Reliability of Concrete Structures

- Loads, Load Factors, and Load Combinations

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Motivations

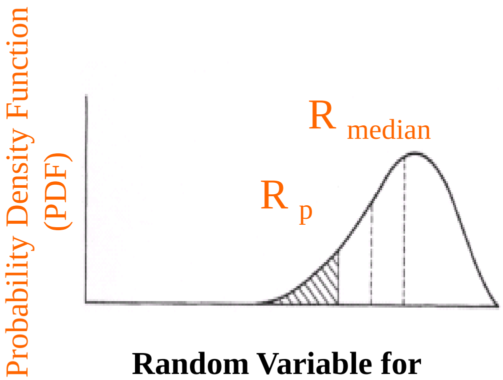
- **ACI 348 Structural Reliability and Safety** committee is currently developing the "Guide to Reliability Basics of Concrete Structures".
- ACI 348 will provide the support to ACI 562 Standard "Assessment, Repair, and Rehabilitation of Existing Concrete Structures – Code and Commentary", in addition to ACI 318 Standard "Building Code Requirements for Structural Concrete and Commentary"
  (*i.e.* assessment of existing structures vs. design of new structures)
- ACI 348 will focus on the effects of **climate change** on structural safety and resilience of structures and infrastructure systems, where non-stationary design loads caused by extreme climate and weather events shall be developed and the corresponding vulnerabilities and risks shall be considered in the initial planning phase of infrastructure systems.

Table of Contents

- Introduction (structural safety margin)
- **ACI 318 / 562** loads, load factors, and load combinations
- Loads in IBC and ASCE/SEI 7
- Modeling design loads: dead, live, snow (primarily, gravity loads)

  wind, earthquake, fluid or soil lateral pressure, flood, and ice (primarily, lateral loads)
- Principles in load combinations
- Calibration of design codes for buildings (ACI 318)
- Summary (future research needs)

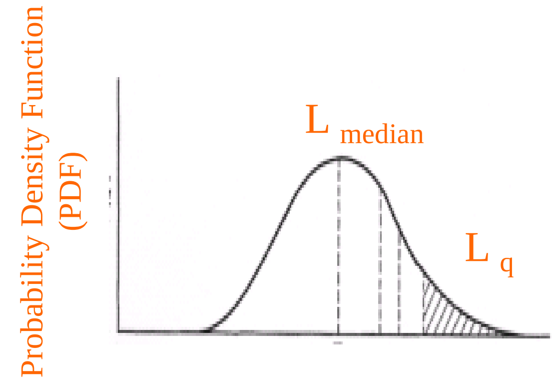
Introduction (structural safety margin, M)



Random Variable for Structural Capacity (R)

$$M = R - L \geq 0$$

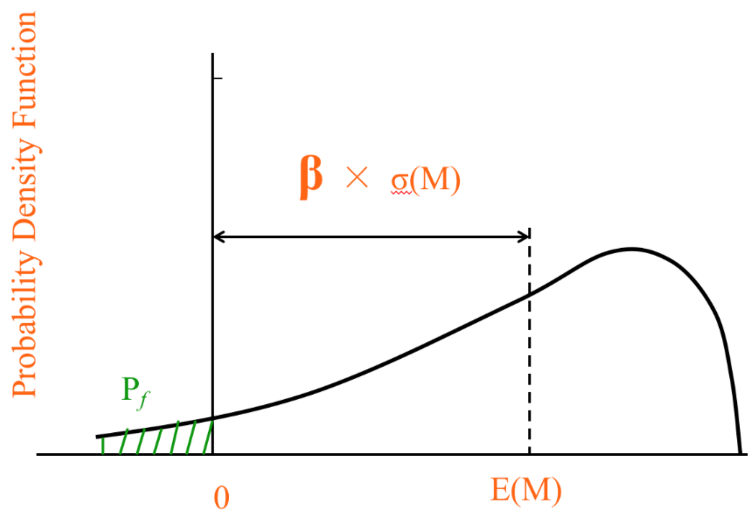
Resistance (Capacity) Load Effect (Demand)



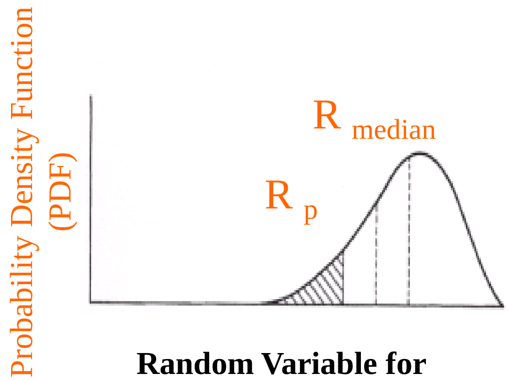
Random Variable for Load Effects (L)

$$\beta = \frac{E(M)}{\sigma(M)}$$

β ⟹	$P_f = \Phi(-\beta) = 1 - \Phi(\beta)$
-2	0.977250
0	0.500000
2	0.022750
4	0.316712E-04
6	0.986588E-09



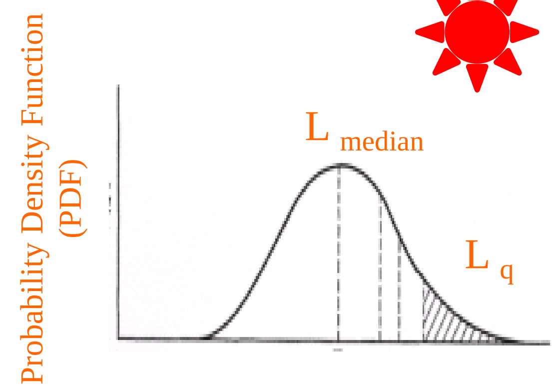
Introduction (structural safety margin, M) (cont.)



Random Variable for Structural Capacity (R)

$$M = R - L \geq 0$$

Resistance (Capacity) Load Effect (Demand)



Random Variable for Load Effects (L)

$$\beta = \frac{E(M)}{\sigma(M)} = \frac{\bar{R} - \bar{L}}{\sqrt{\sigma_R^2 + \sigma_L^2}}$$

