODE CHANGE

§ 12.8.6

- Significant changes throughout
  - Design earthquake displacement, $\delta_{DE}$
  - Maximum considered earthquake displacement, $\delta_{MCE}$
  - Design story drift, $\Delta$

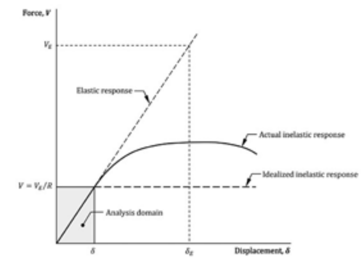


Figure 6.37 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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## ELF – Displacement and Drift

EXAMPLE

§ 12.8.6 & T 12.12-1

Structural separation

| Story | $\delta_e$, in. (mm) | $\delta_{DE}$, in. (mm) | $\Delta$, in. (mm) |
|---|---|---|---|
| 10 | 3.08 (78.2) | 15.40 (391.0) | 2.05 (52.0) |
| 9 | 2.67 (67.8) | 13.35 (339.0) | 2.05 (52.0) |
| 8 | 2.26 (57.4) | 11.30 (287.0) | 2.05 (52.0) |
| 7 | 1.85 (47.0) | 9.25 (235.0) | 1.95 (49.5) |
| 6 | 1.46 (37.1) | 7.30 (185.5) | 1.85 (47.0) |
| 5 | 1.09 (27.7) | 5.45 (138.5) | 1.70 (43.0) |
| 4 | 0.75 (19.1) | 3.75 (95.5) | 1.50 (38.5) |
| 3 | 0.45 (11.4) | 2.25 (57.0) | 1.15 (29.0) |
| 2 | 0.22 (5.6) | 1.10 (28.0) | 0.75 (19.0) |
| 1 | 0.07 (1.8) | 0.35 (9.0) | 0.35 (9.0) |

Allowable story drift: $\Delta_a = 0.020h_{sx} = 0.02(11)(12) = 2.64" > 2.05"$ **OK**

Table 6.47 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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CODE CHANGE

§ 12.10

### Diaphragms, Chords, and Collectors

Diaphragms, chords, and collectors shall be designed in accordance with Sections 12.10.1 and 12.10.2.

**EXCEPTIONS:**

(b) Precast concrete diaphragms in Seismic Design Category B, cast-in-place concrete diaphragms, cast-in-place concrete equivalent precast diaphragms, and wood-sheathed diaphragms supported by wood diaphragm framing, bare steel deck diaphragms, and concrete-filled steel deck diaphragms are permitted to be designed in accordance with Section 12.10.3.

(c) Diaphragms, chords, and collectors in one-story structures that conform to the limitations of Section 12.10.4.1 are permitted to be designed in accordance with Section 12.10.4.

- No changes to Exception "a"
- New provisions for rigid wall flexible diaphragm buildings

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§ 12.10.4

## Alternative Diaphragm Design Provisions – RWFD

**Alternative Diaphragm Design Provisions for One-Story Structures with Flexible Diaphragms and Rigid Vertical Elements**
Where permitted by Section 12.10 and subject to the limitations of Section 12.10.4.1, diaphragm design forces, including design forces for chords, collectors, and their in-plane connections to vertical elements, shall be determined in accordance with Section 12.10.4.2.

- Relatively rigid concrete or masonry walls
- Relatively flexible WSP diaphragms
- a.k.a. rigid wall flexible diaphragm (RWFD)
- Examples
  - Distribution centers
  - "Big-box" retailers such as Costco, Home Depot, Lowes

Photo excerpted from "Horizontal Diaphragm Seismic Design" – *STRUCTURE Magazine* – October 2022

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CODE CHANGE

§ 12.10.4

## Alternative Diaphragm Design Provisions

| Method and ASCE/SEI 7-22 Section | Number of Stories Permitted | Diaphragm Systems Included | Comments |
|---|---|---|---|
| Traditional Sections 12.10.1 and 12.10.2 | Any | All | • Not permitted for precast concrete diaphragms in SDC C through F<br>• Diaphragm design forces are determined using seismic design parameters ($R$, $\Omega_0$, and $C_d$) for the vertical elements of the SFRS |
| Alternative Design Procedure Section 12.10.3 | Any | • Cast-in-place concrete<br>• Precast concrete<br>• Wood structural panel<br>• Bare steel deck<br>• Concrete-filled steel deck | • Required for precast concrete diaphragms in SDC C through F, providing improved seismic performance<br>• Optional for other diaphragm types<br>• Better reflects vertical distribution of diaphragm forces<br>• $R_s$ diaphragm design force reduction factor better reflects effect of diaphragm ductility and displacement capacity on diaphragm seismic forces<br>• Forces in collectors and their connections to vertical elements are amplified by 1.5 in place of $\Omega_0$ |
| Alternative RWFD Design Method Section 12.10.4 | One Story | • Wood structural panel<br>• Bare steel deck<br>• When meeting the scoping limitations of ASCE/SEI 7-22 Section 12.10.4.1 | • Primarily intended for buildings with diaphragm spans of 100 feet or greater<br>• New $T_{diaph}$, $R_{diaph}$, $\Omega_{0\text{-}diaph}$, and $C_{d\text{-}diaph}$ better reflect response of RWFD building type<br>• Provides better performance with the same or reduced construction cost |

Excerpted from "Horizontal Diaphragm Seismic Design" – *STRUCTURE Magazine* – October 2022

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ASCE 7 Seismic Provisions for Buildings Change Summary

1. Three new Seismic Force-Resisting Systems
2. Revisions to defining torsionally irregular structures
3. Direction of loading for SDC D through F significantly revised for Independent Direction Procedure
4. Equivalent lateral force (ELF) procedure more broadly applicable
5. New provisions for rigid wall flexible diaphragm structures

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APPLICATION

**5. Each of the following is a new seismic force-resisting system in Table 12.2-1 except:**

a) Concrete ductile coupled walls
b) Cross-Laminated Timber (CLT) shear walls
c) Steel and concrete coupled composite plate shear walls (SpeedCore)
d) Rigid wall flexible diaphragm structures

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## Resources

Available for use or that can be used for support or help

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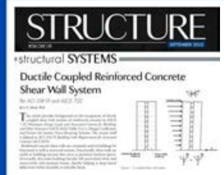
## *STRUCTURE* Magazine Articles




September 2022
https://www.structuremag.org/?p=21341


January 2023
https://www.structuremag.org/?p=24790


October 2022
https://www.structuremag.org/?p=21644


March 2023
https://www.structuremag.org/?p=23058

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## NEHRP Webinar Series

- JAN 20 2022 — INTRODUCTION TO THE 2020 NEHRP RECOMMENDED SEISMIC PROVISIONS: DESIGN EXAMPLES — Virtual Meeting
- FEB 10 2022 — FUNDAMENTALS OF EARTHQUAKE ENGINEERING — Virtual Meeting
- MAR 3 2022 — DIAPHRAGM SEISMIC DESIGN PART 1 — Virtual Training
- MAR 10 2022 — DIAPHRAGM SEISMIC DESIGN PART 2 — Virtual Training
- MAR 31 2022 — REINFORCED CONCRETE DUCTILE COUPLED SHEAR WALLS — Virtual Training
- APR 21 2022 — NONSTRUCTURAL COMPONENTS: FUNDAMENTALS AND DESIGN EXAMPLES – PART 1 — Virtual Meeting
- APR 28 2022 — NONSTRUCTURAL COMPONENTS: FUNDAMENTALS AND DESIGN EXAMPLES – PART 2 — Virtual Meeting
- MAY 19 2022 — EVOLUTION OF SEISMIC DESIGN VALUES OVER THE YEARS AND THE 2018 UPDATE OF THE USGS NATIONAL SEISMIC HAZARD MODEL — Virtual Meeting
- JUN 2 2022 — NEW MULTI-PERIOD RESPONSE SPECTRA AND GROUND MOTION REQUIREMENTS, ADDITIONAL REVISIONS TO GROUND-MOTION PROVISIONS, AND DISSECTION OF EXAMPLE CHANGES TO THE MCER GROUND MOTION VALUES — Virtual Meeting
- JUN 23 2022 — CROSS-LAMINATED TIMBER (CLT) SHEAR WALLS AND RESILIENCE-BASED DESIGN — Virtual Training
- AUG 4 2022 — SEISMIC DESIGN OF COUPLED COMPOSITE PLATE SHEAR WALLS / CONCRETE FILLED (C-PSW/CF) — Virtual Training

https://www.nibs.org/events/nehrp-webinar-series

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COURSE OUTLINE

- Flood Load Provisions
  - Load Combinations
  - Definitions and Symbols
  - Design Provisions
  - Examples
- Tsunami Load Provisions
  - Load Combinations
  - Definitions and Symbols
  - Design Provisions
  - Examples
- Resources

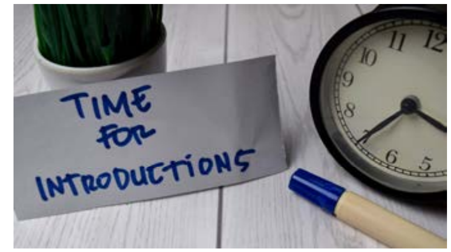
4

4

PARTICIPANTS

**4. Are you located in a tsunami design zone (coastal areas of Alaska, Hawaii, Washington, Oregon or California)?**

a. Yes
b. No
c. I don't know

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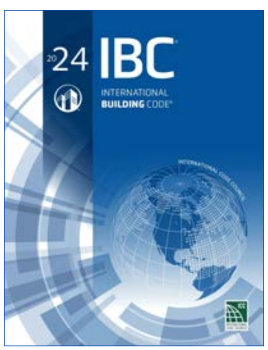
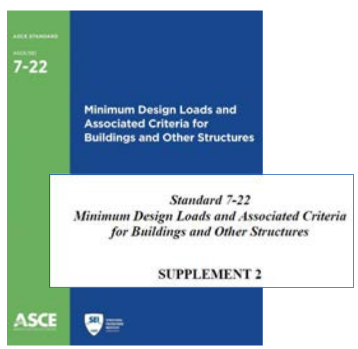
## Codes and Standards

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## Flood Loads

ASCE 7-22 Supplement 2 – Updated based on Risk Category

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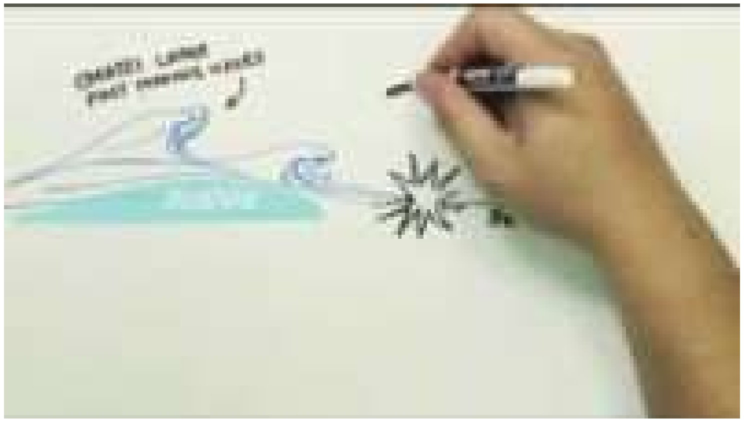

https://www.youtube.com/watch?v=bBa9bVYKLP0
NOAA/NWS National Hurricane Center

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CODE CHANGE

§ 1612.3

## Flood Loads – Design and Construction

The design and construction of buildings and structures located in *flood hazard areas*, including *coastal high hazard areas* and *coastal A zones*, shall be in accordance with Chapter 5 of ASCE 7 and ASCE 24. Elevators, escalators, conveying systems and their components shall conform to ASCE 24 and ASME A17.1/CSA B44 as applicable.

