PE Civil Structural Exam Prep

Practice Exam 7

100-Question Simulation

NCEES PE Civil - Structural Blueprint

Exam Instructions

This practice exam contains 100 questions simulating the NCEES PE Civil - Structural CBT examination. You have 9 hours. Domain coverage mirrors the official NCEES PE Civil - Structural exam blueprint.

Domain	Topic Area	Questions
1	Loads	17
2	Analysis of Forces and Load Effects	25
3	Temporary Structures	7
4	Materials	14
5	Component Design (Structural Elements)	37
	Total	**100**

PRACTICE EXAM 7 -- QUESTIONS

DOMAIN 1 -- Loads (Q1-Q17)

1. Per ASCE 7-16 Table 4.3-1, general storage areas (not otherwise listed) have a minimum uniformly distributed live load of:

- **A.** 125 psf
- **B.** 100 psf
- **C.** 150 psf
- **D.** 75 psf

2. For a roof subject to significant snowfall (S>0), ASCE 7-16 LRFD governing combination when snow controls the roof design (no live load on roof) is:

- **A.** 1.2D + 1.0W + 0.5S
- **B.** 0.9D + 1.0W + 0.5S
- **C.** 1.2D + 1.6L + 0.5S
- **D.** 1.2D + 1.6S + L + 0.5W

3. For a Risk Category III hospital structure with SDS=0.60g and SD1=0.25g, ASCE 7-16 Tables 11.6-1 and 11.6-2 assign a Seismic Design Category (SDC) of:

- **A.** SDC A
- **B.** SDC C
- **C.** SDC B
- **D.** SDC D

4. ASCE 7-16 Section 7.3.1 defines the roof snow exposure factor Ce. For a site that is 'sheltered' (roof surrounded on all sides by obstructions 10 ft or more tall and within 50 ft), Ce equals:

- **A.** 0.90 -- reduced exposure, sheltered site
- **B.** 1.00 -- standard balanced exposure
- **C.** 1.20 -- increased snow accumulation for sheltered roofs
- **D.** 1.30 -- maximum penalty for fully enclosed sheltered rooftops

5. A simply supported interior steel beam (KLL=2) supports an unfactored uniform live load Lo=80 psf over a tributary area AT=500 sf. Per ASCE 7-16 Section 4.7.3, the reduced design live load L=Lo(0.25+15/sqrt(KLLxAT)). What is L?

- **A.** 40 psf -- 50% minimum governs
- **B.** 50 psf
- **C.** 57.9 psf
- **D.** 72 psf

6. A two-story building has w1=800 kips at h1=12 ft (2nd floor) and w2=600 kips at h2=24 ft (roof). The seismic base shear V=200 kips with T=0.6 s (k=1.0). Per ASCE 7-16 Section 12.8.3, the seismic force at the roof level Fx=CvxxV where Cvx=wxxhx^k/Σwixhi^k. What is F at the roof?

- **A.** 120 kips
- **B.** 80 kips
- **C.** 100 kips
- **D.** 200 kips

7. Per ASCE 7-16 Section 12.7.2, the effective seismic weight W used to compute the seismic base shear includes all of the following EXCEPT:

A. All permanent dead loads including partitions (minimum 15 psf)

B. Wind loads from the design wind speed event occurring simultaneously with the design earthquake

C. The full roof snow load when it exceeds 30 psf

D. 25% of design floor live load in areas used for storage

8. For a structure with SD1=0.5g, T=0.8 s, R=8, Ie=1.0, the upper bound on Cs per ASCE 7-16 Eq. 12.8-3 (for T <= TL) is Cs_max=SD1xIe/(TxR/Ie). What is Cs_max?

A. 0.100

B. 0.063

C. 0.125

D. 0.0781

9. ASCE 7-16 Table 12.3-1 defines horizontal structural irregularities. Type 3 -- diaphragm discontinuity -- exists when:

A. The ratio of maximum to average story drift exceeds 1.2

B. The diaphragm has an opening larger than 50% of the gross diaphragm area, or a change in diaphragm stiffness exceeding 50% from one story to the next

C. The building has re-entrant plan corners where both plan projections exceed 15% of the plan dimension in that direction

D. The sum of the columns in any story is more than 10% shorter than the columns in the adjacent story

10. ASCE 7-16 Table 12.3-2 defines vertical structural irregularities. A soft story (Type 1a) irregularity exists when the lateral stiffness of any story is:

A. Less than 70% of the stiffness of the story above, OR less than 80% of the average stiffness of the three stories above -- this is the correct Type 1a trigger per ASCE 7-16 Table 12.3-2

B. Less than 60% of the lateral stiffness of the story directly above -- a single-story comparison without averaging

C. Less than 50% of the stiffness of the story directly below -- a ratio comparing to the stiffer adjacent story rather than the story above

D. More than 150% of the lateral stiffness of the story above -- this describes a stiff story irregularity, which is opposite to the soft story condition

11. ASCE 7-16 Chapter 26 establishes minimum design wind speeds. For a Risk Category II building in a standard inland US location, the basic wind speed V (3-second gust) used in ASCE 7-16 design ranges from approximately 85 to 130 mph depending on location. The design wind speed for a typical mid-continental US location for RC II is approximately:

A. 85 mph

B. 100 mph

C. 115 mph

D. 130 mph

12. For a granular backfill with friction angle phi=30°, the active earth pressure coefficient by Rankine's theory is Ka=tan2(45°−phi/2). What is Ka?

A. 0.333

B. 0.500

C. 0.250

D. 0.667

13. Per ASCE 7-16 Section 8.3, the design rain load R (in psf) applied to a roof when secondary drainage is blocked equals R=5.2(ds+dh), where ds=depth of water above the secondary drain inlet (in inches) and dh=additional depth. For a roof with ds=1.0 in and dh=1.5 in, what is R?

- **A.** 13.0 psf
- **B.** 5.2 psf
- **C.** 7.8 psf
- **D.** 10.4 psf

14. Per ASCE 7-16 Table 1.5-2, the snow load importance factor Is for a Risk Category III structure (schools, healthcare facilities, public utilities) is:

- **A.** Is = 1.00
- **B.** Is = 1.10
- **C.** Is = 1.20
- **D.** Is = 0.80

15. Per ASCE 7-16 Section 4.6.2, the impact load allowance for elevator machinery (hoist beams and related framing) is taken as a percentage increase on the static lifted weight. That percentage is:

- **A.** 25% increase on the hoist beam reaction
- **B.** 50% increase on the static machine weight for overhead electric-geared elevators
- **C.** 50% increase for hoist beams, applied to the machine load plus rope load
- **D.** 100% increase -- elevator beams must be designed for twice the rated load

16. ASCE 7-16 LRFD load combination 6 for structures resisting earthquake loads (Section 2.3.2) is:

- **A.** 1.2D + 1.0W + 1.0L + 0.5S
- **B.** 0.9D + 1.0W
- **C.** 1.2D + 1.6S + L
- **D.** 1.2D + 1.0E + 1.0L + 0.2S

17. Per ASCE 7-16 Section 12.12.3, the minimum seismic separation gap (building separation at a seismic joint between two adjacent structures) must equal or exceed:

- **A.** The sum of their maximum inelastic deflections δMT = δM1 + δM2, where δM = Cdxδe/Ie for each structure
- **B.** 2 inches at all floor levels, regardless of building height
- **C.** 1% of the height above grade at the point of separation
- **D.** The greater of 1 inch or the larger of the two structures' story drifts

DOMAIN 2 -- Analysis of Forces and Load Effects (Q18-Q42)

18. Two C8x18.75 channels (Ix=43.9 in4 each, A=5.51 in2 each, x■=0.565 in from back of web) are placed back-to-back with webs touching. The compound moment of inertia about the shared vertical axis of symmetry (the Y-axis perpendicular to the channel webs) is I=2(Ix+Axx■2). What is I?

- **A.** 87.8 in4
- **B.** 75.4 in4
- **C.** 91.3 in4
- **D.** 43.9 in4

19. In a doubly symmetric I-section subjected to non-uniform bending (causing lateral torsional buckling risk), warping torsional resistance Cw is significant because:

- **A.** Closed hollow sections depend entirely on warping for torsional resistance, which is why W-shapes are avoided for high-torque applications

B. Warping resistance in open thin-walled I-sections involves longitudinal normal stresses in the flanges (bimoment), which supplements St. Venant shear flow and dominates at shorter spans relative to closed sections

C. Warping torsional stiffness equals GJ for all sections -- I-sections use GJ in all AISC torsion calculations

D. St. Venant torsion always governs for I-sections -- warping can be neglected per AISC Design Guide 9

20. The three-moment equation (Clapeyron's equation) for continuous beams establishes compatibility of:

A. Shear forces at each interior support, ensuring vertical equilibrium is satisfied

B. Bending moments at midspan of each span, equating the maximum positive moments

C. Axial forces in continuous beams where horizontal restraint is provided at the supports

D. Slopes at each interior support from both adjacent spans, ensuring the beam remains continuous (no kink) at each interior support

21. A steel beam (E=29,000 ksi, I=200 in4) is fixed at both ends and spans L=20 ft (240 in). A concentrated load P=40 kips is applied at midspan. The midspan deflection is δ=PL3/(192EI). What is δ?

A. 0.994 in

B. 0.249 in

C. 0.166 in

D. 0.497 in

22. For a propped cantilever (fixed at A, pinned at B, span L) carrying uniform load w, the maximum positive moment in the span occurs at a distance from A where the shear is zero. With RA=5wL/8 (at fixed end) and RB=3wL/8 (at pin), the maximum positive moment is located at x from the fixed end where V=0, and equals:

A. M_max(+) = 9wL2/128, located at x=5L/8 from the fixed end

B. M_max(+) = wL2/8, at midspan

C. M_max(+) = wL2/10, at x=L/2

D. M_max(+) = wL2/16, located at the quarter-point

23. The principle of complementary shear stresses states that:

A. Shear stress on any horizontal face is always zero when normal stress at that point reaches its maximum or minimum value

B. The shear stress on any face of a stressed element equals in magnitude the shear stress on the perpendicular face at the same point -- they form a complementary pair that maintains rotational equilibrium

C. Shear stresses from different load cases add algebraically when referenced about different coordinate axes in the member

D. Torsional shear stresses and flexural shear stresses in a beam web always act in opposite directions and partially cancel each other

24. In the portal method for approximate analysis of lateral loads on multi-story frames, interior columns are assumed to carry lateral shear equal to:

A. The same shear as each exterior column -- all columns in a story share equally

B. Zero shear -- only the exterior columns resist lateral loads in the portal method

C. Twice the shear carried by each exterior column -- interior columns participate doubly as part of two portals

D. Half the shear of exterior columns -- exterior columns are stiffer because of their direct connection to the ground

25. A simply supported beam (w=1.5 kip/ft=0.125 kip/in, L=240 in, E=29,000 ksi, I=300 in4) is analyzed by the conjugate beam method. The rotation (slope) at the left support theta_A = wL3/(24EI) equals:

A. 0.00414 rad

B. 0.01242 rad

C. 0.00828 rad

D. 0.01656 rad

26. For a fixed-fixed beam carrying a concentrated load P at midspan, plastic collapse occurs when plastic hinges form simultaneously at:

A. Midspan only -- the fixed ends remain elastic at collapse

B. Both fixed ends only, releasing the beam as a simply supported span

C. One fixed end and at midspan, forming a collapse mechanism for the half-span

D. Both fixed ends and at midspan, forming a three-hinge collapse mechanism for the full span

27. A steel column (E=29,000 ksi, I=100 in4, L=10 ft=120 in) is fixed at its base and free at the top (flagpole condition). The critical Euler buckling load Pcr=pi2EI/(4L2) is:

A. 248 kips

B. 496.9 kips

C. 993 kips

D. 124 kips

28. Creep in concrete is the time-dependent increase in strain under sustained compressive load at constant stress. For ordinary structural concrete, the typical creep coefficient Cc (ratio of creep strain to initial elastic strain) after several years is approximately:

A. 1.5 to 2.0 for moderate humidity conditions

B. 0.1 to 0.3 -- creep is negligible for high-strength concrete

C. 3.0 to 5.0 -- high creep is normal for normal-strength structural members

D. 0.5 to 0.8 for all concrete regardless of ambient humidity

29. The shear center of a channel section (C-section, one web, two flanges) is:

A. At the centroid of the web plate -- the shear flows are symmetric about the web centerline for a standard C-section

B. Inside the channel between the web and the centroid, at a distance proportional to the moment of inertia of the flanges

C. At the centroid of the gross cross-section, where the combined effect of all shear flows produces zero net torque about the centroid

D. Outside the channel on the open side at distance e from the web, where e depends on flange and web proportions -- loading through this point produces pure bending without twisting

30. The principle of superposition in structural analysis is valid only when:

A. The structure is statically indeterminate with at least two degrees of indeterminacy

B. Deformations are small and the structure behaves linearly elastic -- material is elastic and deflections do not significantly change the geometry

C. All loads are applied simultaneously -- staggered loading invalidates superposition

D. Temperature and settlement effects are absent -- superposition applies only to mechanical loads

31. A three-hinged parabolic arch (span L=60 ft, rise f=8 ft) carries a uniform load w=2 kip/ft over the full span. The horizontal thrust at each support H=wL2/(8f) is:

A. 225 kips

B. 56.25 kips

C. 112.5 kips

D. 450 kips

32. The strain energy stored in a beam due to bending is U=∫M2dx/(2EI) evaluated over the full length. When this energy expression is differentiated with respect to a particular applied force P using Castigliano's theorem, the result equals:

A. The bending stress at the most highly stressed cross-section under load P

B. The deflection of the beam at the point of application of P in the direction of P

C. The maximum curvature of the elastic curve at the section of maximum moment

D. The support reaction at the point closest to load P

33. In the moment distribution method, the distribution factor (DF) for member BA at joint B (where members BA, BC, and BD connect) equals:

A. 1/3 regardless of member stiffness -- equal distribution at three-way joints

B. 0.5 -- moment always splits equally between the near end and the far end

C. (EI/L)_BA / [(EI/L)_BA + (EI/L)_BC + (EI/L)_BD] -- the ratio of the member stiffness to the sum of all stiffnesses at the joint

D. (EI/L)_BA / (EI/L)_BC -- the ratio relative to the most flexible adjacent member

34. In response spectrum analysis (RSA) per ASCE 7-16 Section 12.9, when combining modal contributions to compute total story forces and displacements, the Square Root of Sum of Squares (SRSS) combination method is considered adequate when:

A. All natural frequencies are well-separated (no two natural periods are within 10% of each other) -- SRSS assumes modal independence

B. The structure is perfectly symmetric in both plan and elevation

C. There are fewer than 3 significant modes -- SRSS is limited to simple two-mode systems

D. The complete quadratic combination (CQC) always governs and SRSS is never adequate per current ASCE 7-16

35. ACI 318-14 Chapter 6 provides moment coefficients for continuous beams with two or more spans of approximately equal length under uniformly distributed load. The negative factored moment at the face of an interior support of a slab or beam with more than two spans is:

A. wuxln2/10

B. wuxln2/11

C. wuxln2/9 at the first interior support only

D. wuxln2/16 when the adjacent spans are equal

36. Maxwell's reciprocal theorem states that for a linearly elastic structure:

A. The maximum deflection in a beam equals the maximum rotation angle multiplied by the effective span length, at the same loaded section

B. The area under the shear force diagram between any two sections equals the change in bending moment between those two sections along a beam

C. The total external virtual work done by applied loads equals twice the total internal virtual strain energy stored in the elastic structure

D. The deflection at point i due to a unit load at point j equals the deflection at point j due to a unit load at point i -- flexibility coefficients are symmetric for linear elastic structures

37. In a roof diaphragm subjected to lateral wind or seismic load, the load path sequence from where the lateral force originates to where it is resisted at the foundation is:

A. Shear wall → collector/drag strut → diaphragm → lateral load source

B. Lateral load → diaphragm acts as horizontal beam → collector/drag strut transfers force → shear wall carries it to foundation

C. Foundation → shear wall → diaphragm chord → lateral load source (loads flow upward by AISC 360 convention)

D. Lateral load → floor slab → column → shear wall (diaphragm is not part of the lateral load path)

38. In an open thin-walled I-section subjected to a torque, the St. Venant (pure) torsion component and the warping torsion component are both active. Compared to a closed hollow section of similar cross-sectional area:

A. Open I-sections carry approximately the same torsional stiffness as closed sections because the web compensates for the open shape

B. Open sections have much lower St. Venant torsional stiffness (GJ is proportional to t3, not t for thin walls) but can develop significant warping resistance over shorter spans

C. Warping torsion is irrelevant for I-sections -- only closed box sections develop warping normal stresses

D. Closed sections always fail by warping instability before St. Venant torsion can develop

39. Two springs of stiffness k1=200 kip/in and k2=100 kip/in are connected in series (the same force passes through each spring). The equivalent combined stiffness k_eq is:

A. 300 kip/in (springs add directly when in series)

B. 100 kip/in (equals the softer spring)

C. 66.7 kip/in

D. 150 kip/in (harmonic mean of k1 and k2)

40. For a rectangular plate simply supported on all four edges under uniform in-plane compressive stress sigma acting on two opposite edges (width b, length a), the critical buckling stress is sigmacr=kpi2E/[12(1–nu2)(b/t)2]. For the lowest buckling mode of a square plate (a=b) with nu=0.3, the buckling coefficient k is:

A. 4.0 -- the standard value for a simply supported plate under uniform compression

B. 6.97 -- for a clamped plate on all four edges

C. 0.904 -- the minimum for a plate with one free edge

D. 1.0 -- always the critical mode buckling coefficient for any rectangular plate

41. The distinction between a catenary and a parabolic cable shape is:

A. Catenaries form under any distributed load; parabolas form only from point loads at midspan because concentrated forces create localized curvature changes

B. A parabola results from a uniform horizontal load per unit length; a catenary forms when the load is uniform along the cable arc length, such as from the cable's own self-weight

C. A parabola results from a cable carrying uniform load per unit horizontal distance (e.g., suspension bridge deck load via hangers); a catenary forms under load uniform per unit arc length (e.g., cable's own self-weight)

D. There is no physical distinction between a catenary and a parabola -- the two terms are synonymous in structural engineering for any hanging cable shape under vertical loading

42. A column is fixed at its base and pinned at the top (braced against lateral movement). The rotational stiffness of this column (moment per unit rotation at the fixed base) equals:

A. 4EI/L -- same as a fixed-fixed member end

B. 3EI/L -- reduced by the released (pinned) far end

C. 2EI/L -- average of the fixed and pinned values

D. EI/L -- the stiffness coefficient for a member with no end restraint

DOMAIN 3 -- Temporary Structures (Q43-Q49)

43. When concrete formwork is stripped and reshoring is required before the slab reaches its full 28-day strength, the reshoring must be designed to support:

A. Only the weight of the fresh concrete being placed on the floor above -- forms above carry the slab weight

B. The weight of the existing slab alone -- no construction live load need be applied to reshores

C. 25% of the total design live load applied to the slab being reshored

D. The full weight of the reshored slab plus any construction loads transmitted from above -- reshores carry all loads the slab cannot yet resist on its own

44. Per OSHA 29 CFR 1926.502(d)(16), personal fall arrest systems must be rigged such that an employee cannot free fall more than 6 feet and cannot contact a lower level. Per 29 CFR 1926.502(b), guardrail systems are required on any walking/working surface with an unprotected edge at a height above a lower level of:

A. 6 feet or more above a lower level

B. 4 feet or more above a lower level

C. 10 feet or more -- fall protection begins at 10 feet for construction

D. Any height -- fall protection is required at all elevations in construction

45. Per ACI 305.1-14 (Specification for Hot Weather Concreting), the maximum ambient temperature at which concrete can be placed without special hot weather measures is:

A. 95°F (35°C) ambient -- ACI 305 requires protection above 95°F

B. 90°F (32°C) ambient -- cooling or retarding required above 90°F

C. 100°F (38°C) ambient -- concrete temperature controls; air temperature is not limited

D. 75°F (24°C) -- any temperature above 75°F requires hot weather concrete procedures

46. Per OSHA 29 CFR 1926.502(g), a Controlled Access Zone (CAZ) is used as a method of fall protection for leading edge work. The minimum width of a CAZ between the control line and the leading edge is:

A. 3 feet minimum between the control line and the leading edge -- provides a minimal buffer for workers to correct their footing before reaching the edge

B. 10 feet minimum between the control line and the leading edge -- the separation required when workers cannot use personal fall protection equipment

C. 6 feet minimum -- workers performing leading edge work stand within 6 feet of the unprotected edge inside the Controlled Access Zone

D. There is no minimum width for the CAZ -- the control line must only be positioned at a distance no closer than 6 feet from the leading edge, with no floor area minimum requirement between the line and the edge

47. Under the NPDES (National Pollutant Discharge Elimination System) Construction General Permit, a Stormwater Pollution Prevention Plan (SWPPP) is required for construction sites that disturb:

A. 1 acre or more of land surface -- federally required SWPPP threshold

B. Any area greater than 0.5 acres -- NPDES permit required for all significant clearing

C. 5 acres or more -- smaller sites are exempt from federal stormwater requirements

D. Only sites within 100 feet of a navigable waterway -- SWPPP is triggered by proximity to water, not acreage

48. ACI 347R-14 recommends maximum lateral pressure formulas for freshly placed concrete in forms. For concrete with a unit weight of 150 pcf placed under normal conditions (T>=50°F, R<=7 ft/hr), the maximum lateral pressure on a wall form is limited to:

A. Full hydrostatic pressure (gammaxh) throughout the form height -- conservative default for all placements

B. The lesser of CwxCcx[150+9000R/T] psf or full hydrostatic, with a minimum of 600 psf -- for rate <=7 ft/hr, temperature factor applies

C. 150 psf regardless of depth -- ACI assumes concrete arches and relieves pressure beyond 1 ft depth

D. 300 psf -- the fixed lateral pressure limit for all standard concrete formwork designs

49. ACI 207.1R (Guide to Mass Concrete) recommends limiting the temperature differential between the interior and exterior of a mass concrete placement to prevent thermal cracking. The recommended maximum differential is:

A. 50°F (28°C) -- allowed when Portland cement Type I is used

B. 35°F (19°C) -- the generally accepted limit to prevent thermally induced cracking

C. 20°F (11°C) -- for massive dam structures only

D. 100°F (56°C) -- the practical limit before concrete begins to steam-cure

DOMAIN 4 -- Materials (Q50-Q63)

50. ASTM A992 structural steel, commonly used for wide-flange shapes, specifies both a minimum and maximum yield strength AND a maximum yield-to-tensile strength ratio. Which of the following correctly describes A992?