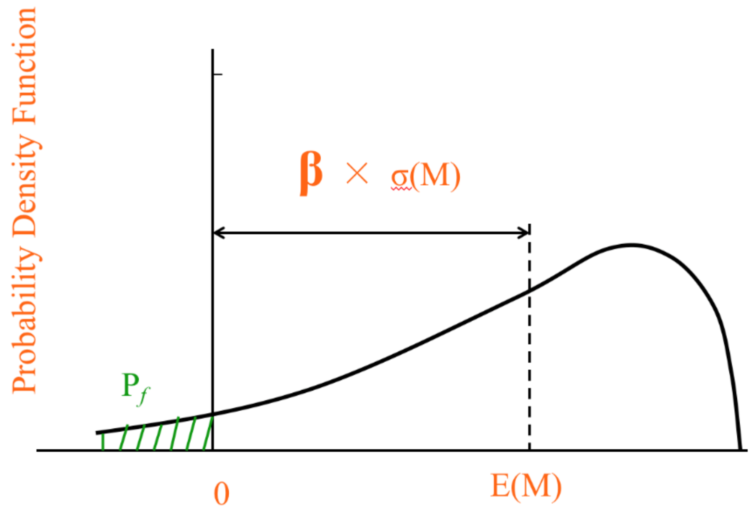
$$Factor\ of\ Safety(F.S.) = \frac{R}{L} \geq 1.0$$

$$F.S. = \frac{\bar{R}}{\bar{L}}$$

loss information carried by σ_R *and* σ_L

$$F.S. = \frac{R_p}{L_q}$$

Implicit use of information carried by σ_R *and* σ_L



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ACI 318-14 Loads, Load Factors, and Load Combinations

- Section 4.3 Design loads → Chapter 5; (R4.3 mentions ASCE/SEI 7)
- Section 5.2.2 **Loads** → **General Building Code** (e.g. IBC, IRC & NFPA 5000)
- **Table 5.3.1 – Load combinations**

Load combination	Equation	Primary load
U = 1.4D	(5.3.1a)	D (Dead)
U = 1.2D + 1.6L + 0.5(L_r or S or R)	(5.3.1b)	L (Live)
U = 1.2D + 1.6(L_r or S or R) + (L or 0.5W)	(5.3.1c)	L_r (Roof) or S (Snow) or R (Rain)
U = 1.2D + W + L + 0.5(L_r or S or R)	(5.3.1d)	W (Wind)
U = 1.2D + E + L + 0.2S	(5.3.1e)	E (Earthquake)
U = 0.9D + (W or E)	(5.3.1f and g)	W and E

- Sections 5.3.7 to 5.3.11 → load factors for fluid (F), lateral soil pressure (H), flood (ASCE/SEI 7), ice (ASCE/SEI 7), and prestressing.

ACI 562-21 Loads, Load Factors, and Load Combinations

- Section 1.7.3 Loads → **Existing Building Code** (*e.g.* **IEBC**)

  Load combinations in Chapter 5;

- Section 1.8.5 Loads → **Design-Basis Code** (see Chapter 4)

  Current or Original Building Code

  (*e.g.* IBC) **(*e.g.* UBC)**

  Load combinations in Chapter 5;

- Section 5.2.3 **Load combinations** → **Design-Basis Code** (*e.g.* IEBC, IBC, or UBC)

- Sections 5.2.4, 5.2.5 and 5.5 provide additional load combinations.

Dead Loads in IBC and ASCE/SEI 7

IBC - 2018

Dead (Section 1606)

- actual weights;
- estimated weights approved by the building official.
- **nominal dead loads, D**

Tables C3-1 and C3-2 of the commentary to **ASCE/SEI 7**

Material	Unit Load (average)
Concrete	150 pcf
Steel	490 pcf
Aluminum	170 pcf
Soil	120 pcf
4” thick Brick	40 psf
8” thick CMU	55 psf

ASCE/SEI 7-16

Dead (Chapter 3)

- maximum weight of the contents;
- vegetative and landscaped roofs;
- PV panel systems

Table C3-1 Minimum Design Dead Load

Table C3-2 Minimum Densities for Design Loads from Materials

Live Loads in IBC and ASCE/SEI 7

IBC - 2018

Live (Section 1607)

(1) 1607.3 Uniform live loads
(2) 1607.4 Concentrated loads
(3) 1607.5 Partition loads
(4) 1607.6 Helipads
(5) 1607.7 Heavy vehicle loads
(6) 1607.8 Handrail and guard loads
(7) 1607.9 Vehicle barriers
(8) 1607.10 Impact loads
(9) 1607.11 Reduction in uniform live loads. Eq. (16-23)
(10) 1607.12 Distribution of floor loads
(11) 1607.13 Roof loads
(12) 1607.14 Crane loads
(13) 1607.15 Interior walls and partitions

ASCE/SEI 7-16

Live (Chapter 4)

(1) 4.3 Uniform live loads
(2) 4.4 Concentrated loads
(3) 4.5 Handrail and guard loads
(4) 4.6 Impact loads
(5) 4.7 Reduction in uniform live loads
(6) 4.8 Reduction in roof live loads
(7) 4.9 Crane loads
(8) 4.10 Garage loads
(9) 4.11 Helipad loads
(10) 4.12 Uninhabitable attics
(11) 4.13 Library stack rooms
(12) 4.14 Seating for assembly uses
(13) 4.15 Sidewalks, driveways
(14) 4.16 Stair treads
(15) 4.17 PV system

Live Loads in IBC and ASCE/SEI 7 (cont.)

Provisions	7-05	7-10	7-16	IBC 09	IBC 12	IBC 15	IBC 18
Uniformly	4.2.1	4.3.1	4.3.1	1607.3	1607.3	1607.3	1607.3
Partitions	4.2.2	4.3.2	4.3.2	1607.5/13	1607.5/14	1607.5/14	1607.5 /15
Concentrated	4.3	4.4	4.4	1607.4	1607.4	1607.4	1607.4
Handrail	4.4	4.5	4.5	1607.7	1607.8	1607.8	1607.8/9
Unspecified	4.5	4.2	4.2	N/A	N/A	1607.2	1607.2
Partial	4.6	4.3.3	4.3.3	1607.10	1607.11	1607.11	1607.12
Impact	4.7	4.6	4.6	1607.8	1607.9	1607.9	1607.10
Reduction	4.8	4.7	4.7	1607.9	1607.10	1607.10	1607.11
Roof	4.9	4.8	4.8	1607.11	1607.12	1607.12	1607.13
Crane	4.10	4.9	4.9	1607.12	1607.13	1607.13	1607.14
Garage, Helipad, PV	N/A	Table 4-1	4.10 -17	1607.6	1607.6/7	1607.6/7 /12	1607.6/7 /13