**Exception:** *Temporary structures* complying with Section 3103.6.1.3.

- Elevators & escalators per ASCE 24 and ASME A17.1
- New exception for temporary structures

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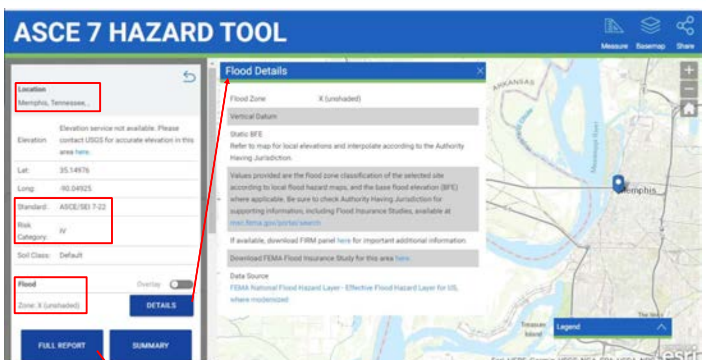
EXAMPLE

## asce7hazardtool.online




Generate PDF...

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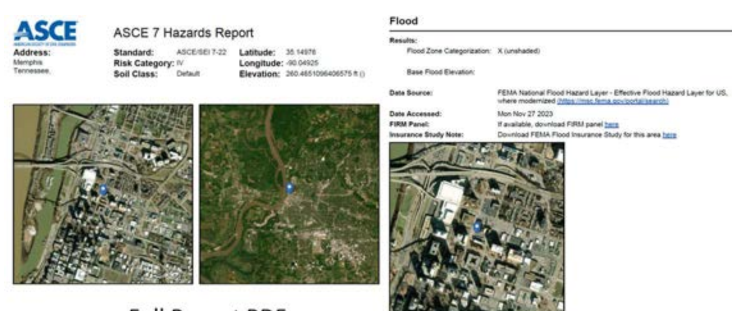
EXAMPLE

## asce7hazardtool.online




Full Report PDF

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CODE CHANGE

S2 § 2.3.2

## Strength Design Flood Load Combinations

When a structure is located in a flood ~~zone~~ hazard area (Section 5.3.1), the following load combinations shall be considered in addition to the basic combinations in Section 2.3.1:

~~In V-Zones or Coastal A-Zones:~~

4b. $1.2D + 1.0W + \cancel{2.0}\ \underline{1.0}F_a + L + (0.5L_r \text{ or } 0.3S \text{ or } 0.5R)$

5b. $0.9D + \cancel{1.0}\ \underline{0.5}W + 2.0F_a$

**Above Grade**

~~In noncoastal A-Zones:~~

~~4b. $1.2D + 0.5W + 1.0F_a + L + (0.5L_r \text{ or } 0.3S \text{ or } 0.5R)$~~

~~5b. $0.9D + 0.5W + 1.0F_a$~~

Exceptions:

1. The load factor on L in the combination 4b is permitted to equal 0.5 for all occupancies in which $L_o$ in Chapter 4, Table 4.3-1 is less than or equal to 100 psf, with the exception of garages or areas occupied as places of public assembly.
2. In the load combination 4b, the companion load $S$ shall be taken as either the flat roof snow load ($p_f$) or the sloped roof snow load ($p_s$).

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CODE CHANGE

§ 2.3.1 & 2.3.2

## Strength Design Flood Load Combinations

Table 7.14 Strength Design Load Combinations Including Flood-Induced Loads for Structural Elements Below Grade

| ASCE/SEI Eq. No. | Load Combination |
|---|---|
| 2a | $1.2D + 1.6(L + H) + (0.5L_r \text{ or } 0.3S \text{ or } 0.5R)$ |
| 3a | $1.2D + (1.6L_r \text{ or } 1.0S \text{ or } 1.6R) + (L \text{ or } 0.5W) + 1.6H$ |
| 4a | $1.2D + 1.0(W \text{ or } W_T) + L + (0.5L_r \text{ or } 0.3S \text{ or } 0.5R) + 1.6H$ |
| 5a | $0.9D + 1.0(W \text{ or } W_T) + 1.6H$ |
| 6 | $1.2D + E_v + E_h + L + 0.15S + 1.6H$ |
| 7 | $0.9D - E_v + E_h + 1.6H$ |

*Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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CODE CHANGE

S2 § 2.4.2

## ASD Flood Load Combinations

When a structure is located in a flood ~~zone~~ hazard area, the following load combinations shall be considered in addition to the basic combinations in Section 2.4.1:

~~In V-Zones or Coastal A-Zones:~~

5b. $D + 0.6W + \cancel{1.5}\ \underline{0.7}F_a$

6b. $D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } 0.7S \text{ or } R) + \cancel{1.5}\ \underline{0.7}F_a$

7b. $0.6D + 0.6W + \cancel{1.5}\ \underline{0.7}F_a$

**Above Grade**

~~In noncoastal A-Zones:~~

~~5b. $D + 0.6W + 0.75F_a$~~

~~6b. $D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } 0.7S \text{ or } R) + 0.75F_a$~~

~~7b. $0.6D + 0.6W + 0.75F_a$~~

Exception:

In the load combination 6b, the companion load $S$ shall be taken as either the flat roof snow load ($p_f$) or the sloped roof snow load ($p_s$).

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CODE CHANGE

§ 2.4.1 & 2.4.2

## ASD Flood Load Combinations

Table 7.15 Allowable Stress Design Load Combinations Including Flood-Induced Loads for Structural Elements Below Grade

| ASCE/SEI Eq. No. | Load Combination |
|---|---|
| 2a | $D + L + H$ |
| 3a | $D + (L_r \text{ or } 0.7S \text{ or } R) + H$ |
| 4a | $D + 0.75L + 0.75(L_r \text{ or } 0.7S \text{ or } R) + H$ |
| 5a | $D + 0.6(W \text{ or } W_T) + H$ |
| 6a | $D + 0.75L + 0.75[0.6(W \text{ or } W_T)] + 0.75(L_r \text{ or } 0.7S \text{ or } R) + H$ |
| 7a | $0.6D + 0.6(W \text{ or } W_T) + H$ |
| 8 | $D + 0.7E_v + 0.7E_v + H$ |
| 9 | $D + 0.525E_v + 0.525E_h + 0.75L + 0.1S + H$ |
| 10 | $0.6D - 0.7E_v + 0.7E_h + H$ |

*Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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## Summary of ASCE 7-22 s2 Flood Load Procedures

1. Flood hazard area
2. Design stillwater flood elevation
3. Design flood velocity
4. Design wave height
5. Scour depth
6. Debris impact and damming
7. Breakaway walls
8. Hydrostatic loads and forces
9. Hydrodynamic loads and forces
10. Drag on components and buildings
11. Breaking and nonbreaking wave loads
12. Debris impact loads
13. Load combinations and stability against uplift and sliding


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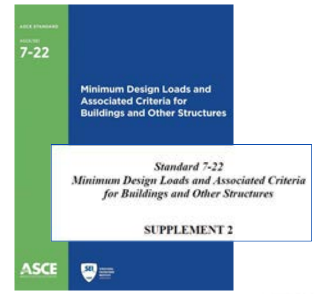
22

CODE CHANGE

S2 Ch 5

## Summary of ASCE 7-22 s2 Major Flood Load Changes

- Flood Hazard Area increased from 100-year flood plain to 500-year flood plain for Risk Categories II, III, and IV structures
- Significant reorganization
- New provisions
  - Flood Velocity
  - Wave Effects
  - Scour
  - Debris Impact

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CODE CHANGE

S2 §5.3-5.5

## Flood Loads – New and Reorganized Sections

**5.3 DESIGN REQUIREMENTS**
5.3.1 Flood Hazard Area
5.3.2 Design Loads
5.3.3 Design Stillwater Flood Depth
5.3.4 Effects of Relative Sea Level Change
5.3.5 Erosion
5.3.6 Flood Velocity
5.3.7 Wave Effects
5.3.8 Scour – moved and updated
5.3.9 Debris
5.3.10 Loads on Breakaway Walls
5.3.11 Site-Specific Studies
5.3.12 Performance-Based Design

**5.4 LOADS DURING FLOODING**
5.4.1 Load Basis
5.4.2 Hydrostatic Loads
5.4.3 Hydrodynamic Loads
5.4.4 Wave Loads
5.4.5 Debris Impact Loads

**5.5 FLOOD LOAD CASES**
5.5.1 Stability Against Uplift
5.5.2 Stability Against Sliding

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CODE CHANGE

S2 §5.3.1

## Flood Hazard Area

For Risk Categories II, III, and IV structures, the Flood Hazard Area shall be the 500-year floodplain designated as the Special Flood Hazard Area and the Shaded X-Zone. For Risk Category I structures, the Flood Hazard Area shall be the 100-year floodplain designated as the Special Flood Hazard Area.

- 500-year floodplain for RC II, III and IV structures
- Better aligns flood hazards with approach to wind and other hazards
- Increases flood depth, which applies loads distributed over a larger height, instead of increasing force over 100-year flood depth on a structure

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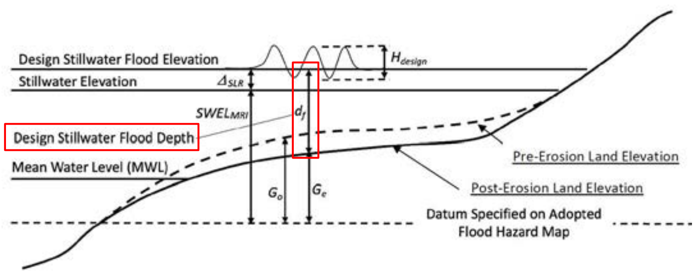
CODE CHANGE

S2 §F5.2-1

## Design Stillwater Flood Depth, $d_f$




Flood Parameters in Coastal Areas

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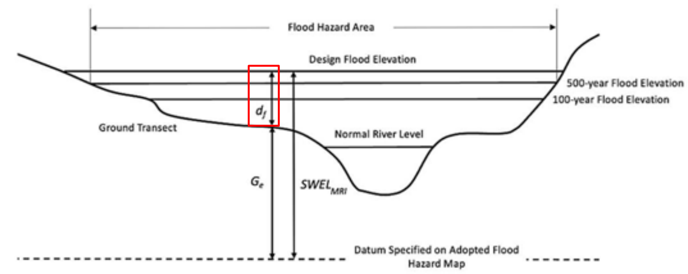
CODE CHANGE

S2 §F5.2-2

## Design Stillwater Flood Depth, $d_f$




Flood Parameters in Riverine Areas

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CODE CHANGE

S2 §5.3.4

## Effects of Relative Sea Level Change

The effects of relative sea level change shall be included in the calculation of flood conditions and flood loads for sites whose flooding comes from coastal sources. A project lifecycle of not less than 50 years shall be used for this quantification. The minimum rate of relative sea level change shall be the historically recorded sea level change rate for the site over a 50-year period. The increase in relative sea level during the project lifecycle of the structure shall be added to the design stillwater flood elevation as required by Section 5.3.3.