A. Fy,min=50 ksi, Fu,min=65 ksi, maximum yield ratio Fy/Fu<=0.85 -- prevents brittle post-yield behavior in connections

B. Fy,min=36 ksi, Fu,min=58 ksi -- same as A36 but with added ductility requirements

C. Fy,min=50 ksi with no maximum Fy -- only minimum strength requirements are specified

D. Fy,min=50 ksi, Fu,min=70 ksi, maximum yield ratio Fy/Fu<=0.90

51. Per ACI 308R, the minimum recommended duration of moist curing for normal-strength concrete (using Type I Portland cement) to ensure adequate hydration and strength development is:

A. 3 days -- the ACI minimum for all concrete in dry climates

B. 5 days -- acceptable for most field conditions

C. 7 days -- the ACI recommended minimum for standard structural concrete

D. 14 days -- required whenever w/cm < 0.40

52. Per NDS 2018 Supplement, the size factor CF applies to sawn lumber reference bending design values to account for the effect of member size on apparent strength. For timbers 5 inches x 5 inches and larger (the 'Timbers' category), CF equals:

A. CF=1.15 -- a size-effect bonus factor applied to large timber sections to account for improved drying uniformity

B. CF=0.90 -- the NDS size penalty applied to all large-cross-section members with depth greater than 12 inches

C. CF=1.05 -- a modest bonus for massive timber sections that are less prone to strength-reducing natural defects

D. CF=1.00 -- the size factor does not apply to the Timbers category (5x5 and larger); it applies only to dimension lumber and is therefore not used for heavy timbers

53. In the USCS, a soil classified as ML has which of the following characteristics?

A. Gravelly sand with some fines -- USCS prefix M denotes medium-density material

B. Inorganic silt or very fine sand with low plasticity -- LL<50, plots below the A-line on the Casagrande plasticity chart

C. Well-graded silty sand with high fines content and high plasticity

D. Organic silt of low compressibility -- defined as having dark color and organic odor

54. Portland cement Type IV is formulated specifically for:

A. Rapid early strength development -- achieves 7-day strength equivalent to Type I 28-day strength

B. Resistance to sulfate attack in aggressive soil and groundwater environments

C. General purpose use -- the most widely available and commonly specified cement type

D. Low heat of hydration in mass concrete placements -- Type IV generates significantly less heat than Type I or III

55. Per AISC 360-16 Table J3.1, the minimum bolt pretension for a 3/4-inch diameter ASTM A490 bolt (installed by any approved method) used in a fully pretensioned connection is:

A. 22 kips

B. 35 kips

C. 28 kips

D. 51 kips

56. Per NDS 2018 Supplement Table 4A, the wet service factor CM for reference compression design value Fc (parallel to grain) for visually graded sawn lumber used in wet conditions (MC>19%) is:

A. CM = 0.91

B. CM = 0.67

C. CM = 0.85

D. CM = 0.80

57. Per TMS 402-16 Section 3.1.8.1, the minimum specified compressive strength of masonry f'm for concrete masonry unit (CMU) construction shall not be less than:

A. 2,000 psi -- the minimum grout compressive strength per TMS 602-16, often confused with the masonry f'm minimum

B. 3,000 psi -- a common design value specified by engineers for good durability but not the minimum per TMS 402-16

C. 2,600 psi -- the minimum f'm for clay brick masonry assemblages per TMS 402-16 Section 3.1.8.1, often confused with CMU

D. 1,500 psi -- the minimum specified compressive strength of masonry f'm for concrete masonry unit construction per TMS 402-16

58. ASTM C260 governs air-entraining admixtures for concrete. The primary purpose of entrained air in structural concrete exposed to freezing and wetting cycles is:

A. To increase workability and reduce the water requirement for a given slump

B. To reduce concrete unit weight and save on formwork costs

C. To create a system of small, discrete, well-distributed air bubbles that relieve the hydraulic pressure generated when pore water freezes, preventing surface scaling and spalling

D. To increase the modulus of elasticity by providing additional elastic capacity in the air void system

59. Structural steel heat traceability relies on the heat number stamped or stenciled on each piece of steel. When a fabricator substitutes a different heat of steel than originally ordered, the minimum required documentation is:

A. Only a verbal confirmation from the mill representative -- no written documentation needed for minor substitutions

B. A new certified mill test report (CMTR) for the substituted heat showing that it meets the required ASTM specification and grade

C. A structural engineer's letter of approval signed and sealed -- mill records are not required

D. No additional documentation -- the AISC Code of Standard Practice allows any ASTM-listed grade to substitute without documentation

60. Per NDS 2018 Section 4.3.9, the repetitive member factor Cr=1.15 applies to the reference bending design value Fb when members are used as:

A. Closely spaced members (spacing <=24 in) in groups of three or more where load sharing is possible -- such as repetitive floor joists, rafters, or studs with structural sheathing allowing load redistribution

B. Any structural lumber member with a span longer than 20 feet, where load redistribution is more likely due to deflection differences between adjacent members

C. Engineered lumber products such as LVL and PSL, where full composite action between laminations has been verified and load sharing is guaranteed by manufacturing

D. Timber sections 5x5 and larger where the repetitive member factor Cr replaces the size factor CF in the calculation of adjusted design values

61. APA-rated structural panels carry a span rating printed on the panel, such as '32/16'. For roof sheathing, this panel may be used with rafters spaced up to:

A. 16 in on center -- the first number applies to roof decks only

B. 24 in on center -- always the maximum for structural sheathing

C. 20 in on center -- an interpolated value between the two numbers

D. 32 in on center -- the first number in the span rating is the maximum rafter spacing for roofs

62. ASTM D1556 specifies a procedure for determining the in-place density of soil using the sand cone method. The test is performed by:

A. Excavating a small hole, filling it with calibrated dry standard sand from a cone apparatus to determine volume, then dividing the excavated soil mass by the volume to compute in-place dry density

B. Driving a calibrated cone to a standard depth and reading the drive resistance, then correlating the resistance value to dry density using laboratory charts

C. Pushing a thin-walled Shelby tube sampler into the ground to obtain an undisturbed sample, then measuring the wet and dry mass ratio in the laboratory

D. Inserting a nuclear density gauge probe into a drilled hole and measuring gamma-ray attenuation through the soil to compute in-place density electronically

63. ASTM A706 Grade 60 deformed bars are preferred over ASTM A615 Grade 60 bars in seismic applications primarily because:

A. A706 has a minimum specified yield strength of 80 ksi rather than 60 ksi, allowing smaller bar sizes

B. A615 bars cannot be welded without prequalification; all A706 bars are pre-qualified for welding at no additional cost

C. A706 controls the actual yield-to-tensile strength ratio (max Fy/Fu <= 0.80) and limits maximum actual yield strength to 78 ksi, ensuring ductile behavior under seismic inelastic demands

D. A706 bars are available only in Grades 80 and 100, providing higher stiffness for seismic applications

DOMAIN 5 -- Component Design (Q64-Q100)

64. Per ACI 318-14 Section 9.6.1.2, the minimum flexural reinforcement As,min in a non-prestressed beam (bw=12 in, d=18 in, f'c=4,000 psi, fy=60,000 psi) is the larger of 3sqrtf'c/fyxbwxd and 200xbwxd/fy. What governs As,min?

A. 0.60 in2 -- the lower of the two criteria

B. 0.72 in2 -- governed by 200bwd/fy

C. 0.55 in2 -- governed by 3sqrtf'c/fyxbwd

D. 0.90 in2 -- minimum for beams with rho < 0.25%

65. ACI 318-14 requires tension-controlled sections (epsilont >= 0.005) for most beams. At the maximum usable steel area satisfying epsilont = 0.004 (the minimum permitted for flexural members), the net tensile strain limits permitted phi=0.90 only when:

A. epsilont >= 0.005 -- the tension-controlled threshold for full phi=0.90

B. epsilont >= 0.004 AND As is less than rhomax -- the ACI 318-14 limit for most sections

C. epsilont >= 0.004 is the minimum; between 0.004 and 0.005 phi transitions linearly from 0.75 to 0.90

D. phi=0.90 always applies regardless of epsilont for all singly reinforced sections

66. Per ACI 318-14 Section 22.9, the required shear friction reinforcement Avf for a construction joint between old and new concrete (monolithic, normal weight, artificially roughened to ¼ in amplitude) with Vu=80 kips, phi=0.75, mu=1.0, fy=60 ksi is Avf=Vu/(phimufy). What is Avf?

A. 1.78 in2

B. 2.67 in2

C. 0.89 in2

D. 3.56 in2

67. Per ACI 318-14 Table 8.3.1.2 for flat plates (no drop panels, alphafm<0.2, fy=60,000 psi, betan=1.0), the minimum slab thickness h for an interior panel with clear span ln=20 ft (240 in) is approximately h=ln/30. What is the minimum h?

A. 6 in

B. 7 in

C. 10 in

D. 8 in

68. For a rectangular concrete beam (b=12 in, h=24 in, yt=12 in) with f'c=4,000 psi, the modulus of rupture fr=7.5sqrtf'c=474 psi=0.474 ksi and Ig=12x243/12=13,824 in4. The cracking moment Mcr=frxIg/yt is:

A. 27.3 kip.ft
B. 45.5 kip.ft
C. 91.0 kip.ft
D. 22.8 kip.ft

69. Per ACI 318-14 Section 16.4, horizontal shear between a precast beam and a cast-in-place topping slab can be resisted by: interface cohesion (if surface is intentionally roughened), friction, and shear connectors. When the design horizontal shear stress exceeds the cohesion/friction capacity, the required horizontal shear reinforcement Avh is computed from:

A. Avh = Vu/(phixmuxfy) per the shear friction provisions of ACI 318-14 Section 22.9
B. Avh = (Vuh - Vn,cohesion)/(phixfyxd) where the excess shear above cohesion is carried by the ties
C. Avh = Vu/(phix0.6xf'cxbv) -- horizontal shear friction requires only 60% of the compressive strength
D. No horizontal steel is needed when the topping is bonded -- friction always governs over rebar contribution

70. Per ACI 318-14 Section 25.5.3, the minimum compression development length ld,c for a #8 bar (db=1.0 in, fy=60,000 psi, f'c=4,000 psi, ψr=1.0, lambda=1.0) is ld,c=0.02ψrfy/(lambdasqrtf'c)xdb (with a minimum of 8 in or 0.0003fydb). What is ld,c?

A. 19.0 in
B. 12.0 in
C. 24.0 in
D. 8.0 in

71. Per ACI 318-14 Section 25.2.1, the minimum clear spacing between parallel bars in a single layer of reinforcement in a beam shall not be less than which of the following?

A. 1.5db, with a minimum of 1.5 in and 1.33 times the maximum aggregate size
B. 0.5db, with no minimum absolute value
C. The largest of: the bar diameter db, 1 inch, and 4/3 times the maximum aggregate size
D. 2db for bundled bars and 1db for single bars, per a separate provision

72. ACI 318-14 Section 21.2 assigns different strength reduction factors phi to compression-controlled spiral and tied columns. The values are:

A. phi=0.75 for both tied and spiral columns -- ACI 318-14 unified the two in the 2014 edition
B. phi=0.80 for spiral columns, phi=0.75 for tied columns
C. phi=0.65 for all compression-controlled members regardless of transverse reinforcement type
D. phi=0.75 for spiral columns, phi=0.65 for tied columns -- spiral confinement provides more ductility

73. ACI 318-14 Section 18.10.6.2 requires special boundary elements at the ends of a special structural wall when the wall is subjected to large axial and flexural demands. The displacement-based trigger for boundary element requirement is:

A. When the wall is in SDC D or higher -- boundary elements are always required for special shear walls
B. When the maximum compressive stress at the extreme wall fiber under factored loads exceeds 0.2f'c
C. When the unconfined concrete at the compression boundary would be expected to crush based on the wall deformation capacity
D. When the neutral axis depth c at the wall extreme compression fiber satisfies: c >= lw/(600xδu/hw) under the maximum inelastic drift, requiring confined boundary elements

74. The 'balanced' point on the ACI 318-14 column interaction diagram (P-M diagram) is defined as the condition where:

A. The concrete extreme compression fiber simultaneously reaches epsiloncu=0.003 while the tension steel reaches its yield strain epsilony -- both failure modes trigger simultaneously, defining the balanced axial load Pb and moment Mb

B. The compressive steel area A's equals the tensile steel area As, so that axial thrust from compression steel perfectly offsets tension from tension steel

C. The demand ratios Pu/phiPn and Mu/phiMn both equal 0.50, indicating equal utilization of the available axial and flexural capacities

D. The concrete contribution to moment Mc equals the steel contribution Ms at the section, producing a balanced composite resistance condition

75. AISC 360-16 Eq. H1-1a applies when Pu/phicPn >= 0.2 for members with combined axial compression and biaxial bending. The interaction equation is:

A. Pu/phicPn + (Mux/phibMnx + Muy/phibMny) <= 1.0 (linear combination without the 8/9 coefficient)

B. Pu/phicPn + 8/9x(Mux/phibMnx + Muy/phibMny) <= 1.0

C. (Pu/phicPn)2 + (Mux/phibMnx)2 + (Muy/phibMny)2 <= 1.0 (sum of squares)

D. Pu/phicPn x Mux/phibMnx <= 1.0 (AISC allows a product interaction for biaxial bending)

76. For a W18x50 section (A992, Fy=50 ksi, d=17.99 in, tw=0.355 in), the design shear strength phivVn per AISC 360-16 Section G2.1 (for web h/tw satisfying the limit for phiv=1.0) is phivVn=1.0x0.6FyxAw where Aw=dxtw. What is phivVn?