Table 1607.1.in IBC 2018 and Table 4-1 in ASCE/SEI 7 -16

TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS[g]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
1. Apartments (see residential)	—	—
2. Access floor systems		
Office use	50	2,000
Computer use	100	2,000
3. Armories and drill rooms	150[n]	—
4. Assembly areas		—
Fixed seats (fastened to floor)	60[m]	
Follow spot, projections and control rooms	50	
Lobbies	100[m]	
Movable seats	100[m]	
Stage floors	150[n]	
Platforms (assembly)	100[m]	
Other assembly areas	100[m]	
5. Balconies and decks[h]	1.5 times the live load for the area served, not required to exceed 100	—
6. Catwalks	40	300
7. Cornices	60	—
8. Corridors		—
First floor	100	
Other floors	Same as occupancy served except as indicated	
9. Dining rooms and restaurants	100[m]	—
10. Dwellings (see residential)	—	—
11. Elevator machine room and controlroom grating (on area of 2 inches by 2 inches)	—	300
12. Finish light floor plate construction (on area of 1 inch by 1 inch)	—	200
13. Fire escapes	100	—
On single-family dwellings only	40	
14. Garages (passenger vehicles only)	40[n]	Note a
Trucks and buses	See Section 1607.7	
15. Handrails, guards and grab bars	See Section 1607.8	
16. Helipads	See Section 1607.6	
17. Hospitals		
Corridors above first floor	80	1,000
Operating rooms, laboratories	60	1,000
Patient rooms	40	1,000
18. Hotels (see residential)	—	—
19. Libraries		
Corridors above first floor	80	1,000
Reading rooms	60	1,000
Stack rooms	150[b, n]	1,000
20. Manufacturing		
Heavy	250[n]	3,000
Light	125[n]	2,000
21. Marquees, except one- and two-family dwellings	75	—
22. Office buildings		
Corridors above first floor	80	2,000
File and computer rooms shall be designed for heavier loads based on anticipated occupancy	—	—
Lobbies and first-floor corridors	100	2,000
Offices	50	2,000

(continued)

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TABLE 1607.1—continued
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS[g]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
23. Penal institutions		—
Cell blocks	40	
Corridors	100	
24. Recreational uses:		—
Bowling alleys, poolrooms and similar uses	75[m]	
Dance halls and ballrooms	100[m]	
Gymnasiums	100[m]	
Ice skating rink	250[n]	
Reviewing stands, grandstands and bleachers	100[c, m]	
Roller skating rink	100[m]	
Stadiums and arenas with fixed seats (fastened to floor)	60[c, m]	
25. Residential		—
One- and two-family dwellings		
Uninhabitable attics without storagei	10	
Uninhabitable attics with storage[i, j, k]	20	
Habitable attics and sleeping areas[k]	30	
Canopies, including marquees	20	
All other areas	40	
Hotels and multifamily dwellings		
Private rooms and corridors serving them	40	
Public roomsm and corridors serving them	100	
26. Roofs		
All roof surfaces subject to maintenance workers		300
Awnings and canopies:		
Fabric construction supported by a skeleton structure	5[m]	
All other construction, except one- and two-family dwellings	20	
Ordinary flat, pitched, and curved roofs (that are not occupiable)	20	
Primary roof members exposed to a work floor		
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages		2,000
All other primary roof members		300
Occupiable roofs:		
Roof gardens	100	
Assembly areas	100[m]	
All other similar areas	Note 1	Note 1
27. Schools		
Classrooms	40	1,000
Corridors above first floor	80	1,000
First-floor corridors	100	1,000
28. Scuttles, skylight ribs and accessible ceilings	—	200
29. Sidewalks, vehicular driveways and yards, subject to trucking	250[d, n]	8,000[e]
30. Stairs and exits		
One- and two-family dwellings	40	300[f]
All other	100	300[f]

(continued)

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Table 4-1 Minimum Uniformly Distributed Live Loads, L_o, and Minimum Concentrated Live Loads

Occupancy or Use	Uniform psf (kN/m²)	Conc. lb (kN)
Apartments (see Residential)		
Access floor systems		
Office use	50 (2.4)	2,000 (8.9)
Computer use	100 (4.79)	2,000 (8.9)
Armories and drill rooms	150 (7.18)[a]	
Assembly areas and theaters		
Fixed seats (fastened to floor)	60 (2.87)[a]	
Lobbies	100 (4.79)[a]	
Movable seats	100 (4.79)[a]	
Platforms (assembly)	100 (4.79)[a]	
Stage floors	150 (7.18)[a]	
Balconies and decks	1.5 times the live load for the occupancy served. Not required to exceed 100 psf (4.79 kN/m²)	
Catwalks for maintenance access	40 (1.92)	300 (1.33)
Corridors		
First floor	100 (4.79)	
Other floors, same as occupancy served except as indicated		
Dining rooms and restaurants	100 (4.79)[a]	
Dwellings (see Residential)		
Elevator machine room grating (on area of 2 in. by 2 in. (50 mm by 50 mm))		300 (1.33)
Finish light floor plate construction (on area of 1 in. by 1 in. (25 mm by 25 mm))		200 (0.89)
Fire escapes	100 (4.79)	
On single-family dwellings only	40 (1.92)	
Fixed ladders	See Section 4.5	
Garages		
Passenger vehicles only	40 (1.92)[a,b,c]	
Trucks and buses	[c]	
Handrails, guardrails, and grab bars	See Section 4.5	
Helipads	60 (2.87)[d,e] Nonreducible	[e,f,g]
Hospitals		
Operating rooms, laboratories	60 (2.87)	1,000 (4.45)
Patient rooms	40 (1.92)	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
Hotels (see Residential)		
Libraries		
Reading rooms	60 (2.87)	1,000 (4.45)
Stack rooms	150 (7.18)[a,h]	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
Manufacturing		
Light	125 (6.00)[a]	2,000 (8.90)
Heavy	250 (11.97)[a]	3,000 (13.40)

Continued

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IBC - 2018

Snow (Section 1608)

- **Chapter 7, ASCE/SEI 7**;
- ground snow load (p_g)

  Figure 1608.2; (7.2-1)

  Table 1608.2; (7.2-1) for Alaska

Location	P_g (psf)	Location	P_g (psf)
Anchorage	50	Talkeetna	120
Fairbanks	60	Petersburg	150
Cordova	100	Whittier	300

- 2% annual probability of being exceeded (50-year return period)

ASCE/SEI 7-16

Snow (Chapter 7)

- 40 + years of ground snow load data;
- Exposure factor (C_e)

  Section 7.3.1 (0.7 ~ 1.2)
- Thermal factor (C_t)

  Section 7.3.2 (0.85, 1.1, 1.2, 1.3)
- Importance factor (I)

  Section 7.3.3 (Table 1.5 -2)
- Balanced with slope factor (C_s)

  Section 7.4 ($\leq$ 1.0)
- Partial: Section 7.5
- Unbalanced: Section 7.6
- Drifting

  Sections 7.7 and 7.8

Wind Loads in IBC and ASCE/SEI 7

IBC - 2018

Wind (Section 1609)

- **Chapters 26 to 31, ASCE/SEI 7**;
- Basic design wind speed (V)
  (3-second gust speed at 33 ft. (10 m) above the ground in Exposure C)
- Allowable stress design wind speed (V_{asd})
- Exposure category
- Any horizontal direction
- Normal pressure to the surface
- ICC 600, AWC WFCM, AISI S230 (V)
- NAAMM FP 1001 and TIA-222 (V_{asd})
- Wind tunnel tests
  ASCE 49
  Sections 31.4 and 31.5 of ASCE/SEI 7

ASCE/SEI 7-16

Wind (Chapters 26 to 31)