- Significant uncertainty exists in predicting sea level change
- Lower bound is straight-line projection of 50-years of data
- USACE Sea Level Change Curve Calculator may be used
  - https://cwbi-app.sec.usace.army.mil/rccslc/slcc_calc.html
- ASCE Manual on Engineering Practice No. 140, *Engineering Practice, Climate-Resilient Infrastructure: Adaptive Design and Risk Management* (2018)

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CODE CHANGE

S2 §5.3.6.1

## Flood Velocity – Coastal Areas

For coastal areas, the velocity of water $V$ in the absence of neighboring structures shall be obtained by one of the following three methods: (1) by using Equation (5.3-4), (2) by numerical modeling, or (3) by laboratory testing (physical modeling). When Method 2 or 3 are used, design flood parameters shall be determined using site-specific studies in accordance with Section 5.3.11.

$$V = C_V (g\, d_f)^{1/2} \qquad (5.3\text{-}4)$$

where:

$V$ = Design flood velocity (ft/s)

$g$ = Acceleration due to gravity = 32.2 ft/s$^2$

$d_f$ = Design stillwater flood depth (ft)

$C_V$ = Velocity coefficient = 0.5

- 3 methods: calculations, modelling or testing
- Calculation method shown

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S2 §5.3.6.1

## Flood Velocity – Coastal Areas

- The maximum velocity of water, $V_{max}$, for coastal areas need not be greater than $C_{VMAX}$ *10 ft/s, where $C_{VMAX}$ is the coefficient obtained from Table 5.3-2 used to scale to the maximum velocity

**Table 5.3-2. Design Flood MRI and Scaling Factors for Maximum Velocity**

| Risk Category | All Coastal Sites $C_{Vmax}$ | $V_{max}$ (ft/s) |
|---|---|---|
| I | 1.00 | 10.0 |
| II | 1.35 | 13.5 |
| III | 1.45 | 14.5 |
| IV | 1.50 | 15.0 |

*ASCE 7-22 Supplement 2*

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CODE CHANGE

S2 §5.3.6.2

## Flood Velocity – Riverine Areas

For riverine areas, the average velocity of water $V$ shall be obtained by one of the following four methods:
(1) from a flood hazard study,
(2) by analytical methods using open channel flow hydraulics,
(3) by numerical modeling, or
(4) by laboratory testing (physical modeling).
When Method 2, 3 or 4 are used, design flood parameters shall be determined using the site-specific hazard procedures of Section 5.3.11.

- 4 methods: study, calculations, modeling or testing

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CODE CHANGE

S2 §5.3.7

## Wave Effects

The effects of waves shall be included for both V-Zones and A-Zones. In areas subjected to riverine flooding only, the effects of waves are permitted to be neglected.

- Wave height, wave period and wavelength are moved from §5.4
- A-zones: waves < 3’
- V-zones: coastal high hazard

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CODE CHANGE

S2 §5.3.7.1

## Design Wave Height, $H_{design}$

The design wave height $H_{design}$ at the site in ft shall be obtained by one of the following four methods: (1) by assuming depth-limited breaking wave conditions, (2) from a flood hazard study, (3) by numerical modeling, or (4) by laboratory testing (physical modeling). When Method 3 or 4 are used, design flood parameters shall be determined using the site-specific hazard procedures of Section 5.3.11.

- Method 1 unchanged: $H_{design} = H_b = 0.78\ d_f$
- Methods 2, 3 or 4
  - $H_{design} = H_c = 1.6\ H_s$
  - $H_c$ = Controlling wave height (ft)
  - $H_s$ = Significant wave height (ft)

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CODE CHANGE
S2 §5.3.7.1

## Wave Height

- If the controlling wave height is specified by a 100-year design flood, then the controlling wave height shall be adjusted to the controlling wave height corresponding to the MRI design flood event using Table 5.3-3 such that $H_{cMRI} = C_{HC} * H_{c100}$

**Table 5.3-3 Design Flood MRI and Scaling Factors for Controlling Wave Height**

| Risk Category | All Coastal Sites $C_{HC}$ |
|---|---|
| I | 1.00 |
| II | 1.30 |
| III | 1.35 |
| IV | 1.40 |

*ASCE 7-22 Supplement 2*

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CODE CHANGE
S2 §5.3.7.2

## Wave Period

The wave period $T_p$ in sec (s) corresponding to the wave height shall be obtained by one of the following four methods: (1) by using Equation (5.3-9), (2) from a flood hazard study, (3) by numerical modeling, or (4) by laboratory testing (physical modeling). When Methods 3 or 4 are used, design flood parameters shall be determined using site-specific studies in accordance with Section 5.3.11.

$T_p = C_t (H_{design} / g)^{1/2}$ (5.3-9) $\boxed{T_p = 2.13 (H_{design})^{1/2}}$

where:

$T_p$ = Wave period corresponding to the wave height, in sec (s)

$C_T$ = Wave period coefficient equal to 12.1

$g$ = Acceleration due to gravity, 32.2 ft/s²

- 4 methods: calculations, study, modeling or testing
- Calculation method shown

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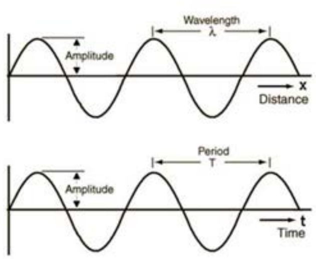
CODE CHANGE
S2 Eq 5.3-10

## Wavelength

- The wavelength, L, in ft shall be calculated by

$$L = \frac{gT_p{}^2}{2\pi}\left(1 - e^{-\left(\frac{2\pi}{T_p}\sqrt{\frac{d_f}{g}}\right)^{\frac{5}{2}}}\right)^{\frac{2}{5}}$$




www.sengpielaudio.com/calculator-period.htm

40

CODE CHANGE
S2 §5.3.8

## Scour Depth, $S_m$

- Walls
  - Nonbreaking waves (5.3-11)
  - Breaking waves (5.3-12)
- Vertical piles and columns
  - (5.3-13)

| | |
|---|---|
| $S_m = \frac{0.25\, H_{design}}{\left[\sinh\left(\frac{2\pi d_f}{L}\right)\right]^{1.35}}$ | 5.3-11 |
| $S_m = H_{design}$ | 5.3-12 |
| $S_m = 2.0D$ (round)<br>$S_m = 1.4D$ (square) | 5.3-13 |

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CODE CHANGE

S2 §5.3.9

## Debris Impact and Damming

Risk Categories II, III, and IV structures shall be designed for debris impact and debris damming in accordance with this section where the Design Stillwater Flood Depth ($d_f$) is greater than 3 ft.

- RC II, III and IV
- $d_f$ > 3 ft
- Impact
- Damming

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CODE CHANGE

S2 §5.3.9.1

## Debris Impact

Structures within the Flood Hazard Area shall be designed for debris impact loads as determined by Section 5.4.5.1. Debris impact loads shall be considered in any direction and at heights as required per Section 5.4.5.2. Debris impact loads need not be considered on multiple structural elements simultaneously.

**EXCEPTIONS:**

1. Only considered from directions shown in site-specific study +/- 22.5°
2. Riverine sites: only considered from upstream direction +/- 22.5° from primary flow direction
3. Detached one- and two-family dwellings
4. Risk Category II structures outside Special Flood Hazard Area

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CODE CHANGE

S2 T5.3-4

## Debris Impact Objects

| Debris Type | Applicable Risk Categories | Threshold Depth (ft)[1] | Impact on columns, piles, bearing walls and transfer beams | Impact on non-load bearing elements[2] |
|---|---|---|---|---|
| Passenger Vehicles | RC II/III/IV | 3 ft (0.91 m) | Yes | Yes |
| Small Vessels | RC II/III/IV | 3 ft (0.91 m) | Yes[3] | Yes[3] |
| Wood Poles | RC III/IV | 3 ft (0.91 m) | Yes | Yes |
| Shipping Containers | RC III/IV | 3 ft (0.91 m) | Yes[3] | n/a |
| Ships/barges | RC III/IV | 6 ft (1.8 m) | Yes[3] | n/a |
| Extraordinary Debris[3] | RC IV | 12 ft (3.7 m) | Yes[3 4] | n/a |

*ASCE 7-22 Supplement 2*

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CODE CHANGE

S2 T5.3-5

## Site Hazard Assessment

| Debris Type | Travel distance in moderate density environment * | Travel distance in heavy density environment [1] |
|---|---|---|
| Small vessels | 2,000 ft (604 m) | 1,000 ft (304 m) |
| Shipping Containers | 2,000 ft (604 m) | 1,000 ft (304 m) |
| Ships/barges | 1,000 ft (302 m) | 500 ft (152 m) |
| Extraordinary debris | 1,000 ft (302 m) | 500 ft (152 m) |

*Heavy density environments are areas where the density of structures with a height of at least 75% of the design flood depth is greater than 30% of plan area within the Flood Hazard Area. All other areas shall be considered moderate density.

**EXCEPTION:** Debris impact loads from shipping containers, ships, and barges not required where

- Flood depth < debris object draft plus 2.0 ft
- Debris object path blocked by structure/topography resulting in inadequate draft

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EXAMPLE

S2 §C5.3.9

## Debris Site Hazard Assessment

Container Yard in Gulfport, Mississippi


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EXAMPLE

S2 §C5.3.9

## Debris Site Hazard Assessment

Container Yard in Charleston, South Carolina


*ASCE 7-22 Supplement 2*

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CODE CHANGE

S2 §5.3.10

### Breakaway Walls

Where required by ASCE/SEI 24 to break away, walls and partitions, including their connections to the structure, shall be designed in accordance with this section. The wall shall be designed to resist the following loads acting perpendicular to the plane of the wall:

1. The wind load specified in this standard,
2. The seismic load specified in Chapter 12, ~~and 10 psf~~
3. The lateral earth pressure specified in Chapter 3, and
4. $16\ \text{lb/ft}^2$

If the largest of the loads above is less than the 100-year flood load, the wall shall be designed to fail during the 100-year flood condition. The structure and its foundation shall be designed against collapse, permanent lateral displacement, and other structural damage due to the expected failure forces as walls break away.

- Moved from 5.3.3
- Designed as breakaway if flood loads control

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CODE CHANGE

S2 §5.3.11

## Site-specific Studies

**Table 5.3-6. Maximum Allowable Reductions for Site-Specific Studies.**

| Hazard | Allowable Reduction with Peer Review | Allowable Reduction without Peer Review |
|---|---|---|
| Velocity, $V$ | 30% | 20% |
| Wave height, $H$ | 30% | 20% |
| Wave period, $T$ | 30% | 20% |

- Moved from 5.3.3
- Velocity and wave height reductions reduce hydrodynamic and wave loads
- Wave height and period reductions reduce amount of scour at walls
- Reduction limits intend to reduce underestimation

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S2 §5.4.2

## Hydrostatic Loads

Hydrostatic loads caused by a depth of water to the design stillwater flood elevation shall be applied over all surfaces contacted, both above and below ground level. Hydrostatic forces shall be calculated in accordance with Sections 5.4.2.1 and 5.4.2.2. The hydrostatic pressures shall be calculated utilizing basic fluid mechanics by applying pressures perpendicular to wetted surfaces proportional to the depth of water such that

$p_h = \gamma_w z$ (5.4-1)

where:

$p_h$ = Hydrostatic pressure at a given depth z (lb/ft$^2$)

$\gamma_w$ = Specific weight of water, taken as 62.4 lb/ft$^3$ for freshwater and 64 lb/ft$^3$ for saltwater

$z$ = Depth below design stillwater flood elevation (ft)

- Previously performance-based
- Reduced loads if provisions allow floodwater entry and exit

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CODE CHANGE

S2 §5.4.2.1

## Vertical Hydrostatic Force – Buoyancy

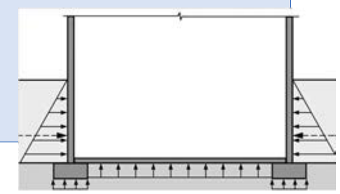
...The vertical uplift force caused by buoyancy for determination of structure uplift shall be applied at the centroid of the submerged volume of the structure, and shall be calculated using Equation (5.4-2):