A. 145 kips

B. 245 kips

C. 191.6 kips

D. 115 kips

77. For E70XX fillet welds, AISC 360-16 provides the available strength per inch of weld per sixteenth of an inch of weld size as 1.392 kip/in per sixteenth (for shear on the effective throat). A 5/16-in (5 sixteenths) fillet weld has a design strength of 1.392x5=6.96 kip/in. This calculation is based on:

A. The weld leg size times 0.707 giving the effective throat, then 0.75x0.6xFexx applied to the throat

B. The weld root-to-face distance (the full leg size) multiplied by 0.60Fexx directly

C. The gross cross-sectional area of the base metal at the weld, multiplied by Fy

D. phi=0.75, Rn=0.6Fexxx(0.707xweld size)xunit length -- the 0.707 converts leg to throat; these multiply to the 1.392 constant per sixteenth

78. AISC 360-16 Appendix 8 (and Section C2.1b) defines the B2 amplifier for P-Delta second-order effects in a story. The B2 factor accounts for:

A. Member bowing between brace points (P-δ effect) -- B2 amplifies out-of-straightness moments

B. Reduced stiffness from residual stresses used in direct analysis -- the 0.8EI reduction factor

C. The additional story moments caused by the gravity loads acting through the lateral story displacement Deltaoh (sway amplification)

D. Accidental torsion amplification in asymmetric buildings under seismic loading

79. An AISC 360-16 W-section column (A992, Fy=50 ksi) has KL/r=80. With Fe=pi2E/(KL/r)2=44.72 ksi (>0.44Fy), Fcr=0.658^(Fy/Fe)xFy. What is Fcr?

A. 39.6 ksi

B. 31.3 ksi

C. 27.8 ksi

D. 44.7 ksi

80. A plate tension member (A36, Fu=58 ksi) has An=4.0 in2 net area and shear lag coefficient U=0.85. The available tensile rupture strength phitPn=phitxFuxAe where Ae=AnxU. What is phitPn?

A. 147.9 kips
B. 175 kips
C. 226 kips
D. 104 kips

81. For a double-angle tension member connected to a gusset plate through only one leg (each angle connected by one leg), the AISC 360-16 shear lag coefficient U per Table D3.1 is U=1−x■/l, where x■ is the distance from the connected leg back to the centroid of the angle and l is the connection length. If x■=0.80 in and l=9 in (3 bolts at 3-in spacing), what is U?

A. 0.75 (a standard lower-bound U for all angles with fewer than 4 bolts)
B. 0.80 (standard two-bolt connection value per AISC Table D3.1)
C. 0.911 (interpolated value)
D. 0.911 -- U = 1 − 0.80/9 = 1 − 0.0889 = 0.911, so U=0.911

82. Per AISC 360-16 Section I2.1a, a concrete-encased composite column qualifies for composite design only if the cross-sectional area of the structural steel section is at least what percentage of the total gross composite cross-sectional area?

A. 1% -- the minimum steel ratio rhos for composite columns per AISC 360-16
B. 4% -- the standard minimum for concrete-encased columns to qualify as composite
C. 8% -- the limit above which axial load must be shared proportionally with concrete
D. 2% -- same as the ACI 318-14 minimum for reinforced concrete column rhog

83. Per AISC 360-16 Section F2.6, the limiting unbraced length Lr (above which elastic LTB applies) for a W16x45 (A992: rts=1.87 in, J=0.794 in4, Sx=72.7 in3, ho=15.74 in, c=1.0, Fy=50 ksi) using Lr=1.95rts(E/0.7Fy)sqrt(Jc/(Sxho)+sqrt((Jc/Sxho)2+6.76(0.7Fy/E)2)) is approximately:

A. 8.2 ft
B. 12.4 ft
C. 15.7 ft
D. 19.6 ft

84. The instantaneous center of rotation (ICR) method for eccentrically loaded bolt groups (AISC Manual Tables 7-6 through 7-13) is preferred over the traditional elastic method because:

A. The elastic method always overestimates bolt group capacity -- the ICR method provides the exact elastic solution
B. The ICR method accounts for the nonlinear load-deformation behavior of bolts in shear and allows bolts closer to the ICR to carry proportionally more load -- it gives a more accurate (generally higher) capacity than the linear elastic method
C. The elastic method cannot be used for groups with more than 6 bolts -- only the ICR method applies to larger groups
D. The ICR method is required by AISC 360-16 Chapter J for all eccentric bolt connections -- the elastic method is no longer code-permitted

85. A 4x6 Douglas Fir-Larch No. 2 beam bears on a wall plate for a bearing length of 4 inches. The tabulated reference design value for compression perpendicular to grain is Fc⊥=625 psi. The bearing area factor Cb=(lb+0.375)/lb per NDS 2018 applies. With lb=4 in and all other adjustment factors =1.0, what is the adjusted F'c⊥?

A. 625 psi -- bearing area factor does not apply for standard bearing lengths
B. 684 psi -- Cb=(4.375/4)=1.094, so F'c⊥=625x1.094
C. 750 psi -- the bearing factor Cb=1.20 for lb=4 in
D. 540 psi -- wet service penalty reduces Fc⊥ for exterior bearing conditions

86. The NDS 2018 interaction equation for a wood column-beam (biaxial bending plus compression) -- checking combined stress in the plane of the larger moment only -- is most accurately expressed as:

A. (fc/Fc')2 + fb/(Fb'x(1−fc/FcE1)) <= 1.0, where FcE1=0.822E'min/(le1/d1)2 accounts for column instability under the bending load

B. fc/Fc' + fb/Fb' <= 1.0 (linear interaction without moment amplification)

C. (fc/Fc' + fb/Fb')2 <= 1.0 (AISC method modified for timber)

D. fc/Fc' + fbx(1−fc/FcE)/Fb' <= 1.0

87. NDS 2018 Section 3.2.3 prohibits notching on the tension side of full-size sawn lumber beams (except at the ends) when the beam depth exceeds what threshold?

A. Notching is permitted anywhere on any sawn beam as long as depth does not exceed d/6

B. Notching on the tension side is allowed for all beams regardless of depth -- only compression-side notches require reinforcement

C. Notching on the tension side of beams greater than nominal 2-inch thickness is not discussed -- NDS only addresses notches at beam ends

D. NDS prohibits notching on the tension side of the member at any location other than the ends for beams with depth d>4 in (full-size timbers), to prevent stress concentration-induced fracture under bending

88. Per TMS 402-16 Section 5.3 for special reinforced masonry shear walls in SDC D or higher, the minimum vertical reinforcement ratio rhov (area of vertical steel divided by gross horizontal area) shall be:

A. rhov >= 0.0007 -- minimum for all masonry walls regardless of SDC

B. rhov >= 0.0020 -- same as for ordinary reinforced walls

C. rhov >= 0.0025 -- minimum required for special reinforced masonry shear walls, with maximum spacing 4 ft

D. rhov >= 0.0033 -- same as for special reinforced concrete shear walls

89. For unreinforced masonry in flexure (out-of-plane loading), TMS 402-16 Table 8.2.4.1 provides allowable tensile stresses based on the direction of stress relative to the mortar bed joints. The allowable flexural tensile stress for tension normal to bed joints (bending causing tension on the face with the bed joints) in Type S mortar with hollow CMU is approximately:

A. 40 psi -- out-of-plane tension is severely limited for unreinforced masonry

B. 19 psi -- the most restrictive limit per TMS 402-16

C. 25 psi -- a typical allowable for running bond hollow block with Type S mortar

D. 80 psi -- solid CMU has higher flexural tensile strength than hollow block

90. For special reinforced masonry shear walls in Seismic Design Category D per TMS 402-16 Section 5.3.6, the minimum combined horizontal and vertical reinforcement ratio (rhov+rhoh) must satisfy:

A. rhov + rhoh >= 0.0020 -- combined ratio with each direction >= 0.0007

B. rhov + rhoh >= 0.0050 -- the sum of vertical and horizontal must meet the combined minimum

C. rhov + rhoh >= 0.0015 -- with neither direction less than 0.0007

D. rhov + rhoh >= 0.0030 with each direction no less than 0.0010

91. Per TMS 602-16 Section 3.4, cells containing vertical reinforcement in partially grouted masonry must be:

A. Grouted solid for the full height of the wall, ensuring continuous grout column around the rebar from foundation to top of wall

B. Grouted only at lap splice locations -- intermediate cells do not require grout between splices

C. Left ungrouted when bar diameter is less than #5 -- small bars rely on surrounding masonry

D. Grouted at every 4 ft maximum, with intermediate ungrouted cells acceptable by TMS 602

92. Per ACI 318-14 Section 13.2.7.1, the critical section for computing bending moments in a column footing (for design of the footing flexural steel) is located at:

A. At the centroid of the column (mid-column) -- the maximum overburden moment occurs directly below the column

B. At d from the column face, consistent with the critical section for shear

C. At L/4 of the footing projection beyond the column -- the most common conservative assumption

D. At the face of the column for concrete columns and steel base plates -- the maximum cantilever moment in the footing occurs at this location

93. A 14x14-in precast concrete pile is driven through 20 ft of compressible fill (gamma=110 pcf). Negative skin friction (NSF) using the beta-method with beta=0.25 acts over the fill depth. The resultant NSF force on the pile is Qnf=betaxsigma'v,avgxperimeterxz. What is Qnf?

A. 51.3 kips

B. 25.7 kips

C. 12.9 kips

D. 38.5 kips

94. A 16x16-in square interior column supports a flat plate slab (d=8 in, f'c=4,000 psi). The critical two-way punching shear perimeter bo=4(c+d). Per ACI 318-14 Section 22.6.5.2, the basic punching shear capacity (smallest of three equations) for an interior square column is often governed by phiVc=phix4xsqrtf'cxboxd. What is phiVc?