- Directional procedure
  Chapter 27
- Envelope procedure (low-rise)
  Chapter 28
- Directional procedure
  Chapter 29 (e.g. posts)
- Component and cladding
  Chapter 30
- Wind tunnel tests
  Chapter 31
- Minimum design wind load
  16 psf or 8 psf

Modeling Dead Loads

- Mean D = (bias coef.) (nominal D)

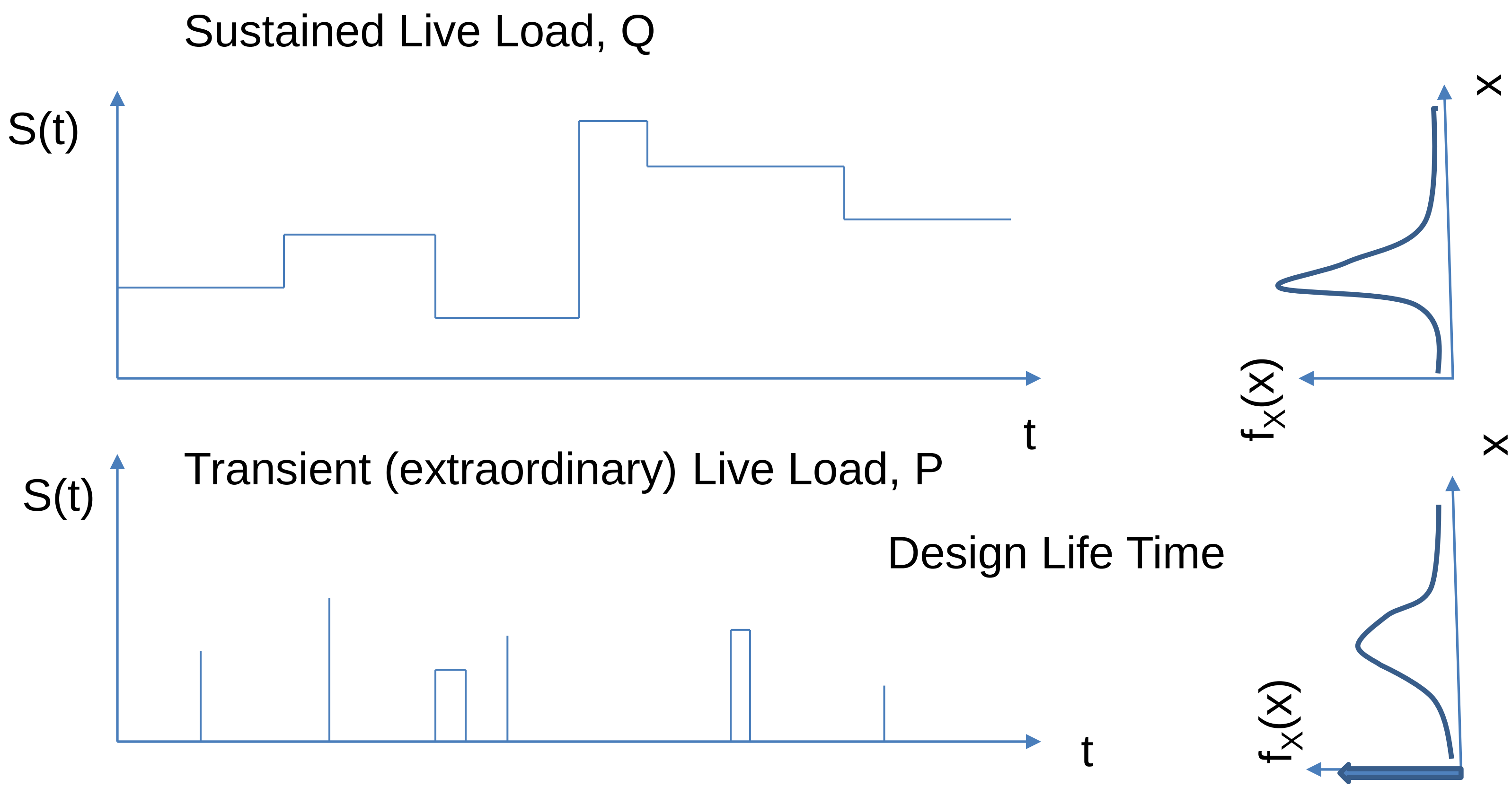
(1.03 ~ 1.05) (Tables C3-1 and C3-2 of **ASCE/SEI 7**)

<table>
<tr><th>Construction Material</th><th>Bias coef.</th><th>COV**</th><th>Probability Distribution</th></tr>
<tr><td>Concrete</td><td rowspan="4">Biased about 5%*,**,***

3% for precast concrete***</td><td>0.04</td><td rowspan="4">Log-normal(*,**) and normal**</td></tr>
<tr><td>Steel</td><td>0.01</td></tr>
<tr><td>Timber
(i) Sawn beam or strut
(ii) Laminated beam</td><td>0.12
0.10</td></tr>
<tr><td>Overall</td><td>0.07*
0.08 ~ 0.10***</td></tr>
</table>

* Refer to “Structural Reliability Handbook” (2015, Australian Building Codes Board)
** Refer to “Risk and Safety in Civil, Surveying and Environmental Engineering” (by Prof. Michael Faber, Swiss Federal Institute of Technology, ETH Zurich)
*** Refer to “Calibration of Design Code for Buildings (ACI318): Part 2” (2003) (by Prof. Andrzej Nowak, University of Michigan, currently, Auburn University)

Modeling Live Loads



from Y. K. Wen (1990)

A live load can be categorized as sustained or transient (extraordinary) live load, in addition to uniformly distributed or concentrated live load.

Modeling Sustained Live Loads (Q)

Q on office floors (Ellingwood 1977, Ref. 4 in ASCE/SEI 7)

Mean (Q) = 11.6 psf.

Var (Q) = 26.2 + 14,300 / Area

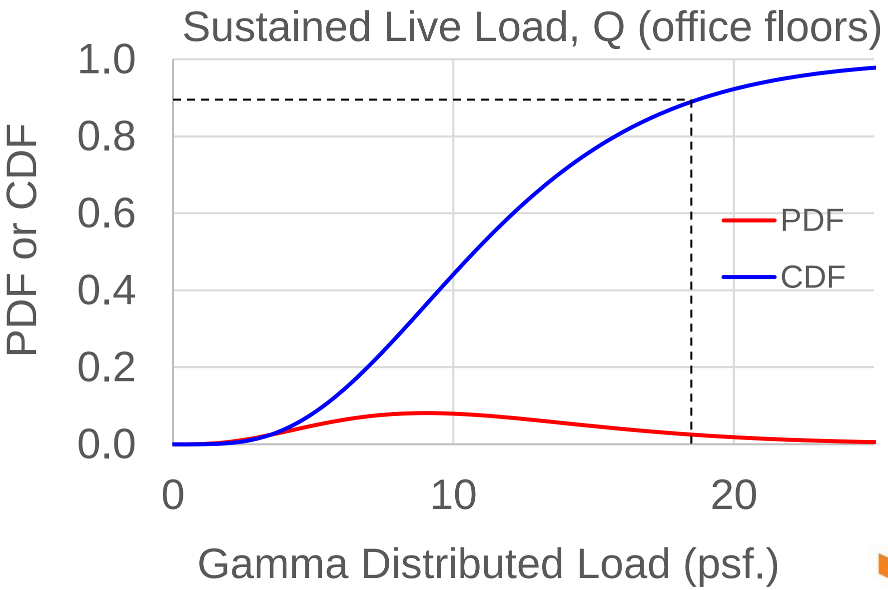
Assume Area = 4,000 ft^2, Var (Q) = 29.8 psf^2, COV (Q) = 0.47