$F_B = \gamma_w V_w$ (5.4-2)

where:

$F_B$ = Uplift force caused by buoyancy (lb)

$V_w$ = Volume of displaced water (ft$^3$)



- Previously performance-based

*Structural Load Determination: 2024 IBC and ASCE/SEI 7-22 – McGraw Hill*

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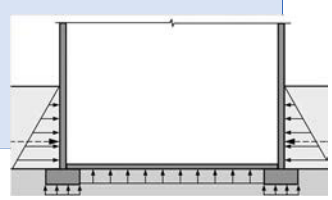
CODE CHANGE

S2 §5.4.2.2

## Lateral Hydrostatic Force

The lateral force, $F_h$, caused by the hydrostatic pressure on one side of a vertical wall per unit width, lb/ft shall be calculated by Equation (5.4-3):

$F_h = (1/2)\, \gamma_w d_f^2$ (5.4-3)



- Previously performance-based

*Structural Load Determination: 2024 IBC and ASCE/SEI 7-22 – McGraw Hill*

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CODE CHANGE

S2 T5.4-1

## Drag Coefficients for Components

| Structural Element Section | Drag Coefficient, $C_d$ |
|---|---|
| Round column or equilateral polygon with six sides or more | 1.2 |
| Rectangular column of at least 2:1 aspect ratio with longer face oriented parallel to flow | 1.6 |
| Free-standing wall submerged parallel to flow | 1.6 |
| Square or rectangular column with longer face oriented perpendicular to flow | 2.0 |
| Triangular column pointing away from flow | 2.0 |
| Wall or flat plate, normal to flow | 2.0 |
| Diamond-shape column, pointed into the flow (based on face width, not projected width) | 2.5 |
| Rectangular beam, normal to flow | 2.0 |
| I, L, and channel shapes | 2.0 |
| Structural components with debris damming per Section 5.3.9.2 | 2.0 |

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CODE CHANGE

S2 T5.4-2

## Drag Coefficients for Buildings

| Ratio of structure width to design stillwater flood depth* $B/d_f$ | Drag Coefficient, $C_d$ |
|---|---|
| ≤12 | 1.25 |
| ≥ 120 | 2.0 |

*Linear interpolation shall be used for intermediate values of $b/d_f$. Where building setbacks occur, drag coefficients shall be determined for each portion of constant width.

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CODE CHANGE

S2 §5.4.4

## Wave Loads

5.4.4 Wave Loads
- 5.4.4.1 Wave Loads on Vertical Piles and Columns
  - 5.4.4.1.1 Nonbreaking Wave Loads on Vertical Piles and Columns
  - 5.4.4.1.2 Breaking Wave Loads on Vertical Piles and Columns
- 5.4.4.2 Lateral Wave Loads on Walls
  - 5.4.4.2.1 Lateral Nonbreaking Wave Loads on Non-elevated Vertical Walls
  - 5.4.4.2.2 Lateral Breaking Wave Loads on Non-elevated Vertical Walls
  - 5.4.4.2.3 Lateral Breaking Wave Loads on Nonvertical Walls
  - 5.4.4.2.4 Lateral Breaking Wave Loads from Obliquely Incident Waves
  - 5.4.4.2.5 Lateral Wave Loads on Elevated Walls

- Reorganized and revised

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CODE CHANGE

S2 §5.4.4

## Wave Loads

| Section | Title | Equation |
|---|---|---|
| 5.4.4.1.1 (5.4-6) | Nonbreaking Wave Loads on Vertical Piles and Columns | $F_m = \phi_m C_D \gamma_w H_{design}^2 D$ |
| 5.4.4.1.2 (5.4-7) | Breaking Wave Loads on Vertical Piles and Columns | $F_{bw} = \phi_m C_{bw} \gamma_w H_{design}^2 D$ |
| 5.4.4.2.1 (5.4-12) | Lateral Nonbreaking Wave Loads on Non-elevated Vertical Walls | $F_t = \begin{cases} \left(\frac{1}{2}(p_1 + p_2)h_c + \frac{1}{2}(p_1 + p_3)d_f\right) & \text{for } \eta^* > h_c \\ \left(\frac{1}{2}p_1(\eta^*) + \frac{1}{2}(p_1 + p_3)d_f\right) & \text{for } \eta^* \le h_c \end{cases}$ |
| 5.4.4.2.2 (5.4-13) | Lateral Breaking Wave Loads on Non-elevated Vertical Walls | $F_{BRK} = \begin{cases} \left(\frac{1}{2}(p_{1B} + p_2)h_c + \frac{1}{2}(p_{1B} + p_3)d_f\right) & \text{for } \eta^* > h_c \\ \left(\frac{1}{2}p_{1B}(\eta^*) + \frac{1}{2}(p_{1B} + p_3)d_f\right) & \text{for } \eta^* \le h_c \end{cases}$ |
| 5.4.4.2.3 (5.4-15) | Lateral Breaking Wave Loads on Nonvertical Walls | $F_{BNV} = F_{BRK} [\sin(\alpha_V)]^2$ |
| 5.4.4.2.4 (5.4-16) | Lateral Breaking Wave Loads from Obliquely Incident Waves | $F_{BOI} = F_{BRK} [\sin(\alpha_H)]^2$ |
| 5.4.4.2.5 (5.4-18) | Lateral Wave Loads on Elevated Walls | $F_t = \begin{cases} \left(\frac{1}{2}p_4(\eta^* - a)\right), & \text{for } a \ge 0 \\ \left(\frac{1}{2}p_1\eta^* + \frac{1}{2}(p_1 + p_4)(\lvert a \rvert)\right), & \text{for } a < 0 \end{cases}$ |

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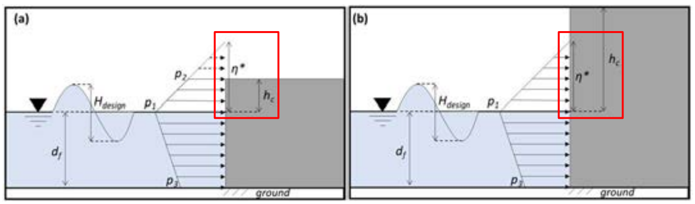
CODE CHANGE

S2 F5.4-1 & §5.4.4.2.1

## Wave Loads




**Normally incident (nonbreaking) wave pressures against a non-elevated vertical wall**

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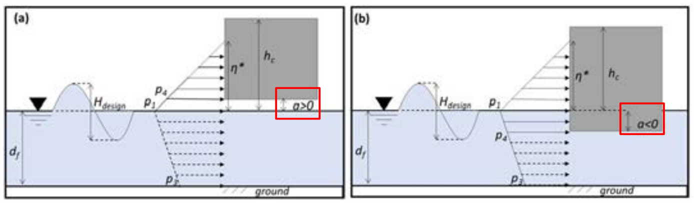
S2 F5.4-2 & §5.4.4.2.5

## Wave Loads




Normally incident (nonbreaking) wave pressures on an **elevated wall**

*ASCE 7-22 Supplement 2*

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CODE CHANGE

S2 §5.4.5

## Debris Impact Loads

| Section | Title | Equation or Method |
|---|---|---|
| 5.4.5.1.1 (5.4-19) | Simplified Debris Impact Load for Passenger Vehicles or Small Vessels | $F_{di} = C_o$ 51,000 (lb) |
| 5.4.5.1.2 (5.4-20) | Elastic Debris Impact Loads | $F_{di} = C_o V C_R C_s (k_e m_{debris})^{1/2}$ |
| 5.4.5.1.3 | Alternative Methods of Debris Impact Analysis | Mass-spring or work-energy method |

OR

- Previously performance-based

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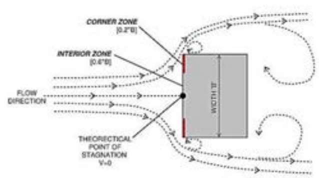
CODE CHANGE

S2 §C5.4.5

## Debris Velocity Stagnation Coefficient, $C_s$



| Location on building | Debris velocity stagnation coefficient ($C_s$) |
|---|---|
| Within a distance of the greater of 0.2*B* or 10 ft (3.05 m) at edges of building | 1.00 |
| Middle 0.6*B* of building | 0.50 |

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CODE CHANGE

S2 §5.4.5.2

## Debris Types and Properties

| Debris Type | Minimum debris weight, lb ($W_{debris}$) | Minimum elastic debris stiffness, lb/ft ($k_e$) | Strike Elevation Range, ft | Impact Area, ft |
|---|---|---|---|---|
| Wood Log/Pole | 1,000 | 4,200,000 | 3.0 AG to $d_f$ | 1.5 x 1.5 |
| Passenger Vehicle | 2,400 | 72,000 | 3.0 AG to ($d_f$ – 1.0) | 5 wide x 2 high |
| Small Vessels | 2,500 | 360,000 | 3.0 AG to ($d_f$ + 3.0) | 4 wide x 2 high |
| 20 ft Shipping Container | 5,000 | 2,940,000 | 3.0 AG to $d_f$ | 1.0 x 1.0 |
| 40 ft Shipping Container | 8,400 | 2,040,000 | | |
| Ships/Barges | Established based on local conditions | | | |

AG = above grade

Adapted from Table 5.4-4 of *ASCE 7-22 Supplement 2*

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S2 §5.5.1

### Stability Against Uplift

Structures shall be designed to resist flotation due to buoyancy forces as defined in Section 5.4.2.1. Uplift resistance shall be provided by satisfying Equation (5.5-1) with load factors as shown. This stability load combination is in addition to those in Chapter 2.

$$0.9D_{SW} + R_B + F_B + 0.6W_{uplift} \geq 0 \qquad (5.5\text{-}1)$$

where:

$D_{SW}$ = Self-weight of the structure or portion of structure being evaluated inclusive of permanent fixed elements and equipment (lb)

$R_B$ = Allowable uplift resisting capacity of structural foundation elements and/or other conditions resisting uplift (lb)

$F_B$ = Uplift force caused by buoyancy in lb, always taken as less than zero

$W_{uplift}$ = Maximum total vertical uplift wind load on the structure as defined in this standard (lb). Wind load shall not be used to counteract buoyancy and is always taken less than zero.