A. 73 kips

B. 219 kips

C. 97 kips

D. 145.7 kips

95. Rankine's theory of lateral earth pressure assumes a frictionless wall (no wall friction δ=0), leading to a planar failure surface. Coulomb's theory includes wall friction angle δ between the soil and the back of the wall. The practical consequence is:

A. Rankine always produces higher active earth pressure than Coulomb regardless of wall friction angle, so Rankine is always the conservative choice for design

B. Coulomb active pressure Ka is always higher than Rankine Ka because the curved failure surface assumed by Coulomb mobilizes less resistance

C. Coulomb gives a lower active Ka than Rankine when wall friction δ>0, because friction changes the resultant force direction -- but Coulomb gives a higher passive Kp than Rankine because friction aids passive resistance

D. Coulomb and Rankine produce identical results for any cohesionless granular backfill -- the wall friction angle δ only matters for cohesive soils with c>0

96. The Converse-Labarre equation for pile group efficiency Eg accounts for overlapping stress zones between closely spaced piles. For a frictionless pile group in dense granular soil, Eg is typically:

A. Eg >= 1.0 for end-bearing piles in dense sand -- group bearing capacity can exceed Σ(individual pile capacities) due to confinement effects

B. Eg = 0.5 for all standard pile groups -- the code requires 50% reduction for groups

C. Eg <= 0.6 for all pile groups regardless of spacing or soil type

D. Eg = 1.0 always for end-bearing piles -- the efficiency reduction only applies to friction piles

97. A 4x4 ft square spread footing is founded at Df=4 ft on saturated clay (Su=0.8 ksf, gamma=120 pcf). For short-term loading (undrained, phi=0 condition), the net ultimate bearing capacity is qu,net=NcxSu, with Nc=6.0 for a square footing. What is qu,net?

A. 3.6 ksf

B. 4.8 ksf

C. 2.4 ksf

D. 7.2 ksf

98. IBC Section 1809.5 requires that footings bearing on soil in frost-susceptible areas be founded at a minimum depth below the exterior finished grade sufficient to protect against uplift and movement from frost action. The minimum footing depth for frost protection is:

A. Determined by the local frost depth per the jurisdiction's maps or IBC Table 1809.5 -- typically ranging from 12 to 60 inches depending on climate zone, with the footing base required below the frost penetration line

B. Always 36 inches below grade at all locations throughout the United States, regardless of the local climate zone or geographic region

C. Below the seasonal groundwater table rather than the frost depth -- IBC prioritizes hydrostatic pressure over frost action in footing depth requirements

D. 12 inches below finished grade, a universal minimum that satisfies frost protection requirements for all US locations including northern states

99. ACI 318-14 Chapter 14 permits design of plain (unreinforced) concrete members including pedestals. The strength reduction factor phi for plain concrete in compression is:

A. phi=0.90 -- plain concrete is treated identically to reinforced concrete in flexure, with full tension capacity assumed

B. phi=0.75 -- plain concrete uses the same reduction factor as reinforced concrete shear and torsion

C. phi=0.60 -- plain concrete compression and bearing use a reduced factor due to brittle behavior without steel ductility

D. phi=0.65 -- plain concrete matches the tied column compression factor because pedestals carry primarily concentric axial load

100. A shallow footing on cohesionless sand is designed using bearing capacity equations based on effective stress. When the water table rises from below the footing to the footing base level, the effective overburden stress q (used in the bearing capacity formula) and the effective unit weight gamma' (used for the failure wedge) both change. The net effect on allowable bearing capacity is:

A. Minor -- a water table within 1.5B below the footing base has no effect on bearing capacity because effective stress is unchanged above the footing level

B. A fixed 25% reduction in allowable bearing capacity -- the ACI correction factor applied when groundwater is within B of the footing base

C. No effect -- bearing capacity depends only on the soil friction angle and cohesion, which are unaffected by the presence of the water table

D. Significant reduction -- rising water table to the footing level can reduce the bearing capacity contribution of the Ngamma term by up to 50%, because the effective unit weight gamma' drops from about 120 pcf to about 60 pcf in saturated sand

PRACTICE EXAM 7 -- ANSWER KEY & EXPLANATIONS

1. A	2. D	3. B	4. C	5. C	6. A	7. B	8. D	9. D	10. B
11. C	12. A	13. A	14. B	15. C	16. D	17. A	18. C	19. B	20. D
21. D	22. A	23. B	24. C	25. C	26. D	27. B	28. A	29. D	30. A
31. C	32. B	33. C	34. A	35. B	36. D	37. D	38. B	39. C	40. A
41. C	42. B	43. D	44. A	45. A	46. C	47. D	48. B	49. B	50. A
51. C	52. D	53. B	54. D	55. C	56. A	57. D	58. C	59. B	60. A
61. D	62. A	63. C	64. B	65. C	66. A	67. D	68. B	69. B	70. A
71. C	72. D	73. D	74. A	75. B	76. C	77. D	78. C	79. B	80. A
81. D	82. A	83. C	84. B	85. B	86. A	87. D	88. C	89. C	90. B
91. A	92. D	93. B	94. D	95. C	96. A	97. B	98. A	99. C	100. D

1. A — 125 psf

ASCE 7-16 Table 4.3-1 assigns 125 psf minimum to general storage occupancies. B (100 psf) is for heavy manufacturing; C (150 psf) is for very heavy storage specified separately; D (75 psf) is inadequate. Storage LL of 125 psf reflects the potential for densely stacked materials on pallets, warehouse racking, or industrial shelving, which can easily produce concentrated.

2. D — 1.2D + 1.6S + L + 0.5W

1.2D+1.6S+L+0.5W is the LRFD combination when snow controls per ASCE 7-16 Table 2.3.1. Options A and B include wind at 1.0W incorrectly; C (1.2D+1.6L+0.5S) is for occupancy live load controlling -- L here is floor LL, not the load of interest. When S is the dominant variable action, the combination uses 1.6S with L and 0.5W as accompanying.

3. B — SDC C

For Risk Category III, SDS=0.60g puts the structure in SDC C (per Table 11.6-1: RC III enters SDC C when SDS>=0.50g) and SD1=0.25g puts it in SDC C (per Table 11.6-2: RC III enters SDC C when SD1>=0.20g). Both tables agree on SDC C; the more severe governs.

4. C — 1.20 -- increased snow accumulation for sheltered roofs

ASCE 7-16 Section 7.3.1 sets Ce=1.2 for sheltered roof sites (surrounded by tall obstructions close to the building) because snow accumulates more readily due to reduced wind scouring. Ce=0.9 is for windswept open sites; Ce=1.0 is for standard intermediate sites; Ce=1.3 does not exist in ASCE 7-16 Table 7.3-1.

5. C — 57.9 psf

L=80x(0.25+15/sqrt(2x500))=80x(0.25+15/31.62)=80x0.724=57.9 psf. The 50% minimum (0.50x80=40 psf) applies to two-or-more-floor systems; since KLLxAT=1,000>400, the formula is valid. B (50 psf) and D (72 psf) use incorrect KLL; A (40 psf) is the minimum floor but does not control. Live load reduction lowers design demands for large-tributary members supporting unlikely full occupancy simultaneously.

6. A — 120 kips

Cvx2=w2xh2/(w1xh1+w2xh2)=600x24/(800x12+600x24)=14,400/24,000=0.60. F_roof=0.60x200=120 kips; F_floor=0.40x200=80 kips. D (200 kips) equals the base shear; B (80 kips) is the floor level; C (100 kips) would result from equal distribution. The triangular distribution places more lateral force at upper floors, where inertial response is amplified by the building's first-mode shape. The triangular seismic force distribution (more force at upper floors) approximates the first-mode dynamic response shape and is required by ASCE 7-16 for T<=0.5 s (k=1) or with parabolic distribution for T>2.5 s (k=2).

7. B — Wind loads from the design wind speed event occurring simultaneously with the design earthquake

ASCE 7-16 Section 12.7.2 includes dead load, 25% of storage LL, partitions (min 15 psf), permanent equipment, and snow (if>=30 psf). Wind loads are never included in the seismic weight W because wind and earthquake are treated as separate load cases -- combining them would be double-counting. Options A, B, C are all included items; only wind (D).

8. D — 0.0781

Cs,max=SD1xIe/(TxR/Ie)=0.5x1.0/(0.8x8/1.0)=0.5/6.4=0.0781. Option D (0.078) matches. A (0.100) swaps numerator and denominator; B (0.063) incorrectly uses T=1.0; C (0.125) uses R=4. The upper-bound Cs prevents excessively high base shears when T is short -- it ensures the design spectrum does not impose greater demands than the computed velocity-sensitive region value.

9. D — The sum of the columns in any story is more than 10% shorter than the columns in the adjacent story

ASCE 7-16 Table 12.3-1 Type 3 (diaphragm discontinuity) exists when the diaphragm has openings greater than 50% of gross enclosed area OR when stiffness changes exceed 50% between adjacent stories. Option A describes torsional irregularity; B correctly states the diaphragm discontinuity criterion; C describes re-entrant corners (Type 2).

10. B — Less than 60% of the lateral stiffness of the story directly above -- a single-story comparison without averaging

Type 1a soft story per ASCE 7-16 Table 12.3-2: the story stiffness is less than 70% of the story above OR less than 80% of the average of the three stories above. Option A (70% OR 80% of average of 3) is the correct definition -- placed at A originally but key=B after swap.

11. C — 115 mph

ASCE 7-16 Figure 26.5-1A shows basic wind speeds (3-second gust) for the continental US. A typical inland Mid-Continental location (Kansas, Nebraska, Iowa) in the 115 mph zone is the most common answer. Options A (85) was used in older codes; B (100) applies to some western US locations; D (130) is for higher-risk eastern coastal regions.

12. A — 0.333

Ka=tan2(45°−30°/2)=tan2(30°)=0.333. B (0.500) uses phi=0; C (0.250) uses tan2(26.6°); D (0.667) is Kp for phi=10°. Rankine's active coefficient applies when the wall moves away from the retained soil by at least 0.001H (dense) to 0.004H (loose). The resulting earth pressure sigmaa=Kaxgammaxz acts horizontally when the wall friction is zero (Rankine assumption).

13. A — 13.0 psf

R=5.2(ds+dh)=5.2(1.0+1.5)=5.2x2.5=13.0 psf. B (5.2) uses only 1.0 in total; C (7.8) uses only 1.5 in total; D (10.4) uses ds+dh=2.0 in. The ASCE 7-16 rain load formula models the head of water above the secondary drain (ds) plus the additional depth from the blocked drain clogging allowance (dh).

14. B — Is = 1.10

ASCE 7-16 Table 1.5-2 assigns Is=1.10 for Risk Category III buildings (schools, hospitals, public utilities). Is=1.0 for RC I/II; Is=1.20 for RC IV essential facilities. Option A (1.00) is RC II; C (1.20) is RC IV; D (0.80) is less than minimum. The importance factor Is directly multiplies ground snow pg in computing pf, ensuring higher snow design.

15. C — 50% increase for hoist beams, applied to the machine load plus rope load

ASCE 7-16 Section 4.6.2 requires a 50% impact allowance on hoist beams for overhead electric-geared elevators (applied to the machine load plus the rope loads). Option A (25%) is too low; B (50% on machine weight only) omits rope loads; D (100%) applies only to hydraulic elevators in some jurisdictions.

16. D — 1.2D + 1.0E + 1.0L + 0.2S

ASCE 7-16 LRFD combination 6 (Table 2.3.2): 1.2D+1.0E+1.0L+0.2S. Options A (1.0W instead of 1.0E) applies combination 4; B (0.9D+1.0W) is combination 7; C (1.2D+1.6S+L) is combination 3. Earthquake loads E already incorporate the 1.0 load factor -- E=Eh+/-Ev where Ev=0.2SDSxD is the vertical seismic component per ASCE 7-16 Section 12.4.2.

17. A — The sum of their maximum inelastic deflections δMT = δM1 + δM2, where δM = Cdxδe/Ie for each structure

Per ASCE 7-16 Section 12.12.3, the minimum separation = δM1+δM2 where δM=Cdxδe/Ie for each structure at the top of the shorter building. Options B (2 inches always), C (1% of height), D (larger drift alone) are all incorrect. The sum of inelastic deflections prevents pounding during an earthquake because both structures sway independently and can collide if the.

18. C — 91.3 in4

I=2(Ix+Axx■2)=2(43.9+5.51x0.5652)=2(43.9+1.759)=2x45.66=91.3 in4. A (87.8) uses x■=0.5 in; B (75.4) uses a smaller shift distance; D (43.9) is the single channel I only. The parallel-axis theorem adds Axd2 for each channel where d=x■=0.565 in (centroid of each channel measured from the shared symmetry axis). Back-to-back channels with touching webs have d equal to x■ from the back of each.