Gamma or Type I Extreme Value Distribution

$$f(x;\alpha,\beta) = \frac{x^{\alpha-1} e^{-\beta x} \beta^{\alpha}}{\Gamma(\alpha)}$$

where x, $\boldsymbol{\alpha}$, $\boldsymbol{\beta}$ > 0

$\boldsymbol{\Gamma(\alpha)}$ is the gamma function.



Modeling Transient (extraordinary) Live Loads (P)

P on residential floors (Corotis 1983)

Mean (P) = 26.7 psf.

COV (P) = 0.69

Gamma or Exponential distribution

Historical surveys in ANSI A58

(i) 1949 local survey (ref. 5 in ASCE/SEI 7)

(ii) 1955 ANSI survey (ref. 5 in ASCE/SEI 7)

(iii) 1971 survey by Bruce Ellingwood (ref. 10 in ASCE/SEI 7)

(iv) 1971 local survey by MIT (ref. 8 in ASCE/SEI 7)

Total Live Load (sustained Q + extraordinary P)

(i) $L = Q_{max.} + P$ at the occurrence of $Q_{max.}$

(ii) $L = P_{max.} + Q$ at the occurrence of $P_{max.}$

(iii) $L = Q_{max}$ + the largest P with relatively small probability

(iv) Other cases

(v) Total Live Load (L) = weighted (i) to (iii) using the total probability concept.

(vi) The design live loads in Table 4-1 of ASCE/SEI 7 came from a Delphi method that involved top 25 structural engineers in 1978 (Corotis 1981 Ref. 7)

Total live load (L) can be modeled as Type I Extreme Value Distribution.

According to "Calibration of Design Code for Buildings (ACI318): Part 2" (2003) by Prof. Andrzej Nowak, University of Michigan, currently, Auburn University

For average live loads, bias coef. = 0.24 and COV = 0.65

Modeling Snow Loads

- p_f = (0.7). (C_e). (C_t). (C_s) (I). (nominal p_g)

(0.8 ~ 1.2) (0.85 ~ 1.3 C_s) (Figure 7-1 or Table 7-1 of **ASCE/SEI 7**)
(exposure factor) (thermal and slope roof)

Random Variable	Bias coef. *	COV*	Probability Distribution
(C_e)	1.0	0.15	Log-normal* and Gamma or Gumbel **
(C_t) (C_s)	1.0	0.10	
(I) (p_g)	0.27 ~ 0.32	0.57	
maximum snow average snow	0.82*** 0.20***	0.26*** 0.87***	

* Refer to "Structural Reliability Handbook" (2015, Australian Building Codes Board)
** Refer to "Risk and Safety in Civil, Surveying and Environmental Engineering" (by Prof. Michael Faber, Swiss Federal Institute of Technology, ETH Zurich)
*** Refer to "Calibration of Design Code for Buildings (ACI318): Part 2" (2003) (by Prof. Andrzej Nowak, University of Michigan, currently, Auburn University)

Modeling Wind Loads (velocity pressure)

- $q_z = (0.00256).(K_z).(K_{zl}).$ $(K_e).$ $(K_d).$ (nominal V^2)

(0.57 ~ 1.43 when z < 100 ft.) (0.8 ~ 1.0) (0.85 ~ 1.0) (Section 26.5 of **ASCE/SEI 7-16**)

(shape factor) (exposure factor) (directional factor)

Random Variable	Bias coef.	COV	Probability Distribution
(K_z) (K_{zl})	1.0	0.2* 0.10 ~ 0.30**	Log-normal*, ** except for V ~ Gumbel**
(K_e) (K_d)	1.0	0.15* 0.10 ~ 0.20**	
(V)	0.14 ~ 0.41* 10 min. mean**	0.49 ~ 0.72* 0.20 ~ 0.30**	
maximum wind	0.78***	0.37***	

* Refer to "Structural Reliability Handbook" (2015, Australian Building Codes Board)

** Refer to "Risk and Safety in Civil, Surveying and Environmental Engineering" (by Prof. Michael Faber, Swiss Federal Institute of Technology, ETH Zurich)

*** Refer to "Calibration of Design Code for Buildings (ACI318): Part 2" (2003) (by Prof. Andrzej Nowak, University of Michigan, currently, Auburn University)

Principles in Load Combinations

- **Turkstra's** load combination rule **(1980)**

  The maximum value of the combined loads occurs when one of the loads reaches its maximum value (primary load), while other loads have their instantaneous or arbitrary-point-in-time values (companion load).

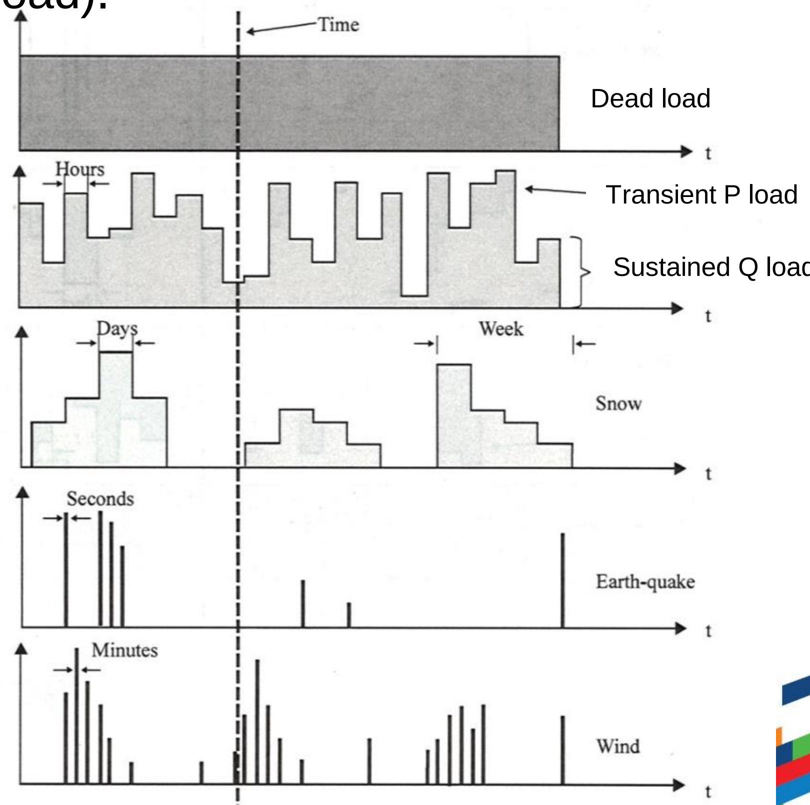
$$Z_1 = \max_T\{X_1(t)\} + X_2(t^*) + X_3(t^*) + \ldots + X_n(t^*)$$