- In addition to Chapter 2 load combinations
- Buoyancy and wind uplift offset by dead loads and foundation elements

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CODE CHANGE

S2 §5.5.2

### Stability Against Sliding

Sliding resistance shall be provided by satisfying Equation (5.5-2) with load factors as shown. This stability load combination is in addition to those in Chapter 2:

$$\mu(0.9D_{SW} + F_B + 0.6W_{uplift}) + H_p + R_p - H_a - F_{lateral} - 0.6W_{lateral} \geq 0 \qquad (5.5\text{-}2)$$

where:

$\mu$ = Coefficient of sliding friction at slip plane being considered between structure on shallow foundations and subgrade

$D_{SW}$ = Self-weight of the structure or portion of structure being evaluated inclusive of permanent fixed elements and equipment (lb)

$F_B$ = Uplift force caused by buoyancy (lb) always taken as less than zero

$W_{uplift}$ = Maximum total vertical uplift wind load on the structure as defined in this standard (lb). Wind load shall not be used to counteract buoyancy and is always taken less than zero

$H_p$ = Resultant force from passive lateral earth pressures (lb)

$R_p$ = Allowable lateral resisting capacity of deep foundations, external structural foundation elements and/or other conditions resisting sliding (lb)

$H_a$ = Resultant force from active lateral earth pressures (lb)

$W_{lateral}$ = Maximum total lateral wind load on the structure as defined in this standard (lb)

$F_{lateral}$ = Maximum lateral component of the flood load, $F_a$, as determined in Section 5.5 (lb)

- In addition to Chapter 2 load combinations

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ASCE 7-22s2 Flood Loads Criteria Change Summary

1. Flood Hazard Area increased from 100-year flood plain to 500-year flood plain for Risk Categories II, III, and IV structures
2. Significant reorganization
3. New provisions for flood velocity, wave effects, scour and debris impact
4. Revised and new load combinations

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APPLICATION

**4. Flood Hazard Areas are based on what flood plain for Risk Categories II, III, and IV structures?**

a) 100-year
b) 200-year
c) 500-year
d) 1000-year

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# Tsunami Loads

Alaska, Hawaii, Washington, Oregon and California Coasts


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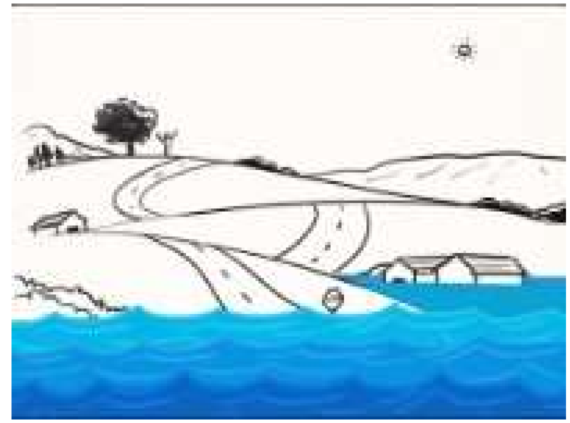

https://youtu.be/x0GX_kc7JZo
https://www.weather.gov/safety/tsunami-outreach

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CODE CHANGE

§ 1615.1

## Tsunami Loads – General

The design and construction of Risk Category III and IV buildings and structures located in the Tsunami Design Zones defined in the Tsunami Design Geodatabase shall be in accordance with Chapter 6 of ASCE 7, except as modified by this code.

**Exception**: Temporary structures complying with Section 3103.6.1.6.

- New exception for temporary structures

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## Summary of ASCE 7-22 Tsunami Procedures

Chapter 6

1. Determine tsunami risk category
2. Determine permitted analysis procedure
3. Perform $R/H_T$ analysis
4. Perform energy grade line analysis
5. Determine hydrostatic loads
6. Determine hydrodynamic loads
7. Determine debris impact loads
8. Determine tsunami load combinations

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## Risk Category

| Risk Category | Use or Occupancy |
|---|---|
| I | Buildings and other structures that represent a low risk to humans. |
| II | All buildings and other structures except those listed in risk categories I, III, and IV. |
| III | • Buildings and other structures the failure of which could pose a substantial risk to human life.<br>• Buildings and other structures with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure. |
| IV | • Buildings and other structures designated as essential facilities.<br>• Buildings and other structures, the failure of which could pose a substantial hazard to the community. |

- Tsunami Vertical Evacuation Refuge Structures (TVERS) are RC IV

§ 6.4

Table 7.17 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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## Analysis Procedures

§ 6.1 & 6.2

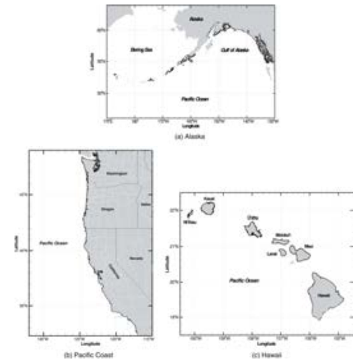

Fig 6.1-1 Inundation depth and runup elevation given

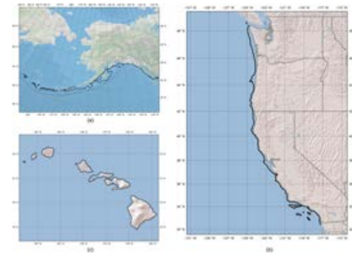

Fig 6.7-1 Offshore tsunami amplitude and period used to calculate runup

*ASCE 7-22 Supplement 2*

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## EXAMPLE

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## Analysis Procedures

§ 6.5

**Table 7.18 Inundation Depth and Flow Velocity Analysis Procedures Where Runup Is Given in the Tsunami Design Zone Maps of ASCE/SEI Figure 6.1-1**

| Analysis Procedure | Tsunami Risk Category (TRC) | | | |
|---|---|---|---|---|
| | II | III | IV: Excluding TVERS | IV: TVERS |
| $R/H_T$ analysis (ASCE/SEI 6.5.1.1) | Not permitted | Not permitted | Not permitted | Not permitted |
| Energy grade line analysis (EGLA) [ASCE/SEI 6.6] | Required* | Required* | Required | Required |
| Site-specific probabilistic tsunami hazard analysis (PTHA) [ASCE/SEI 6.7] | Permitted* | Permitted* | Required** | Required |

*NR if inundation depth is less than or equal to 3 ft and includes sea level rise
**PTHA required if inundation depth > 12 feet

*Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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## Analysis Procedures

Table 7.19 Inundation Depth and Flow Velocity Analysis Procedures Where Runup Is Calculated from ASCE/SEI Figure 6.7-1

| Analysis Procedure | Tsunami Risk Category (TRC) | | | |
|---|---|---|---|---|
| | II | III | IV | |
| | | | Excluding TVERS | TVERS |
| $R/H_T$ analysis (ASCE/SEI 6.5.1.1) | Required | Required | Not permitted | Not permitted |
| Energy grade line analysis (EGLA) [ASCE/SEI 6.6] | Required* | Required* | Required | Required |
| Site-specific probabilistic tsunami hazard analysis (PTHA) [ASCE/SEI 6.7] | Permitted* | Permitted* | Required | Required |

*NR if inundation depth is less than or equal to 3 ft and includes sea level rise

§ 6.5

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Structural Load Determination: 2024 IBC and ASCE/SEI 7-22 – McGraw Hill

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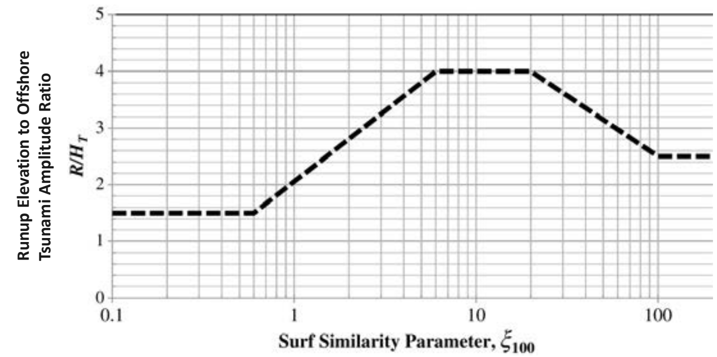
## Analysis Procedures




ASCE 7-22 Supplement 2

§ 6.5.1.1

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CODE CHANGE

## Summary of ASCE 7-22 Tsunami Load Changes

- Updated subsidence maps
- High-resolution maps for highly populated areas (CA)
- Higher order models permitted
- Overtopped wall pressure provisions updated
- Tumbling debris impact on interior columns for slab on grade design
- Building drag coefficient simplified
- Clarification for push-over analysis
- Provisions for hydrodynamic load on pipes
- Debris impact zone extended to grounding limit or resilient structures
- Debris damming for warehouses and parking garages
- Improved provisions for scour and pore pressure softening around foundations
- Exception for SDC D-F if SFRS is adequate to resist tsunami loads

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CODE CHANGE

## Definitions

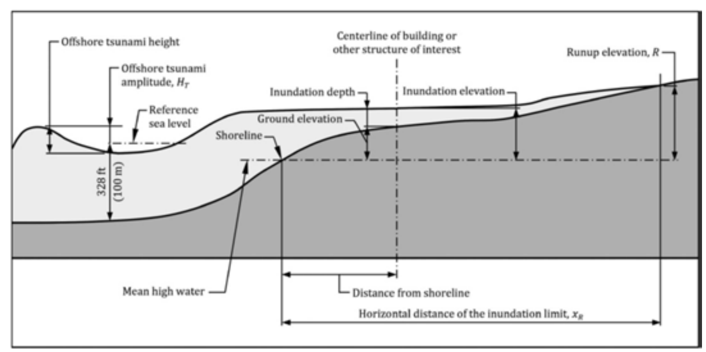


Figure 7.13 from Structural Load Determination: 2024 IBC and ASCE/SEI 7-22 – McGraw Hill

§6.2

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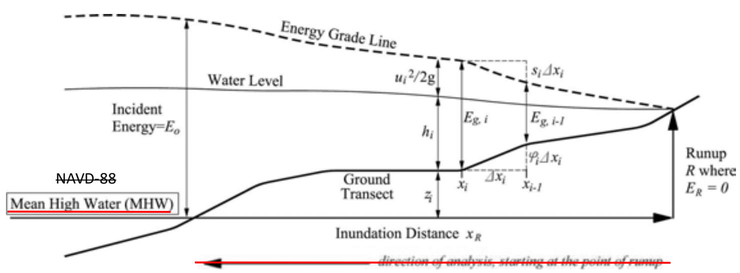
F6.6-1

## EGL for Inundation Depth and Velocity

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CODE CHANGE

§ 6.7.5.3

## Use of Higher-Order Tsunami Model Features

Higher-order tsunami model features, including wave dispersion, elasticity of the earth, variation in gravity, water compressibility, or water density stratification, shall be permitted as analysis enhancements for use within the same model software and parameters selected to comply with Section 6.7, including but not limited to bathymetric and terrain data, grid configuration, and bottom friction, provided these features are also validated in accordance with Section 6.7.6.7 and the higher-order tsunami model satisfies the following conditions:

{2 conditions not shown for brevity}

- New provisions
- Provide refinement in tsunami arrival time and shape of incoming waves
- Peer review required to ensure that tsunami inundation resulting from higher-order model is consistent with probabilistic hazard level

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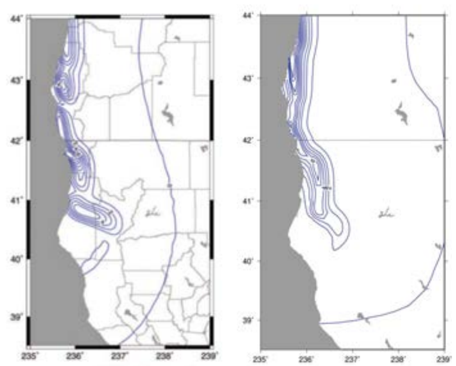
CODE CHANGE

F6.7-3

## Regional Ground Subsidence



ASCE 7-16 Northern California ASCE 7-22

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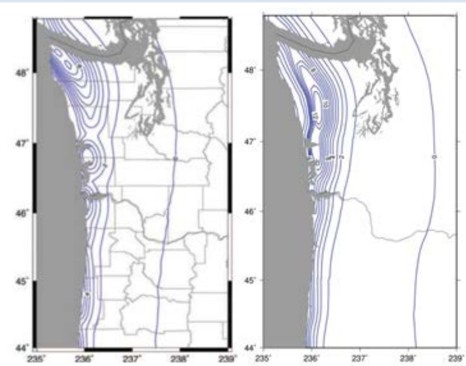
CODE CHANGE

F6.7-3

## Regional Ground Subsidence



ASCE 7-16 Washington & Oregon ASCE 7-22

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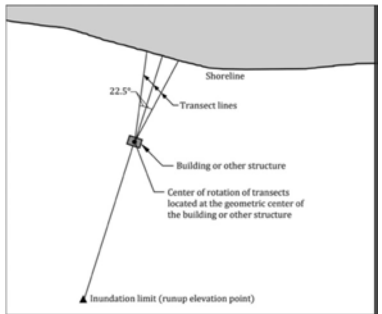
§ 6.7.6.8