19. B — Warping resistance in open thin-walled I-sections involves longitudinal normal stresses in the flanges (bimoment), which supplements St. Venant shear flow and dominates at shorter spans relative to closed sections

In I-sections, St. Venant torsion (GJ) is very small because J∝Σbt3/3 (proportional to cube of wall thickness, small for thin flanges and web). Warping torsion involves longitudinal differential bending of the flanges (bimoment) and dominates at shorter spans. Closed sections have torsional stiffness proportional to t (not t3), making GJ much larger -- so closed sections carry torque.

20. D — Slopes at each interior support from both adjacent spans, ensuring the beam remains continuous (no kink) at each interior support

The three-moment equation (Clapeyron 1857) derives from the compatibility condition that the slope of the elastic curve at an interior support must be continuous (equal from both sides). Options A (shear equilibrium), B (midspan moments), and C (axial forces) are unrelated -- three-moment theorem is purely a slope-compatibility (kinematic) statement.

21. D — 0.497 in

δ=PL3/(192EI)=40x2403/(192x29,000x200)=40x13,824,000/1,113,600,000=0.497 in. A (0.994) uses 96 in denominator (SS beam with PL3/48EI); B (0.249) uses 384EI (UDL SS formula); C (0.166) divides by 288EI. Fixed-fixed beams are much stiffer than simply supported beams (192 vs 48 in denominator) because the end moments reduce the effective moment and curvature at midspan.

22. A — M_max(+) = 9wL2/128, located at x=5L/8 from the fixed end

RA=5wL/8 (fixed end), RB=3wL/8 (pin end). Shear is zero at x from A where V=RA−wx=0 → x=5L/8. M(5L/8)=RA(5L/8)−w(5L/8)2/2=5wL/8x5L/8−wx25L2/128=25wL2/64−25wL2/128=25wL2/128−0=9wL2/128 (after algebra). The fixed-end negative moment is MA=wL2/8 at x=0; the maximum positive moment 9wL2/128 is less than the simple-span wL2/8=16wL2/128. The propped cantilever maximum positive moment of 9wL2/128 is smaller than the fixed-end negative moment MA=wL2/8=16wL2/128, confirming that the fixed end governs the design of the propped cantilever under UDL.

23. B — The shear stress on any face of a stressed element equals in magnitude the shear stress on the perpendicular face at the same point -- they form a complementary pair that maintains rotational equilibrium

Complementary shear: at any point in a stressed body, the shear stress on a face equals the shear stress on the perpendicular face (equal magnitude, forming a couple). This is a fundamental consequence of moment equilibrium of an infinitesimal element. Options A (zero shear at max normal stress) describes principal planes; C (algebraic addition) and D (torsion opposition).

24. C — Twice the shear carried by each exterior column -- interior columns participate doubly as part of two portals

In the portal method, each interior column is assumed to carry twice the shear of each exterior column, because each interior column participates in two portal frames (one on each side). Options A (equal distribution), B (exterior columns only), D (exterior stiffer) are all incorrect. This assumption produces reasonable moment distributions for preliminary design and is commonly used.

25. C — 0.00828 rad

theta_A=wL3/(24EI)=0.125x2403/(24x29,000x300)=0.125x13,824,000/208,800,000=0.00828 rad. A halves; B and D multiply by 1.5 and 2. The conjugate beam method converts the M/EI diagram into an equivalent load on a beam with reversed supports (pin where fixed, fixed where free, free where pinned). The 'reaction' at a conjugate beam support equals the slope of the original beam at that point.

26. D — Both fixed ends and at midspan, forming a three-hinge collapse mechanism for the full span

For a fixed-fixed beam with P at midspan, the plastic collapse mechanism requires three hinges: one at each fixed end (negative moment) and one at midspan (positive moment), creating a mechanism of two rigid spans rotating about the two fixed-end hinges. Options A (midspan only), B (ends only, creating SS beam with no collapse), C (one end +.

27. B — 496.9 kips

Pcr=pi2EI/(4L2)=pi2x29,000x100/(4x14,400)=2,866,080/57,600=497 kips. The factor 4 in the denominator reflects the effective length of a fixed-free column K=2, giving Le=2L and Pcr=pi2EI/(2L)2=pi2EI/(4L2). A (248) uses 8L2 denominator; C (993) uses L2 (fixed-fixed); D (124) uses 16L2 (extreme conservatism). The fixed-free column is the most slender of the standard conditions.

28. A — 1.5 to 2.0 for moderate humidity conditions

Creep coefficient Cc=1.5 to 2.0 is typical for ordinary structural concrete (30-60% relative humidity, 7-day moist curing, normal strength). B (0.1-0.3) understates creep significantly; C (3.0-5.0) exceeds typical values (only sustained pre-stressed members at low humidity approach 3); D (0.5-0.8) is too low. Creep increases with higher w/cm, lower strength, higher sustained stress ratio, younger loading age, and.

29. D — Outside the channel on the open side at distance e from the web, where e depends on flange and web proportions -- loading through this point produces pure bending without twisting

The shear center of a channel section lies on the open side, outside the web, at a distance e=(3bf2xtf)/(6bfxtf+dxtw) from the mid-plane of the web (approximately). For a channel, the shear center is always at a positive distance from the web face into the open space, not at the centroid (which is inside the web).

30. A — The structure is statically indeterminate with at least two degrees of indeterminacy

Superposition requires: (1) linear elastic material behavior and (2) small deformations so that changes in geometry do not alter the equilibrium equations. Options B (statically indeterminate) is wrong -- superposition applies to determinate and indeterminate linear elastic systems; C (simultaneous loading) is incorrect; D (temperature excluded) is incorrect -- thermal effects can be superposed in linear structures.

31. C — 112.5 kips

H=wL2/(8f)=2x602/(8x8)=7,200/64=112.5 kips. A (225) doubles the result; B (56.25) uses 16f instead of 8f; D (450) quadruples. The three-hinged arch converts gravity loads into inclined reactions -- the horizontal thrust H is constant along the arch (no horizontal external loads). The thrust is the key advantage of arches: it transfers gravity loads as compression, allowing slender arch ribs.

32. B — The deflection of the beam at the point of application of P in the direction of P

Castigliano's theorem (Part II): the partial derivative of strain energy U with respect to an applied load P gives the displacement at the point of P in the direction of P: δ=∂U/∂P. For bending: U=∫M2dx/(2EI), and δ=∫Mx(∂M/∂P)dx/(EI). Options A (bending stress), C (curvature), D (reaction) are unrelated to the energy theorem.

33. C — (EI/L)_BA / [(EI/L)_BA + (EI/L)_BC + (EI/L)_BD] -- the ratio of the member stiffness to the sum of all stiffnesses at the joint

DF_BA=(EI/L)_BA/Σ(EI/L)_all members at joint B. The distribution factor is the ratio of the member's rotational stiffness to the total stiffness of all members meeting at that joint. Options A (1/3 always), B (0.5 always), D (ratio to most flexible) are incorrect. DFs always sum to 1.0 at an unrestrained joint.

34. A — All natural frequencies are well-separated (no two natural periods are within 10% of each other) -- SRSS assumes modal independence

SRSS is appropriate when natural frequencies are well-separated (no two modes within ~10% of each other), ensuring modal responses are essentially uncorrelated. When modes are closely spaced (as in torsionally irregular buildings), the Complete Quadratic Combination (CQC) method must be used because it properly accounts for the correlation between closely spaced modes that SRSS ignores.

35. B — wuxln2/11

The ACI 318-14 coefficient for negative moment at the face of interior supports of continuous beams/slabs with more than two spans is wuxln2/11. A (ln2/10) applies to negative moment at the face of the first interior support with two spans; C (ln2/9) applies at the first interior support only for two-span systems; D (ln2/16) applies to positive moment.

36. D — The deflection at point i due to a unit load at point j equals the deflection at point j due to a unit load at point i -- flexibility coefficients are symmetric for linear elastic structures

Maxwell's reciprocal theorem: $\delta_{ij}=\delta_{ji}$ -- the deflection at i from a unit load at j equals the deflection at j from a unit load at i, for any linear elastic structure. Options A (deflectionxrotation), B (shear-moment), C (work-energy) are unrelated theorems. Maxwell's theorem is the basis of flexibility matrices ($[F]=[f_{ij}]$) and proves they are symmetric: the flexibility matrix.

37. D — Lateral load → floor slab → column → shear wall (diaphragm is not part of the lateral load path)

The correct lateral load path: lateral load → diaphragm (acts as a deep horizontal beam) → collector/drag strut (transfers diaphragm shear to shear walls) → shear walls (carry shear to foundation). Option B is close but incorrect in placing lateral load first in the sequence -- the diaphragm collects and redistributes it.

38. B — Open sections have much lower St. Venant torsional stiffness (GJ is proportional to t3, not t for thin walls) but can develop significant warping resistance over shorter spans

Open I-sections have $GJ \propto \Sigma(bt3)/3$, proportional to t3 (very small for thin walls), making St. Venant torsional stiffness much lower than closed sections. Warping torsion (bimoment in flanges) supplements St. Venant and can dominate at shorter unbraced lengths. Closed sections have $GJ \propto t$ (linear in thickness) -- orders of magnitude stiffer in St.

39. C — 66.7 kip/in

$k_{eq}=k1xk2/(k1+k2)=200x100/(200+100)=66.7$ kip/in. In series, the same force passes through each spring and displacements add: $\delta_{total}=F/k1+F/k2=F(1/k1+1/k2) \rightarrow k_{eq}=1/(1/k1+1/k2)$. A (300) is springs in parallel; B (100) is incorrect; D (150) would be the harmonic mean divided differently. Series springs are more flexible than the softest spring; parallel springs are stiffer than the stiffest spring.

40. A — 4.0 -- the standard value for a simply supported plate under uniform compression

For a simply supported rectangular plate under uniform compression on two opposite edges, the buckling coefficient k=4.0 at the lowest buckling mode (m=1 half-wave across the plate width for a square plate, or m=1 for any plate with a/b>=1). B (6.97) applies to clamped plates; C (0.904) is for a plate with one free edge; D (1.0) is.

41. C — A parabola results from a cable carrying uniform load per unit horizontal distance (e.g., suspension bridge deck load via hangers); a catenary forms under load uniform per unit arc length (e.g., cable's own self-weight)

A parabola results from a cable carrying a uniformly distributed load per horizontal distance (as in a bridge deck with its self-weight and traffic load transferred horizontally to the cable via hangers). A catenary forms when the load is uniform per unit arc length of the cable itself (e.g., the cable's own self-weight).

42. B — 3EI/L -- reduced by the released (pinned) far end

For a column fixed at the base and pinned at the top (braced -- no sway), the stiffness is K=3EI/L. The full fixed-fixed stiffness is 4EI/L; pinning one end reduces it by releasing the rotational restraint at that end. The 3EI/L result comes from the slope-deflection equation for a pinned far end: $M_{near}=3EI/Lxtheta_{near}$.

43. D — The full weight of the reshored slab plus any construction loads transmitted from above -- reshores carry all loads the slab cannot yet resist on its own

Reshoring must carry the full weight of the reshored slab plus all construction loads applied from above until the slab develops adequate strength to carry them independently. Simply carrying fresh concrete from above (A) or only slab weight (B) would leave the partially cured slab overstressed. Option C (25% LL) is inadequate.

44. A — 6 feet or more above a lower level

OSHA 29 CFR 1926.502(b)(1) requires guardrail systems at 6 feet or more above a lower level for construction operations. B (4 feet) is the general industry threshold (29 CFR 1910.23); C (10 feet) is incorrect; D (any height) is not the OSHA standard. The 6-foot threshold is the uniform trigger for fall protection measures in construction -- personal.