$$Z_2 = X_1(t^*) + \max_T\{X_2(t)\} + X_3(t^*) + \ldots + X_n(t^*)$$

$$\vdots$$

$$Z_n = X_1(t^*) + X_2(t^*) + X_3(t^*) + \ldots + \max_T\{X_n(t)\}$$

$$X_{max}(T) \approx \max_i\{Z_i\}$$



Recall the example on live loads

(i) L = Q $_{max.}$ + P at occurrence of Q $_{max.}$

(ii) L = P $_{max.}$ + Q at occurrence of P $_{max.}$

Load Combinations in IBC, ASCE/SEI 7, and ACI 318

Load combination	IBC	ASCE/SEI 7	ACI 318
U = 1.4D	Y	Y	Y
U = 1.2D + 1.6L + 0.5(L_r or S or R) 1.2D + 1.6L + 0.5(Lr or 0.6S or R)	Y	Y in 7-22	Y
U = 1.2D + 1.6(L_r or S or R) + (L or 0.5W) 1.2D + 1.6(L_r or 0.6S or R) + (L or 0.5W)	Y	Y in 7-22	Y
U = 1.2D + W + L + 0.5(L_r or S or R) 1.2D + W + L + 0.5(L_r or 0.6S or R)	Y	Y In 7-22	Y
U = 1.2D + E + L + 0.2S 1.2D + E + L + (0.7 or 0.2)S	Y New	Deleted in 7-16 due to EQ	Y
U = 0.9D + W	Y	Y	Y
U = 0.9D + E	Y	Deleted in 7-16	Y
U = 1.4D + 1.7L 0.75(1.4D + 1.7L + 1.7W) 0.9D + 1.3W 0.75(1.4D + 1.7L +1.87E)		ACI 318-99 since 1950's Appendix C in ACI 318-02	

Calibration of Design Code for Buildings (ACI 318) in 2003

ACI STRUCTURAL JOURNAL — TECHNICAL PAPER

Title no. 100-S41

Calibration of Design Code for Buildings (ACI 318): Part 1— Statistical Models for Resistance

by Andrzej S. Nowak and Maria M. Szerszen

This paper summarizes the resistance models for calibration of the ACI 318 Code. The reliability analysis and calculation of the

be formulated for each possible failure mode for design and during service life of the considered structure. The load and

ACI STRUCTURAL JOURNAL — TECHNICAL PAPER

Title no. 100-S42

Calibration of Design Code for Buildings (ACI 318): Part 2— Reliability Analysis and Resistance Factors

by Maria M. Szerszen and Andrzej S. Nowak

Calibration of the design code for concrete structures is presented in two studies. The first one focused on the development of resistance models. This paper deals with the reliability analysis and selection of resistance factors. The structural types considered in this study include beams, structural slabs, and columns. The analysis is performed for reinforced concrete and prestressed concrete elements. A wide range of materials is covered: ordinary concrete, high-strength concrete, lightweight concrete, reinforcing bars No. 3 through 11, and two grades of prestressing strands. The reliability analysis requires the knowledge of the statistical parameters for

from material tests performed in 2000 and 2001 (Nowak and Szerszen 2003). The quality of workmanship can have a strong influence on materials, and it is assumed to be of average level.

CALIBRATION PROCEDURE

The calibration procedure used for selection of resistance factors is based on the structural reliability theory (Nowak and Collins 2000). The calibration procedure includes five steps. First, the types of structural elements and materials covered

Engineering Faculty Profile

Faculty and staff can submit directory updates here.

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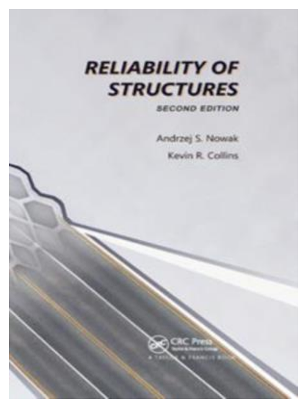

Calibration of LRFD Bridge Code

Andrzej S. Nowak,[1] Member, ASCE

Abstract: This paper reviews the code development procedures used for the new load and resistance factor design (LRFD) bridge code. The new code is based on a probability-based approach. Structural performance is measured in terms of the reliability (or probability of failure). Load and resistance factors are derived so that the reliability of bridges designed using the proposed provisions will be at the predefined target level. The paper describes the calibration procedure (calculation of load and resistance factors). A new live load model is proposed, which provides a consistent safety margin for a wide spectrum of spans. The dynamic load model takes into account the effect of road roughness, bridge dynamics, and vehicle dynamics. Statistical models of resistance (load-carrying capacity) are summarized for noncomposite steel, composite steel, reinforced concrete, and prestressed concrete. The reliability indices for bridges designed using the proposed code are compared with the reliability indices corresponding to the current specification. The proposed code provisions allow for a consistent design with a uniform level of reliability.

INTRODUCTION

The objective of this paper is to present the procedures used in the calibration of a new load and resistance factor design (LRFD) bridge code. The allowable stress method and load factor design, specified in the current AASHTO code (*Standard* 1992), do not provide for a consistent and uniform safety level for various groups of bridges. One of the major goals of the new code is to provide a uniform safety reserve. The main parts of the current AASHTO (*Standard* 1992) specification were written about 50 yr ago. There were many changes and adjustments at different times, which resulted in gaps and inconsistencies. Therefore, the work on the LRFD code also involves rewriting the document based on the state-of-the-art knowledge about various branches of bridge engineering. This paper summarizes some of these changes related to load and resistance models.

The theory of code writing has advanced in the last 20 yr. Some of the important contributions were summarized by Madsen et al. (1986), Melchers (1987), Ellingwood et al. (1980), and Nowak and Lind (1979). The major tool in the development of a new code is the reliability analysis procedure. Structural performance is measured in terms of the reliability or probability of failure. The code provisions are formulated so that structures designed using the code have a consistent and uniform safety level. The available reliability methods are reviewed in several textbooks (Thoft-Christensen and Baker 1982; Madsen et al. 1986; Melchers 1987). The methods vary with regard to accuracy, required input data, computational effort, and special features (time variance).