## Determining Site-Specific Inundation Flow Parameters

- Where a site-specific inundation analysis spatially defines the topographic diversion of the direction of inundation flow across the Tsunami Design Zone, the minimum flow velocity limits for the site determined by Section 6.6 shall be permitted to be determined using angularly segmented transects that follow the site-specific inundation flow field vectors from shoreline to Runup, subject to the requirements of Section 6.8.6.2.




asce7tsunami.online can provide transects

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CODE CHANGE

T6.8-1

## Tsunami Importance Factors

| Tsunami Importance Factors for Hydrodynamic and Impact Loads | |
|---|---|
| Tsunami Risk Category | $I_{tsu}$ |
| II | 1.0 |
| III, Tsunami Risk Category IV, Vertical Evacuation Refuges, and Tsunami Risk Category III Critical Facilities | 1.25 |
| ~~Tsunami Risk Category IV, Vertical Evacuation Refuges, and Tsunami Risk Category III Critical Facilities~~ | ~~1.25~~ |
| Tsunami Risk Category IV where the inundation depth is less than 12 ft and a site-specific Probabilistic Tsunami Hazard Analysis is not performed, per the exception to Section 6.5.2 | 1.5 |

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CODE CHANGE

§ 6.10.2.1

## Overall Drag Force on Buildings and Other Structures

| Width to Inundation Depth* Ratio, B/h | Drag Coefficient, $C_d$ |
|---|---|
| ≤12 | 1.25 |
| ~~16~~ | ~~1.3~~ |
| ~~26~~ | ~~1.4~~ |
| ~~36~~ | ~~1.5~~ |
| 60 | 1.75 |
| ~~100~~ | ~~1.8~~ |
| ≥120 | 2.0 |

* Inundation depth for each of the three Load Cases of inundation specified in Section 6.8.3.1. Interpolation shall be used for intermediate values of width to inundation depth ratio $B/h_{sx}$. Where building setbacks occur, drag coefficients shall be determined for each portion of a constant width. For each portion along the inundated height of the building, its equivalent inundated depth is taken as its submerged vertical dimension.

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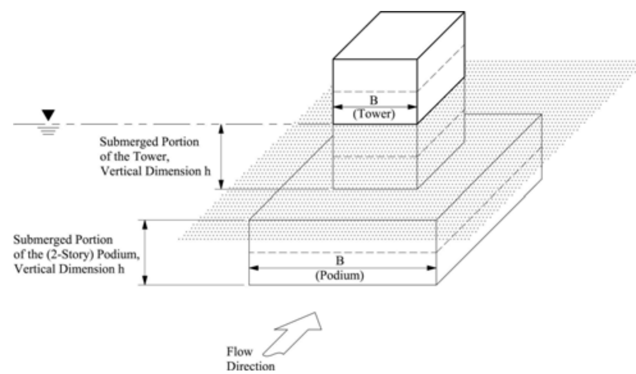
EXAMPLE

F C6.10-1

## Drag Force Coefficient Determination

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§ 6.10.2.2

### Drag Force on Components

The lateral hydrodynamic load given by Equation (6.10-4) shall be applied as a pressure resultant on the projected inundated height, $h_e$, of all structural components and exterior wall assemblies below the inundation depth:

$$F_d = ½ \rho_s I_{tsu} C_d b (h_e u^2) \qquad (6.10\text{-}4)$$

where for interior components, <u>except vertical load-bearing components in storage warehouses, and truck and bus garages,</u> the values of $C_d$ given in Table 6.10-2 shall be used, and b is the component width perpendicular to the flow. For exterior components, a $C_d$ value of 2.0 shall be used, and width dimension b shall be taken as the tributary width multiplied by the closure ratio value given in Section 6.8.7. <u>For interior vertical load-bearing components in storage warehouses, a $C_d$ value of 2.0 shall be used, and width dimension b shall be taken as 9 ft or the length of structural wall, whichever is greater. For interior vertical load-bearing components in truck and bus garages, a $C_d$ value of 2.0 shall be used, and width dimension b shall be taken as 40 ft, or the tributary width multiplied by the closure ratio value given in Section 6.8.7, whichever is smaller.</u>

- Observations indicate debris accumulation (damming) in warehouses and truck/bus garages lead to collapse

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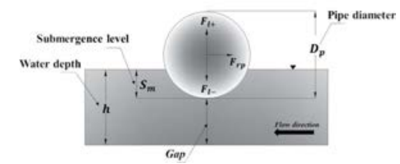
CODE CHANGE

§ 6.10.2.6.1

## Hydrodynamic Loads on Above-Ground Horizontal Pipelines

- <u>Table 6.10-3 Pipe Resistance Coefficient, $C_r$</u>
- <u>Figure 6.10-1 Pipe Resistance coefficient, $C_r$, as a Function of Froude number, $F_r$</u>
- <u>Table 6.10-4 Upward Lift Coefficient, $C_l^+$, for Pipelines</u>
- <u>Figure 6.10-2 Upward Lift Coefficient, $C_l^+$, as a Function of Froude number, $F_r$</u>
- <u>Table 6.10-5 Downward Lift Coefficient, $C_l^-$, for Pipelines</u>
- <u>Figure 6.10-3 Downward Lift coefficient, $C_l^-$, as a Function of Froude number, $F_r$</u>




*ASCE 7-22* Figure C6.10-4

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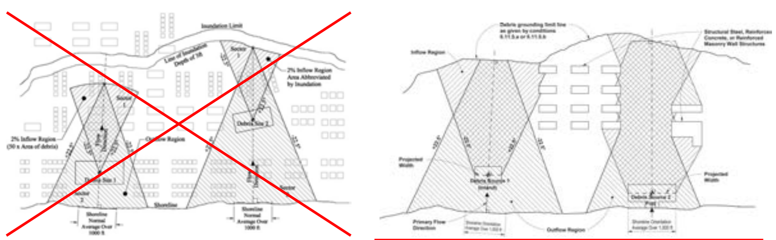
CODE CHANGE

§ 6.11.5.1

## Debris Impact Hazard Region



**Site Hazard Assessment for Shipping Containers, Ships, and Barges**

- Significant section rewrite
- Debris disbursed in geometrically defined area up to grounding limits
- Transport limited by resilient structures, geography and inundation depth

*ASCE 7-22* Figure 6.11-1

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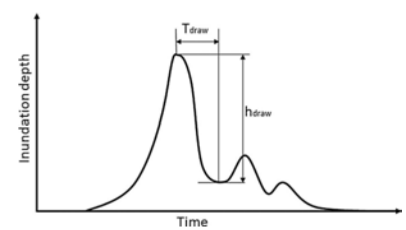
CODE CHANGE

§ 6.12.2.2

## Foundation Design – Loss of Soil Strength

- <u>Table 6.12-1 Drainage Factor, $K_d$, for Different Soil Types</u>
- <u>Figure 6.12-1 Site-specific tsunami inundation wave parameters used for pore pressure softening evaluation</u>
- <u>Figure 6.12-2 Susceptibility to pore pressure softening due to tsunami inundation</u>
- <u>Figure 6.12-3 Susceptible depth, $D_e$, affected by tsunami induced pore pressure softening</u>




*ASCE 7-22* Figure 6.12-1

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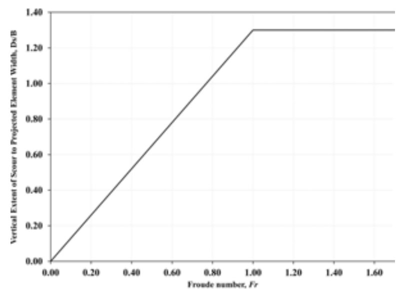
§ 6.12.2.4.1

## Sustained Flow Scour

- Revisions and new table and figure
  - Table 6.12-3 Vertical Extent of Pile Scour to Projected Element Width Ratio, $D_s/b$
  - Figure 6.12-5 Vertical extent of pile scour to projected element width ratio




*ASCE 7-22* Figure 6.12-5

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CODE CHANGE

§ 6.8.3.3

## Strength Design Tsunami Load Combinations

- $0.9D + F_{TSU} + H_{TSU}$ (6.8-1a)
- $1.2D + F_{TSU} + 0.5L + 0.2S + H_{TSU}$ (6.8-1b)

- $H_{TSU}$ = load caused by tsunami-induced lateral foundation pressures developed under submerged conditions
- $H_{TSU}$ load factor = 0.9 where net effect of $H_{TSU}$ counteracts principal load effect
- φ = 0.67 for stability, bearing capacity, lateral pressure, internal stability of geotextile and reinforced earth systems, slope stability including drawdown conditions, and uplift-resisting anchorage elements

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CODE CHANGE

§ 6.8.3.4

## LFRS Acceptance Criteria

- SDC A-C: LFRS designed for Max Considered Tsunami
- SDC D-F
  - LFRS adequate if $F_{TSU} < 0.75\ \Omega_0\ E_h$
  - $F_{TSU} > 0.75\ \Omega_0\ E_h$
    - LFRS below maximum inundation depth designed for a minimum seismic force $Q_E \geq F_{TSU}/(0.75\ \Omega_0)$, or
    - LFRS explicitly analyzed to validate resistance to Maximum Considered Tsunami

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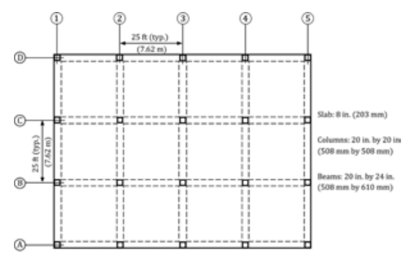
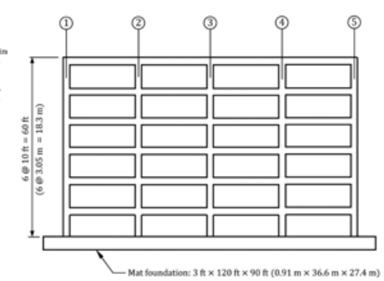
EXAMPLE

## Tsunami Loads on Essential Facility




- Monterey, CA
- Impact zone for shipping containers
- Cladding not designed for impact or blast
- Reinforced concrete

Example 7.5 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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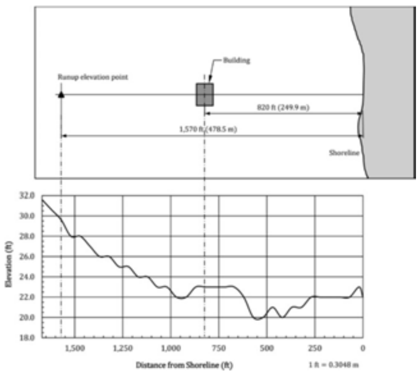
EXAMPLE

## Tsunami Loads on Essential Facility

- Runup elevation = 29.6 ft at 1,570 ft from shoreline
- RC = IV
- EGLA method permitted
- $R/H_T$ not required




Example 7.5 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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EXAMPLE