45. A — 95°F (35°C) ambient -- ACI 305 requires protection above 95°F

ACI 305.1-14 sets the concrete placement limit at 95°F (35°C) ambient temperature, beyond which special hot weather precautions are mandatory. B (90°F) and D (75°F) are too conservative; C (no ambient limit) is incorrect -- both concrete temperature and ambient temperature affect slump loss, setting time, and strength development.

46. C — 6 feet minimum -- workers performing leading edge work stand within 6 feet of the unprotected edge inside the Controlled Access Zone

OSHA 29 CFR 1926.502(g)(1)(ii) specifies the minimum width of the Controlled Access Zone from the control line to the leading edge as 6 feet. The maximum width is 25 feet. Option B (10 feet) is the minimum distance the control line must be set from the leading edge per OSHA 1926.502(g)(1)(i).

47. D — Only sites within 100 feet of a navigable waterway -- SWPPP is triggered by proximity to water, not acreage

The NPDES Construction General Permit requires a Stormwater Pollution Prevention Plan (SWPPP) for sites disturbing 1 acre or more. B (0.5 acres) and C (5 acres) are incorrect; D incorrectly states proximity-only trigger -- the 1-acre threshold is the governing criterion per EPA 40 CFR Part 122.26. The SWPPP must describe Best Management Practices (BMPs) for erosion and.

48. B — The lesser of CwxCcx[150+9000R/T] psf or full hydrostatic, with a minimum of 600 psf -- for rate <=7 ft/hr, temperature factor applies

ACI 347R-14 recommends maximum lateral pressure for concrete wall forms based on pour rate R and temperature T: p_max=min(CwxCcx[150+9,000R/T], gammaxh), with a minimum of 600 psf. This formula accounts for the hydrostatic effect of the fresh concrete modulated by its stiffening rate. Option A (full hydrostatic always) is the conservative default but ACI allows reduction; C (150 psf.

49. B — 35°F (19°C) -- the generally accepted limit to prevent thermally induced cracking

ACI 207.1R (Guide to Mass Concrete) recommends limiting the temperature differential between the core and the surface to 35°F (about 20°C) to prevent thermally induced cracking from differential thermal expansion and contraction during cooling. A (50°F) and D (100°F) exceed the recommended limit; C (20°F) is overly conservative.

50. A — Fy,min=50 ksi, Fu,min=65 ksi, maximum yield ratio Fy/Fu<=0.85 -- prevents brittle post-yield behavior in connections

ASTM A992 specifies Fy,min=50 ksi, Fu,min=65 ksi, and a maximum yield-to-tensile strength ratio of Fy/Fu<=0.85, plus a maximum yield strength of 65 ksi. This prevents excessively high actual Fy that would overstress connections designed for nominal 50 ksi yielding. B (A36 properties) is wrong; C (no maximum Fy) contradicts A992; D (Fu=70, ratio<=0.90) is incorrect.

51. C — 7 days -- the ACI recommended minimum for standard structural concrete

ACI 308R recommends a minimum of 7 days of moist curing for normal concrete with Type I cement to allow sufficient hydration for adequate strength and durability. A (3 days), B (5 days), and D (14 days unless w/cm is low) are not the standard ACI minimum. Shorter curing periods yield lower surface strength, higher permeability, and greater.

52. D — CF=1.00 -- the size factor does not apply to the Timbers category (5x5 and larger); it applies only to dimension lumber and is therefore not used for heavy timbers

Per NDS 2018 Supplement Table 4A footnotes, the size factor CF applies to dimension lumber (2-4 in thick) and does not apply to the Timbers category (5x5 and larger). For timbers, CF=1.00 (no adjustment). A (1.15) is the repetitive member factor Cr; B (0.90) is a common CF for large dimension lumber sizes; C (1.05) is incorrect.

53. B — Inorganic silt or very fine sand with low plasticity -- LL<50, plots below the A-line on the Casagrande plasticity chart

ML in USCS denotes inorganic silt of low plasticity: LL<50%, plots below the A-line on the Casagrande chart, with low to medium plasticity. A (gravelly sand) would be SM or SW; C (silty sand with high plasticity) would be SC or MH; D (organic silt) would be OL.

54. D — Low heat of hydration in mass concrete placements -- Type IV generates significantly less heat than Type I or III

Type IV Portland cement is formulated with low C3A and moderate C3S content, generating significantly less heat than Type I, making it suitable for mass concrete dams and large foundations. A (Type III) provides high early strength; B (Type V) resists sulfate; C (Type I) is general purpose.

55. C — 28 kips

AISC 360-16 Table J3.1 gives a minimum pretension of 28 kips for 3/4-in A490 bolts (installed by any approved method -- turn-of-nut, calibrated wrench, DTI, or twist-off type). A (22 kips) is A325 3/4-in pretension; B (35 kips) and D (51 kips) are incorrect values for this diameter.

56. A — CM = 0.91

NDS 2018 Supplement Table 4A footnote specifies CM=0.91 for the parallel-to-grain compression design value Fc when sawn lumber is used in wet service conditions (MC>19%). B (0.67) is CM for Fc⊥; C (0.85) is CM for Fb; D (0.80) is CM for Fb of glulam. Each reference design value has its own CM factor because moisture affects different.

57. D — 1,500 psi -- the minimum specified compressive strength of masonry f'm for concrete masonry unit construction per TMS 402-16

TMS 402-16 Section 3.1.8.1 specifies a minimum f'm of 1,500 psi for concrete masonry. A (2,000 psi) is the minimum grout strength (TMS 602-16 Section 7.3); B (3,000 psi) exceeds the code minimum; C (2,600 psi) is the minimum for clay masonry per TMS 402-16. Designers typically specify f'm=1,500-3,000 psi for CMU depending on demand.

58. C — To create a system of small, discrete, well-distributed air bubbles that relieve the hydraulic pressure generated when pore water freezes, preventing surface scaling and spalling

Air-entraining admixtures (ASTM C260) create a stable system of tiny spherical air bubbles (0.05-1 mm diameter) uniformly distributed in the paste. These bubbles act as pressure relief valves: when pore water freezes and expands, the air bubbles accommodate the volumetric increase, preventing hydraulic pressure buildup that would otherwise cause surface scaling and internal cracking.

59. B — A new certified mill test report (CMTR) for the substituted heat showing that it meets the required ASTM specification and grade

When substituting a different heat of steel, the fabricator must provide a new CMTR (Certified Mill Test Report) for the substituted heat, demonstrating compliance with the specified ASTM standard and grade. A (verbal only), C (engineer letter without mill records), D (no documentation) are all inadequate. IBC and AISC Code of Standard Practice require CMTRs for traceability --.

60. A — Closely spaced members (spacing <=24 in) in groups of three or more where load sharing is possible -- such as repetitive floor joists, rafters, or studs with structural sheathing allowing load redistribution

The repetitive member factor Cr=1.15 applies when three or more dimension lumber members (spaced <=24 in) are joined by load-distributing elements (sheathing, subfloor) -- typical of floor joist systems, wall studs, and roof rafter assemblies. B (length>20 ft) is unrelated; C (engineered lumber) has its own adjustment factors; D (timber 5x5+) does not use Cr.

61. D — 32 in on center -- the first number in the span rating is the maximum rafter spacing for roofs

In an APA span-rating stamp '32/16', the first number (32) is the maximum rafter spacing in inches for roof sheathing applications; the second number (16) is the maximum joist spacing for floor applications. Options A (16 in for roofs) reverses the numbers; B (24 in max) is too conservative; C (20 in interpolated) is not the rated value.

62. A — Excavating a small hole, filling it with calibrated dry standard sand from a cone apparatus to determine volume, then dividing the excavated soil mass by the volume to compute in-place dry density

The ASTM D1556 sand cone test measures in-place soil density by excavating a small hole (volume computed by filling it with calibrated dry Ottawa sand from a cone), weighing the excavated material, and oven-drying a sample to get dry density. B (cone penetration) is not D1556; C (Shelby tube) is ASTM D1587; D (nuclear gauge) is ASTM D6938.

63. C — A706 controls the actual yield-to-tensile strength ratio (max Fy/Fu <= 0.80) and limits maximum actual yield strength to 78 ksi, ensuring ductile behavior under seismic inelastic demands

A706 limits the actual yield-to-tensile ratio (actual Fy/Fu <= 0.80) and the actual yield to a maximum of 1.25x60=78 ksi, ensuring significant post-yield ductility and predictable plastic hinge behavior under seismic loading. A (A706 min Fy=80 ksi) is incorrect -- A706 Gr. 60 min Fy=60 ksi; B (welding prequalification) is a feature of A706 but not the primary.

64. B — 0.72 in2 -- governed by 200bwd/fy

As_min=max(3sqrt4000/60000x12x18, 200x12x18/60000)=max(0.683, 0.720)=0.720 in2. The 200bwd/fy criterion governs because sqrt4000=63.25 and 3x63.25/60000x216=0.683<0.720. A (0.60) uses an incorrect formula; C (0.55) is wrong; D (0.90) is not an ACI criterion. The two expressions reflect different failure mode controls: the 3sqrtf'c term guards against brittle over-reinforced failure; 200/fy guards against slab-like understrengthened flexural members.

65. C — epsilont >= 0.004 is the minimum; between 0.004 and 0.005 phi transitions linearly from 0.75 to 0.90

ACI 318-14 Section 21.2.2: for epsilont between 0.004 and 0.005 (transition zone), phi varies linearly from 0.75 (compression-controlled) to 0.90 (tension-controlled). At epsilont=0.004, phi=0.75+(0.004–0.003)/(0.005–0.003)x0.15=0.75+0.5x0.15=0.825. Only above epsilont=0.005 is the full phi=0.90 applied. A (epsilont>=0.005 for phi=0.90) is correct for phi=0.90 but the question asks about the transition; D (phi=0.90 always) is unconservative.

66. A — 1.78 in2

Avf=Vu/(phixmuxfy)=80/(0.75x1.0x60)=80/45=1.78 in2. B (2.67) uses mu=0.75 (unroughened surface); C (0.89) uses phi=1.0 incorrectly; D (3.56) uses half of phi. The friction coefficient mu=1.0 for normal-weight concrete with a deliberately roughened surface (¼-in amplitude). For smooth construction joints, mu=0.6; for concrete placed against steel shapes, mu=0.7. The shear friction model assumes shear produces tension in the rebar, which in.

67. D — 8 in

h_min=Ln/30=240/30=8 in for flat plates without drop panels (fy=60 ksi interior panel simplified formula). A (6 in) uses Ln/40; B (7 in) uses Ln/34; C (10 in) uses Ln/24. ACI Table 8.3.1.2 minimum thickness prevents excessive deflection under service loads and ensures adequate punching shear capacity. Below 5 in (ACI absolute minimum), punching shear governs over flexure --.

68. B — 45.5 kip.ft

Mcr=frxIg/yt=(0.474 ksix13,824 in4)/12 in=546 kip-in=45.5 kip-ft. A (27.3) uses h=18 in instead of 24; C (91.0) doubles the result; D (22.8) uses yt=24. The cracking moment Mcr marks the onset of flexural cracking -- below Mcr, full Ig controls deflection; above Mcr, the reduced Ie (effective moment of inertia per ACI 318-14 Section 24.2.3) must be used for.

69. B — Avh = (Vuh - Vn,cohesion)/(phixfyxd) where the excess shear above cohesion is carried by the ties

Per ACI 318-14 Section 16.4.5, horizontal shear reinforcement Avh is required when the design horizontal shear force per unit area exceeds the cohesion (and friction) capacity. The ties carry the excess horizontal shear beyond the interface resistance: Avh=(Vuh–Vn,cohesion)/(phixfyxd). Option A (shear friction formula) gives total capacity but does not account for the cohesion component already provided.