In an LRFD code, the basic design formula is

$$\sum \gamma_i X_i < \phi R_n \quad (1)$$

where X_i = nominal (design) load component i; γ_i = load factor i; R_n = nominal (design) resistance; and ϕ = resistance factor. The objective of calibration is to determine load and resistance factors so that the safety of bridges designed according to the code will be at the preselected target level.

This paper presents the calibration procedure, including load models, resistance models, reliability analysis, and the development of load and resistance factors. Bridge load and resistance models are only summarized here because they are described in other papers (Nowak 1993; Nowak and Hong 1991; Hwang and Nowak 1991; Tabsh and Nowak 1991; Ting and Nowak 1991; Nowak et al. 1993). Load and resistance are treated as random variables and are described by bias factors (ratio of mean to nominal), denoted by λ, and by coefficients of variation, denoted by V.

CALIBRATION PROCEDURE

The development of a new code involves the following steps:

1. Selection of representative bridges: About 200 structures were selected from various geographical regions of the United States. These structures cover materials, types, and spans, which are characteristic of the region. Emphasis is placed on current and future trends; instead of on very old bridges. For each selected bridge, load effects were calculated for various components. Load-carrying capacities were also evaluated.
2. Establishment of statistical database for load and resistance parameters: The available data on load components, including results of surveys and other measurements, were gathered. Truck survey and weigh-in-motion (WIM) data were used for modeling live load. There was little field data on the dynamic load and, therefore, a numerical procedure was developed to simulate the dynamic bridge behavior. Statistical data for resistance include material tests, component tests, and field measurements. Numerical procedures were developed to simulate the behavior of large structural components and systems.
3. Development of load and resistance models: Loads and resistance are treated as random variables. Their variation is described by cumulative distribution functions (CDF) and correlations. The CDFs for loads were derived using the available statistical database (step 2). The live load model includes the multiple presence of trucks in one lane and in adjacent lanes. Multilane reduction factors were calculated for wider bridges. The dynamic load was modeled for single trucks and two trucks, side-by-side. Resistance models were developed for girder bridges. The variation of the ultimate strength was determined by simulations.
4. Development of reliability analysis procedure: Structural performance is measured in terms of the reliability or probability of failure. Reliability is measured in terms of the reliability index β, calculated by using an iterative procedure. The developed load and resistance models (step 3) are part of the reliability analysis procedure.

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Note. Associate Editor: Dennis R. Mertz. Discussion open until January 1, 1996. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on January 8, 1993. This paper is part of the ***Journal of Structural Engineering***, Vol. 121, No. 8, August, 1995. ©ASCE, ISSN 0733-9445/95/0008-1245–1251/$2.00 + $.25 per page. Paper No. 5403.

Published (1995) in ASCE J. Structural Engineering

Summary on Loads

- **Loads:** ACI 318 (new structure design) → **General Building Code** (*e.g.* current IBC, IRC & NFPA 5000)

  ACI 562 (existing structure assessment) → **Design-Basis Code** (*i.e.* IEBC and original design code)

  ↓ (current IBC) (current IRC) (current NFPA 5000)

  ↓ (current IBC) (previous IBC) (UBC or BOCA etc.)

Concrete design and assessment loads are based on the nominal loads in ASCE/SEI 7.

IBC-18: Dead (Section 1606) → Tables C3-1 and C3-2 of ASCE/SEI 7-16

Live (Section 1607), Table 1607.1 → Table 4-1 of ASCE/SEI 7-16

Snow (Section 1608) → Chapter 7 of ASCE/SEI 7-16

Wind (Section 1609) → Chapters 26 – 31 of ASCE/SEI 7-16

Summary on Loads (cont.)

maximum 25-year load for ACI 562 repairs ?

Load Component	Arbitrary-point-in time load		Probability Distribution	Maximum 50-year load		Probability Distribution
	λ mean-to-nominal ratio	v% coefficient of variation		λ mean-to-nominal ratio	v% coefficient of variation	
Dead load	1.14	18	Normal	1.14	18	Normal
Live load	0.2	70	Gamma	1.10	25	Type I
Snow	0.48	0.35	Type II	1.01	17	Log-normal
Wind	0.0	0.0	-	0.97	0.24	Type I

Table 2. Statistical Parameters for Load Combinations (Assi, 2001).

ACI 348 will have a position paper.

Published in 2005

Reliability-Based Load Criteria for Structural Design: Load Factors and Load Combinations

Summary on Load Combinations

- **Load Combinations:** ACI 318-14 → **Table 5.3.1**

  ACI 562-21 → **Design-Basis Code** (*i.e.* IEBC and original design code)

  ↓ (current IBC) (current IRC) (current NFPA 5000)

  ↓ (current IBC) (previous IBC) (UBC or BOCA etc.)

ASCE/SEI 7 is the key document on loads and load combinations.

ACI 318 -14: Table 5.3.1 → Section 2.3.2 of ASCE/SEI 7-10

without seismic loads → Section 2.3.1 of ASCE/SEI 7-16

IBC-18: Section 1605.2 with additional snow load factor of 0.7

→

Section 2.3.2 of ASCE/SEI 7-10

Future Research Needs on Loads

- **Dead Loads:**

  Self-weights or dead loads ➡ deterministic variables in structural assessment?

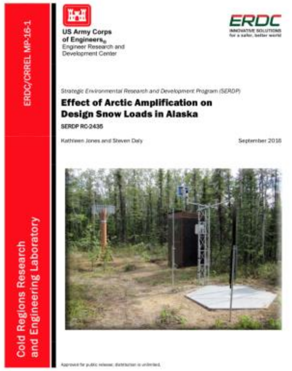
- **Live Loads:**

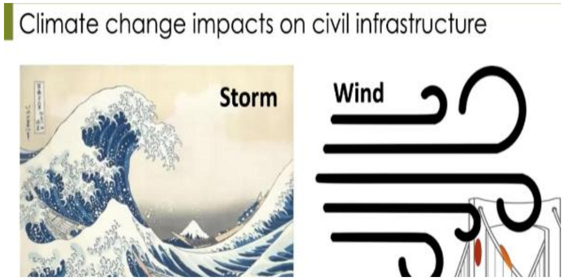
  Effects of the different design lifespans *T* in new structure design (50 ~ 75 years) and in existing structure repairs (10 ~ 25 years).

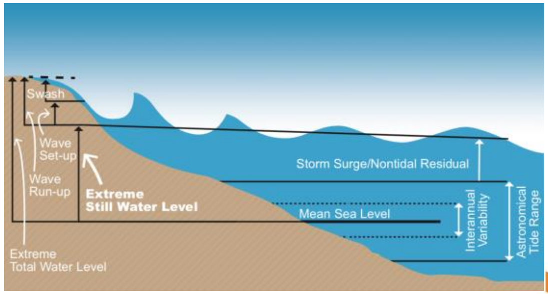
  $$Max.\,Live\ Load = \exp(-\nu T(1 - CDF)) \qquad \text{(Ellingwood 1977)}$$

  where ν = average rate of occupancy changes = 0.125 or 0.5 for every 8 or 2 years; CDF = cumulative function of the live loads. (new survey data?)