## Tsunami Loads on Essential Facility

Table 7.32 EGLA Calculations for the Building in Example 7.5

| Point | Distance from Shoreline, $x_i$ (ft) | Transect Elevation, $z_i$ (ft) | Topographic Slope, $\phi_i$ | Froude Number, $F_r$ | Friction Slope, $s_i$ | Topographic Slope, $\phi_{i,ave}$ | Energy Head, $E_g$ (ft) | Inundation Depth, $h_i$ (ft) | Flow Velocity, $u_i$ (ft/s) |
|---|---|---|---|---|---|---|---|---|---|
| Runup | 1,570.0 | 29.6 | | 0.000 | 0.000000 | | 0.00 | 0.10 | 0.00 |
| 1 | 1,520.0 | 28.0 | 0.0320 | 0.178 | 0.000896 | 0.0180 | 1.64 | 1.62 | 1.29 |
| 2 | 1,470.0 | 28.0 | 0.0000 | 0.252 | 0.000708 | 0.0180 | 1.68 | 1.63 | 1.83 |
| 3 | 1,420.0 | 27.0 | 0.0200 | 0.309 | 0.001060 | 0.0100 | 2.73 | 2.61 | 2.83 |
| 4 | 1,370.0 | 26.0 | 0.0200 | 0.357 | 0.001208 | 0.0150 | 3.79 | 3.57 | 3.82 |
| 5 | 1,320.0 | 26.0 | 0.0000 | 0.399 | 0.001360 | 0.0100 | 3.86 | 3.58 | 4.28 |
| 6 | 1,270.0 | 25.0 | 0.0200 | 0.437 | 0.001631 | 0.0100 | 4.94 | 4.51 | 5.27 |
| 7 | 1,220.0 | 25.0 | 0.0000 | 0.472 | 0.001761 | 0.0100 | 5.03 | 4.53 | 5.70 |
| 8 | 1,170.0 | 24.0 | 0.0200 | 0.505 | 0.002010 | 0.0100 | 6.13 | 5.44 | 6.68 |
| 9 | 1,120.0 | 24.0 | 0.0000 | 0.535 | 0.002127 | 0.0100 | 6.24 | 5.46 | 7.10 |
| 10 | 1,070.0 | 23.0 | 0.0200 | 0.564 | 0.002361 | 0.0100 | 7.36 | 6.35 | 8.07 |
| 11 | 1,020.0 | 23.0 | 0.0000 | 0.592 | 0.002470 | 0.0100 | 7.48 | 6.36 | 8.47 |
| 12 | 970.0 | 22.0 | 0.0200 | 0.618 | 0.002692 | 0.0000 | 8.61 | 7.23 | 9.43 |
| 13 | 920.0 | 22.0 | 0.0000 | 0.643 | 0.002795 | 0.0000 | 8.75 | 7.25 | 9.83 |
| 14 | 870.0 | 23.0 | −0.0200 | 0.668 | 0.003007 | −0.0050 | 7.90 | 6.46 | 12.19 |
| 15 | 820.0 | 23.0 | 0.0000 | 0.691 | 0.003348 | −0.0050 | 8.07 | 6.51 | 12.67 |
| 16 | 770.0 | 23.0 | 0.0000 | 0.714 | 0.003561 | 0.0000 | 8.25 | 6.57 | 10.39 |
| 17 | 720.0 | 23.0 | 0.0000 | 0.736 | 0.003772 | 0.0050 | 8.44 | 6.64 | 10.76 |
| 18 | 670.0 | 23.0 | 0.0000 | 0.757 | 0.003981 | 0.0150 | 8.64 | 6.71 | 11.13 |
| 19 | 620.0 | 22.0 | 0.0200 | 0.778 | 0.004187 | 0.0150 | 9.85 | 7.56 | 12.14 |

Example 7.5 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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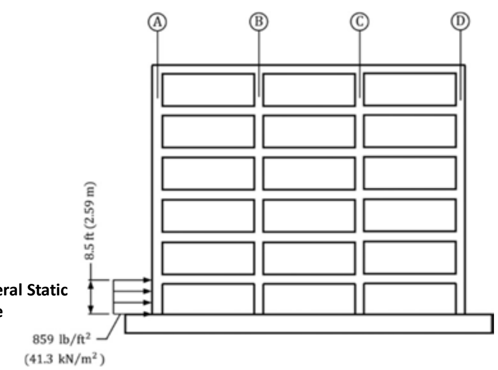
EXAMPLE

## Tsunami Loads on Essential Facility

Simplified Uniform Lateral Static Hydrodynamic Pressure




Example 7.5 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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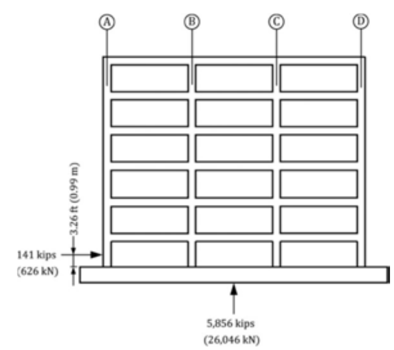
EXAMPLE

## Tsunami Loads on Essential Facility

- Load Case 1 = 141 kips
- Load Case 2 = 112 kips
- Load Case 3 = 99 kips




Example 7.5 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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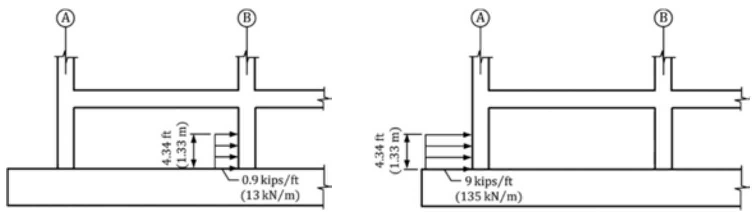
## Tsunami Loads on Essential Facility

EXAMPLE

Drag Forces

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Example 7.5 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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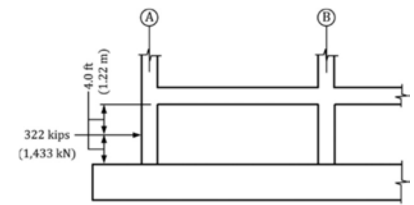
## Tsunami Loads on Essential Facility

EXAMPLE

- Debris Impact – Alternative Simplified Static Load

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Example 7.5 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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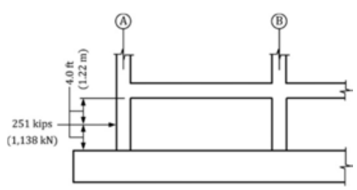
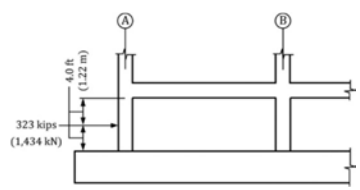
## Tsunami Loads on Essential Facility

EXAMPLE




**Dynamic Impact Force Due to Wood Logs and Poles**




**Dynamic Impact Force Due to Shipping Containers**

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Example 7.5 from *Structural Load Determination: 2024 IBC and ASCE/SEI 7-22* – McGraw Hill

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## ASCE 7-22 Tsunami Loads Criteria Change Summary

1. Provisions for hydrodynamic load on pipes
2. Debris damming for warehouses and parking garages
3. Improved provisions for scour and pore pressure softening around foundations
4. Building drag coefficient simplified
5. Debris impact zone extended to grounding limit or resilient structures

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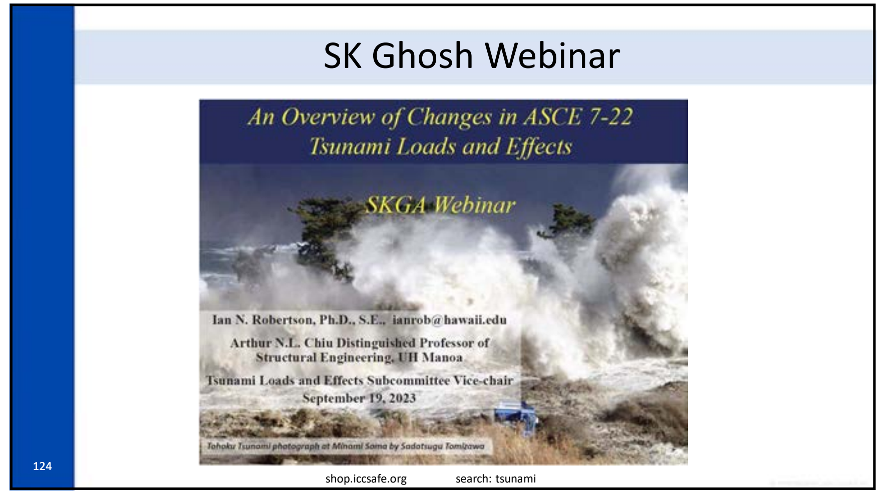


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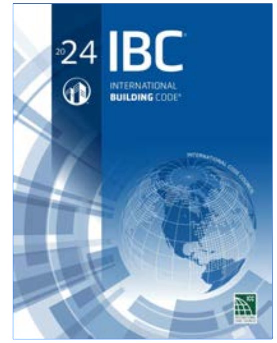
COURSE OUTLINE

- Temporary Structures
  - Definitions
  - Environmental Loads
  - Inspections
  - Durability and Maintenance
  - Controlled Occupancy Procedures
  - Examples

4

4

## Codes and Standards

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## Temporary Structures

Any building or structure erected for ≤180 days to support temporary events

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## Summary of Temporary Structure Changes

- New definitions for temporary structures, events and service life
- Reduced environmental loads based on risk category, service life and controlled occupancy procedures
- Requirements for installation and maintenance inspections
- Controlled occupancy procedures for public-occupancy temporary structures
- Non-building structure risk categories
  - Correspond to the public assembly
  - Factored assembly area establishes the occupant load

§ 3103

12

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CODE CHANGE

## Public-occupancy Temporary Structure

Any building or structure erected for a period of 1 year or less that serves an assembly occupancy or other public use.

- Period of <u><</u>1 year
- Expectation of similar reliability and safety as permanent structures

§ 202

13

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CODE CHANGE

## Public-occupancy Temporary Structure

The period of time that a structure serves its intended purpose. For temporary structures, this shall be the cumulative time of service for sequential temporary events which may occur in multiple locations. For public-occupancy temporary structures this is assumed to be a minimum of 10 years.

- Period of 10 years
- Cumulative time
- May occur in different locations

§ 202

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CODE CHANGE

## Temporary Event

A single use during the service life of a public-occupancy temporary structure at a given location that includes its installation, inspection, use and occupancy, and dismantling.

- Single use during service life
- Includes installation, inspection, use and occupancy, and dismantling

§ 202

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§ 202

## Temporary Structure

Any building or structure erected for a period of 180 days or less to support temporary events. Temporary structures include a range of structure types (public-occupancy temporary structures, temporary special event structures, tents, umbrellas and other membrane structures, relocatable buildings, temporary bleachers, etc.) for a range of purposes (storage, equipment protection, dining, workspace, assembly, etc.

- Period of ≤180 days
- Range of structures
- Range of purposes

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CODE CHANGE

§ 3103.1

## Temporary Structures – General

The provisions of Sections 3103.1 through ~~3103.4~~ 3103.8 shall apply to structures erected for a period of less than 180 days. Temporary special event structures, tents, umbrella structures and other membrane structures erected for a period of less than 180 days shall also comply with the *International Fire Code*. ~~Those~~ Temporary structures erected for a longer period of time and public-occupancy temporary structures shall comply with applicable sections of this code.

**Exceptions:**

1. Public-occupancy temporary structures complying with Section 3103.1.1 shall be permitted to remain in service for 180 days or more but not more than 1 year where approved by the building official.
2. Public-occupancy temporary structures within the confines of an existing structure are not required to comply with Section 3103.6.