- **Snow and Wind Loads** (extreme weather events e.g. non-stationary high/low temperatures, heavy precipitation, and wind speeds V due to climate change)

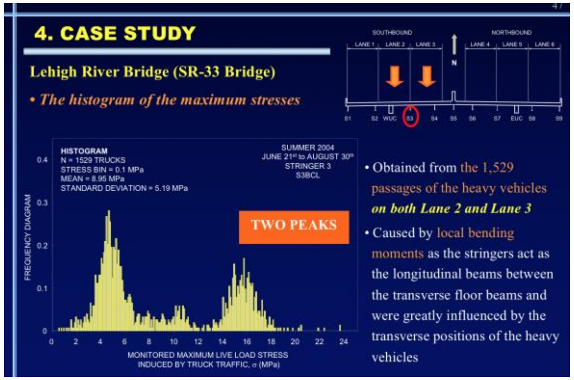


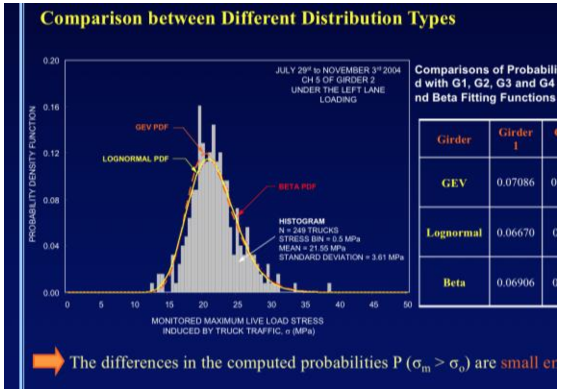




Future Research Needs on Loads (cont.)

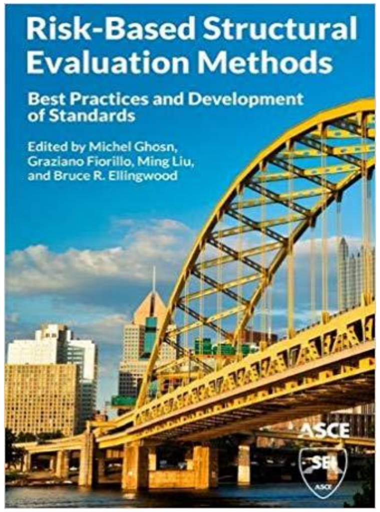
- **Data from Structural Health Monitoring (SHM):**





- **Data from Weight-in-Motion (WIM):** (photos from ACI PRC – 444.2-21)

- **Data from routine inspection:**
  (*e.g.* reservoir water heights for dam safety)



Future Research Needs on Load Combinations

- **Load combinations for structural assessment (ACI 562):**
  Since the root causes of uncertainties in assessment of existing structures are significantly different from those in design of new structures, the load factors, load combinations, and strength reduction factors should be determined using the calibration procedures presented by Prof. Nowak. Currently, such calibration of code requirements for structural assessment has not been conducted yet.
- **Other rules and methods in load combinations (e.g. Monte Carlo simulation)**
  (i) Ferry Borges-Castanheta's model (1980, 1982)
  The loads x_1 (Dead) and x_2 (Live) have the time duration of t_1 and t_2, respectively. During the design life *T* years, the occurrence of x_1 (*i.e.* change of the Dead) will be $n_1 = T/t_1$. During the duration of t_1, the occurrence of x_2 (*i.e.* change of the Live) will be $n_2 = t_1/t_2$ ($t_1 > t_2$). The maximum value of x_2 during $n_2 = max_{n_2}[x_2]$ Thus, the maximum value of the combined load $Y = x_1 + x_2 = max_{n_1}\left[x_1 + max_{n_2}(x_2)\right]$

(ii)Wen's load coincidence method (1978, 1981)

$$P(E,T) \approx 1 - \left\{exp\{-[\sum_{i=1}^{n} \lambda_i p_i + \sum_{i-1}^{n-1} \sum_{j=i+1}^{n} \lambda_{ij} p_{ij} + \cdots]T\}\right\}$$

Welcome to ACI 348 committee meeting

348 - Structural Reliability and Safety

Modify Committee Home

Committee Mission: Develop and report information on the use of reliability-based methods in the design, assessment, and rehabilitation of new and/or existing concrete structures.

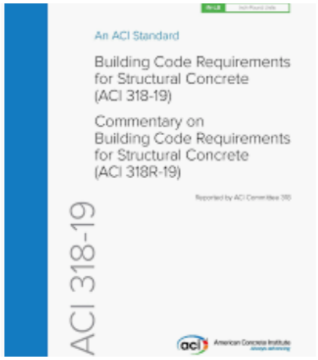
Goals: 1) Provide input to 318 on code calibration and load and resistance factors; 2) Prepare an article for Concrete International targeting the dissemination of structural reliability concepts and methods; 3) Develop Tech Notes on: i)the use of statistics in the evaluation of the equivalent design strength of concrete cores and ii)risk based procedure for sampling and assessment of structural deficiencies; iii) reliability of existing structures

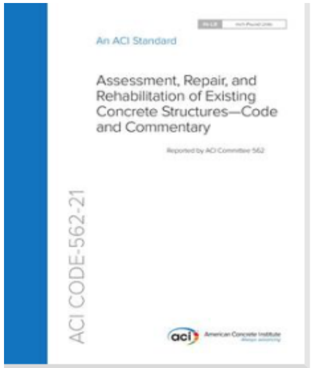
Chair: Mahmoud Maamouri

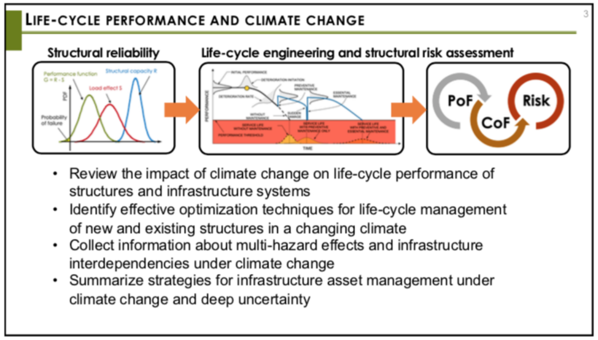
TAC Contact: Carl Larosche

Upcoming Open Meetings:
ACI Spring Convention 2022 - 3/28/2022 1:30 PM
Orlando, FL

Upcoming Convention Sessions:
Reliability and Safety of Existing Concrete Struc
ACI Spring Convention 2022 - Orlando, FL







ACI 348 committee meeting (Spring 2022)
Monday, March 28, 2022   1:30 PM – 3:00 PM (EDT)
Room C-Curacao 1, Caribe Royale Orlando, Orlando, FL