- Public-occupancy temporary structures
  - In service >1 year must meet IBC requirements for new buildings
  - Within existing structures exempt from environmental load provisions

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CODE CHANGE

§ 3103.1

## Temporary Structures – General

Public-occupancy temporary structures shall be permitted to remain in service for 180 days or more without complying with requirements in this code for new buildings or structures where extensions for up to 1 year are granted by the building official in accordance with Section 108.1 and where the following conditions are satisfied:

- Additional inspections during installation
- Follow-up inspections after occupancy
- Design for environmental loads by RDP
- Relocation requires new permit
- Use or occupancy unchanged
- Extensions only as approved

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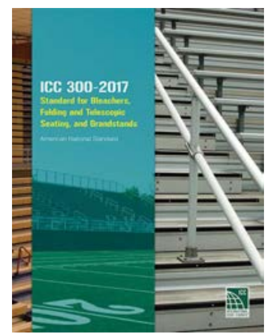
CODE CHANGE

§ 3103.5

## Bleachers

- Temporary bleachers, grandstands and folding and telescopic seating, that are not building elements, shall comply with ICC 300.

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§ 3103.6

## Structural Requirements

Temporary structures shall comply with the structural requirements of this code. Public-occupancy temporary structures shall be designed and erected to comply with the structural requirements of this code and Sections 3103.6.1 through 3103.6.4.

**Exception**: Where approved, live loads less than those prescribed by Table 1607.1 shall be permitted provided that a registered design professional demonstrates that a rational approach has been used and that such reductions are warranted.

- RDP may design for reduced live loads

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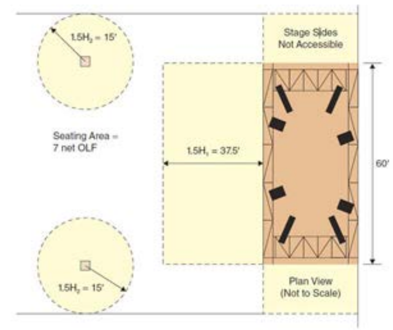
CODE CHANGE

§ 3103.6

## Structural Requirements (continued)

- Temporary non-building structures ancillary to public assemblies or special event structures whose structural failure or collapse would endanger assembled public shall be assigned a risk category corresponding to the risk category of the public assembly. For the purposes of establishing an occupant load for the assembled public endangered by structural failure or collapse, the applicable occupant load determination in Section 1004.5 or 1004.6 shall be applied over the assembly area within a radius equal to 1.5 times the height of the temporary non-building structure.

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CODE CHANGE

Chapter 16

## Environmental Loads for Temporary Structures

| Environmental Load | Chapter 16 Reference | Chapter 31 Reference |
|---|---|---|
| Snow | 1608.1 | 3103.6.1.1 |
| Wind | 1609.1.1 | 3103.6.1.2 |
| Flood | 1612.3 | 3103.6.1.3 |
| Seismic | 1613.1 | 3103.6.1.4 |
| Ice | 1614.1 | 3103.6.1.5 |
| Tsunami | 1615.1 | 3103.6.1.6 |

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CODE CHANGE

§ 3103.6.1.1

## Temporary Structures – Snow Loads

Snow loads on public-occupancy temporary structures shall be determined in accordance with Section 1608. The ground snow loads, $p_g$, in Section 1608 shall be modified according to Table 3103.6.1.1.

**Exception:** Ground snow loads, $p_g$, for public-occupancy temporary structures that employ controlled-occupancy procedures per Section 3103.8 shall be permitted to be modified using a ground snow load reduction factor of 0.65 instead of the ground snow load reduction factors in Table 3103.6.1.1.

{additional provisions not shown for brevity}

- Reduced ground snow load based on
  - Risk category and service life
  - Controlled-occupancy procedures

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§ 3103.6.1.1

## Snow Loads – Temporary Structures

| Reduction Factors for Ground Snow Loads for Public-occupancy Temporary Structures | | |
|---|---|---|
| Risk Category | Service Life | |
| | ≤ 10 yr | >10 yr |
| II | 0.7 | 1.0 |
| III | 0.8 | 1.0 |
| IV | 1.0 | 1.0 |

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CODE CHANGE

§ 3103.6.1.2

## Temporary Structures – Wind Loads

The design wind load on public-occupancy temporary structures shall be permitted to be modified in accordance with the wind load reduction factors in Table 3103.6.1.2.

**Exceptions:**

1. Design wind loads for public-occupancy temporary structures that implement controlled occupancy procedures per Section 3103.8 shall be permitted to be modified using a wind load reduction factor of 0.65.
2. For public-occupancy temporary structures erected in a hurricane-prone region outside of hurricane season, the basic wind speed, V, shall be permitted to be set as follows, depending on Risk Category:
   2.1 For Risk Category II: 115 mph.
   2.2 For Risk Category III: 120 mph.
   2.3 For Risk Category IV: 125 mph.

- Reduced wind load based on
  - Risk category and service life
  - Controlled-occupancy procedures
  - Location in hurricane-prone regions

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CODE CHANGE

§ 3103.6.1.2

## Temporary Structures – Wind Loads

| Table 3103.6.1.2 Reduction Factors for Wind Loads for Public-occupancy Temporary Structures | | |
|---|---|---|
| Risk Category | Service Life | |
| | ≤ 10 yr | >10 yr |
| II | 0.8 | 1.0 |
| III | 0.9 | 1.0 |
| IV | 1.0 | 1.0 |

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CODE CHANGE

§3103.6.1.3

## Temporary Structures – Flood Loads

Public-occupancy temporary structures need not be designed for flood loads specified in Section 1612. Controlled occupancy procedures in accordance with Section 3103.8 shall be implemented.

- Flood load design not required
- Controlled occupancy procedures

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§3103.6.1.4

## Temporary Structures – Seismic Loads

Seismic loads on public-occupancy temporary structures assigned to Seismic Design Categories C through F shall be permitted to be taken as 75% of those determined by Section 1613. Public-occupancy temporary structures assigned to Seismic Design Categories A and B need not be designed for seismic loads.

- SDC C through F
- 75% of seismic design loads for permanent structures
- SDC A and B – seismic design not required

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CODE CHANGE

§ 3103.6.1.5

## Temporary Structures – Ice Loads

Ice loads on public-occupancy temporary structures shall be permitted to be determined with the largest maximum nominal thickness being 0.5 inches, for all Risk Categories. Where the public-occupancy temporary structure is not subject to ice loads or not constructed and occupied during times when ice is to be expected, ice loads need not be considered, provided that where the period of time when the public-occupancy temporary structure is in service shifts to include times when ice is to be expected, either of the following conditions is met:

1. The design is reviewed and modified, as appropriate, to account for ice loads
2. Controlled occupancy procedures in accordance with Section 3103.8 are implemented.

- Prescriptive ½" nominal ice thickness for all risk categories
- Need not be considered for temporary structures
  - Not subject to ice loads
  - Not occupied during icing periods
  - Designed for ice loads
  - Where controlled occupancy procedures implemented

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CODE CHANGE

§3103.6.1.6

## Temporary Structures – Tsunami Loads

Public-occupancy temporary structures in a tsunami design zone are not required to be designed for tsunami loads specified in Section 1615. Controlled occupancy procedures in accordance with Section 3103.8 shall be implemented.

- Tsunami load design not required
- Controlled occupancy procedures

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CODE CHANGE

§3103.6.2

## Temporary Structures – Foundations

Public-occupancy temporary structures shall be permitted to be supported on the ground with temporary foundations where approved by the building official. Consideration shall be given for the impacts of differential settlement where foundations do not extend below the ground or where foundations are supported on compressible materials. The presumptive load-bearing value for public-occupancy temporary structures supported on a pavement, slab on grade or on other collapsible or controlled low-strength substrate soils such as beach sand or grass shall be assumed not to exceed 1,000 pounds per square foot unless determined through testing and evaluation by a registered design professional. The presumptive load-bearing values listed in Table 1806.2 shall be permitted to be used for other supporting soil conditions.

- Temporary foundations if approved
  - Differential settlement
- Soil bearing capacity $\leq$1,000 psf
  - Pavement
  - Slab on grade
  - Sand
  - Grass

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CODE CHANGE

§3103.6.3

24 IBC

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## Temp Structures – Installation & Maintenance Inspections

A qualified person shall inspect public-occupancy temporary structures that are assembled using transportable and reusable materials. Components shall be inspected when purchased or acquired and at least once per year. The inspection shall evaluate individual components, and the fully assembled structure, to determine suitability for use based on the requirements in ESTA ANSI E1.21. Inspection records shall be kept and shall be made available for verification by the building official. Additionally, public-occupancy temporary structures shall be inspected at regular intervals when in service to ensure that the structure continues to perform as designed and initially erected.

- Qualified person to inspect public-occupancy temporary structures
  - Components inspected once per year minimum
  - ESTA ANSI E1.21 standard
  - Inspection records available
- Public-occupancy temporary structures also inspected in service

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CODE CHANGE

§3103.6.4

24 IBC

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## Temporary Structures – Durability

Reusable components used in the erection and the installation of public-occupancy temporary structures shall be manufactured of durable materials necessary to withstand environmental conditions at the service location. Components damaged during transportation or installation or due to the effects of weathering shall be replaced or repaired.

- Reusable components made of durable materials
- Damaged components repaired or replaced

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CODE CHANGE

§3103.8

24 IBC

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## Temporary Structures – Controlled Occupancy

Where controlled occupancy procedures are required to be implemented for public occupancy temporary structures in Section 3103.6.1, the procedures shall comply with this section and ANSI ES1.7. An operations management plan in accordance with ANSI E1.21 shall be submitted to the building official for approval as a part of the permit documents. In addition, the operations management plan shall include an emergency action plan that documents the following information, where applicable:

- Monitor and remove excess snow or ice – vacate if loads are exceeded
- Monitor wind speeds – vacate if loads are exceeded
- Evacuation procedures for flood and tsunami events
- Procedures for each environmental hazard
- Anchoring or removal of structure to mitigate hazards

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EXAMPLE

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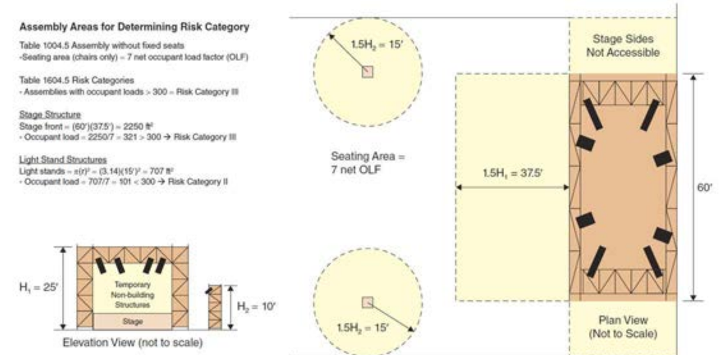
## Risk Category Determination

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EXAMPLE

## Risk Category Assignment to Poles

**Examples of Minimum Areas and Heights for Assigning Pole-Type Temporary Non-Building Structures as Risk Category III.**

| Assemblies without fixed seats | Occupant Load Factor (Table 1004.5) | Risk Category III* | |
|---|---|---|---|
| | | Area** ($ft^2$) | Pole Height (ft) |
| Standing space | 5 | >1500 | >14.57 |
| Concentrated (chairs only—not fixed) | 7 | >2100 | >17.24 |
| Unconcentrated (tables and chairs) | 15 | >4500 | >25.24 |

*Assemblies with occupant loads >300 are classified as Risk Category III (Table 1604.5).
**Assumes the full circle around the pole overlaps the assembly area.

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APPLICATION

**4. Reduced environmental loads can be based which of the following?**

a) Risk category
b) Service life
c) Controlled occupancy procedures
d) All of the above

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Temporary Structure Loads Criteria Change Summary

1. New definitions for temporary structures, events and service life
2. Reduced environmental loads based on risk category, service life and controlled occupancy procedures
3. Requirements for installation and maintenance inspections
4. Controlled occupancy procedures for public-occupancy temporary structures
5. Non-building structure risk categories correspond to the public assembly and factored assembly area sets the occupant load

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