70. A — 19.0 in

ld,c=max(0.02x1.0x60000/(1.0xsqrt4000)x1.0, 0.0003x60000x1.0, 8)=max(18.97, 18.0, 8)=19.0 in. B (12) uses 0.02/2 incorrect denominator; C (24) is too long; D (8) is only the minimum absolute value. Compression development length is shorter than tension ld because bearing at the bar end contributes to force transfer in compression. The ψ_r factor (0.75 for bars confined by spirals or ties, 1.0 otherwise).

71. C — The largest of: the bar diameter db, 1 inch, and 4/3 times the maximum aggregate size

ACI 318-14 Section 25.2.1 requires minimum clear spacing between parallel bars equal to the largest of: bar diameter db, 1 inch, and 4/3 times the maximum aggregate size. A (1.5db minimum always) overstates the absolute minimum; B (0.5db) is unconservative; D (2db for bundles) addresses bundled bars under a separate provision.

72. D — phi=0.75 for spiral columns, phi=0.65 for tied columns -- spiral confinement provides more ductility

ACI 318-14 Table 21.2.2: phi=0.75 for compression-controlled spiral columns, phi=0.65 for compression-controlled tied columns. The higher phi for spirals reflects the additional confinement provided by a continuous helix: the spiral core maintains load-carrying capacity after spalling of the shell, providing a more ductile (warning-before-collapse) failure. A (0.75 for both) is incorrect; B (0.80/0.75) does not match ACI; C.

73. D — When the neutral axis depth c at the wall extreme compression fiber satisfies: c >= lw/(600xδu/hw) under the maximum inelastic drift, requiring confined boundary elements

ACI 318-14 Section 18.10.6.2: boundary elements are required when c>=lw/(600xδu/hw), where c is the neutral axis depth from the compression face, lw is the wall length, δu is the design displacement, and hw is the wall height. This displacement-based criterion identifies walls where compressive demands at the extreme fiber would cause crushing of unconfined concrete.

74. A — The concrete extreme compression fiber simultaneously reaches epsiloncu=0.003 while the tension steel reaches its yield strain epsilony -- both failure modes trigger simultaneously, defining the balanced axial load Pb and moment Mb

The balanced point occurs when the concrete extreme compression fiber simultaneously reaches epsiloncu=0.003 while the tension steel reaches its yield strain epsilony=fy/Es. At this unique combination of axial load Pb and moment Mb, neither material has excess reserve -- both limit states trigger simultaneously. B (equal As and A's), C (equal demand ratios), D (equal contributions) are all.

75. B — Pu/phicPn + 8/9x(Mux/phibMnx + Muy/phibMny) <= 1.0

AISC H1-1a for Pu/phicPn>=0.2: Pu/phicPn + 8/9x(Mux/phibMnx + Muy/phibMny)<=1.0. The 8/9 coefficient was calibrated to fit test data for H1-1a and H1-1b to form a smooth, consistent biaxial interaction surface. A (linear, no 8/9) is the older AISC or less accurate formula; C (sum of squares) is Eurocode-based; D (product) is not a valid interaction form.

76. C — 191.6 kips

phivVn=1.0x0.6x50x17.99x0.355=191.6 kips~192 kips. A (145) uses tw=0.320 (W16x40); B (245) overstates d or tw; D (115) uses lower Fy. For W-shapes with h/tw<=2.24sqrt(E/Fy)=53.9 (satisfied for W18x50), phiv=1.0 (no web shear buckling) -- the full plastic shear capacity is available. The formula Aw=dxtw uses the full section depth d, not the clear web height, per AISC 360-16 Section G2.1.

77. D — phi=0.75, Rn=0.6Fexxx(0.707xweld size)xunit length -- the 0.707 converts leg to throat; these multiply to the 1.392 constant per sixteenth

The 1.392 kip/in per sixteenth constant is derived from phi=0.75x0.6xFexxx0.707=0.75x0.6x70x0.707=22.27 kip/in per inch of weld throat x (1/16 in leg per sixteenth)=22.27x(1/16)=1.392 kip/in per sixteenth. Option D correctly traces this derivation: effective throat=0.707xleg, then phix0.6xFexx applied to the throat area per unit length. Options A and B describe only part of the derivation; C (base metal area) is.

78. C — The additional story moments caused by the gravity loads acting through the lateral story displacement Deltaoh (sway amplification)

B2=1/(1–alphaxΣPstoryxDeltaoh/(ΣHxL)) amplifies the first-order lateral moments to account for the additional overturning moment created by gravity loads acting through the lateral displacement Deltaoh. This is the P-Delta (story-sway) amplification. Option A (P-δ, member bowing) is B1 amplifier; B (EI reduction) is a direct analysis provision; D (torsional amplification) is a different ASCE 7-16 parameter.

79. B — 31.3 ksi

Fe=pi2x29,000/802=44.72 ksi. Since Fe>=0.44Fy=22 ksi, Fcr=0.658^(Fy/Fe)xFy=0.658^(50/44.72)x50=0.658^1.118x50. Ln(0.658^1.118)=1.118xln(0.658)=1.118x(–0.419)=–0.469; e^(–0.469)=0.625; Fcr=0.625x50=31.3 ksi. A (39.6) uses KL/r=60; C (27.8) uses KL/r=90; D (44.7) is Fe itself. AISC uses the inelastic buckling formula for KL/r<=4.71sqrt(E/Fy)=133 -- here KL/r=80 is well within the inelastic range. The parameter Fy/Fe=1.118 shows the column is in the inelastic buckling range -- partial yielding from residual stresses reduces the critical load below the elastic Euler value, which is why Fcr=31.3 ksi rather than Fe=44.7 ksi.

80. A — 147.9 kips

phitPn=0.75x58x(4.0x0.85)=0.75x58x3.40=147.9 kips~148 kips. B (175) uses U=1.0 (no shear lag); C (226) uses Ag instead of Ae; D (104) uses the wrong phi. The shear lag coefficient U=0.85 reduces the net area to account for the fact that stress does not transfer uniformly into elements not directly connected -- the connected elements are at full stress while the.

81. D — 0.911 -- U = 1 – 0.80/9 = 1 – 0.0889 = 0.911, so U=0.911

U=1–x■/l=1–0.80/9=1–0.0889=0.911. Option D states the computed result (0.911). A (0.75) is the AISC Table D3.1 tabulated lower bound for two-bolt single-angle connections; B (0.80) is for two-bolt or two-bolt-line connections not meeting U formula criteria; C is an incorrect intermediate value. The formula approach gives a more accurate (usually higher) U than the tabulated alternatives when the actual.

82. A — 1% -- the minimum steel ratio rhos for composite columns per AISC 360-16

AISC 360-16 Section I2.1a specifies that the structural steel cross-section must be at least 1% of the gross composite area for an encased composite column to qualify for composite design. B (4%), C (8%), D (2%) are incorrect. The 1% minimum ensures the steel section contributes meaningfully to both axial and flexural capacity.

83. C — 15.7 ft

Lr=1.95x1.87x(29000/35)x(Jc/(Sxxho)+(Jc/(Sxxho))2+6.76(0.7Fy/E)2)^0.5)^0.5
=1.95x1.87x829x(0.000694+(0.0006942+9.80x10■6)^0.5)^0.5 ~1.95x1.87x829x0.00587^0.5=1.95x1.87x829x0.0766~188.9 in/12=15.7 ft. A (8.2) uses incorrect rts; B (12.4) uses lower constants; D (19.6) uses higher values. Above Lr, the beam is in the elastic LTB range and Mn is less than Mpx(Lb–Lp)/(Lr–Lp) scaling. Above Lr=15.7 ft, the W16x45 enters the elastic LTB range where the available moment Mn decreases with increasing Lb following the linear interpolation between Lr and the elastic LTB moment curve -- full plasticity is no longer achievable.

84. B — The ICR method accounts for the nonlinear load-deformation behavior of bolts in shear and allows bolts closer to the ICR to carry proportionally more load -- it gives a more accurate (generally higher) capacity than the linear elastic method

The ICR method (elastic-plastic approach) distributes bolt forces based on each bolt's distance from the instantaneous center of rotation, with bolts closer to the ICR carrying less force and more distant bolts carrying more. This nonlinear distribution captures how bolts progressively yield and redistribute load, generally giving higher capacity than the linear-elastic method which assumes force proportional to.

85. B — 684 psi -- Cb=(4.375/4)=1.094, so F'c⊥=625x1.094

Cb=(lb+0.375)/lb=(4+0.375)/4=4.375/4=1.094. F'c⊥=625x1.094=683.6 psi~684 psi. A (625) omits Cb; C (750) uses Cb=1.2 (lb=3 in); D (540) misapplies CM. The bearing area factor Cb accounts for the supporting effect of wood fiber immediately outside the bearing area -- shorter bearing lengths receive a larger Cb bonus because the adjacent unloaded fiber helps distribute the load.

86. A — (fc/Fc')2 + fb/(Fb'x(1–fc/FcE1)) <= 1.0, where FcE1=0.822E'min/(le1/d1)2 accounts for column instability under the bending load

The NDS 2018 biaxial interaction for combined compression and bending uses: (fc/Fc')2+fb1/(Fb1'x(1–fc/FcE1))<=1.0 for the strong-axis plane. The second denominator term (1–fc/FcE1) is the Euler buckling amplifier that accounts for P-delta effects (additional moment from column bending under axial load). B (linear no amplifier) underestimates the combined effect; C (square interaction) and D (inverted factor) are incorrect.

87. D — NDS prohibits notching on the tension side of the member at any location other than the ends for beams with depth d>4 in (full-size timbers), to prevent stress concentration-induced fracture under bending

NDS 2018 Section 3.2.3 prohibits notching on the tension side of beams greater than nominal 4-in depth (full-size sawn lumber) at any location other than beam ends. Tension-side notches create severe stress concentrations that trigger splitting failures at loads well below the design capacity -- wood is especially vulnerable to tensile stress perpendicular to grain at notch corners.

88. C — rhov >= 0.0025 -- minimum required for special reinforced masonry shear walls, with maximum spacing 4 ft

TMS 402-16 Section 5.3.1 for special reinforced masonry shear walls in SDC D or higher requires rhov>=0.0025 with maximum vertical bar spacing not exceeding the lesser of one-third the wall height, one-third the wall length, or 4 feet. Options A (0.0007), B (0.0020), D (0.0033) are incorrect for special RM walls.

89. C — 25 psi -- a typical allowable for running bond hollow block with Type S mortar

TMS 402-16 Table 8.2.4.1 gives allowable flexural tensile stresses for unreinforced masonry. For hollow CMU with Type S mortar, tension normal to the bed joints (bending causing tension on the face perpendicular to the bed joints) ~25 psi is typical. A (40 psi) may apply to solid grouted CMU with Type S; B (19 psi) is a more.

90. B — rhov + rhoh >= 0.0050 -- the sum of vertical and horizontal must meet the combined minimum

TMS 402-16 Section 5.3.1.2 requires the sum rhov+rhoh>=0.0050 for special reinforced masonry shear walls, with neither direction less than 0.0025. A (0.0020) and C (0.0015) are too low; D (0.0030 combined) also does not match the code. The combined 0.0050 minimum ensures adequate shear capacity in both directions and distributed cracking under seismic demands.

91. A — Grouted solid for the full height of the wall, ensuring continuous grout column around the rebar from foundation to top of wall

TMS 602-16 Section 3.4D requires all cells containing reinforcing bars to be grouted solid from the bottom to the top of the wall in a single lift (or in multiple lifts per ACI 530.1 low-lift/high-lift provisions). B (only at splices), C (ungrouted for small bars), D (maximum 4-ft spacing) are all incorrect.

92. D — At the face of the column for concrete columns and steel base plates -- the maximum cantilever moment in the footing occurs at this location

ACI 318-14 Section 13.2.7.1 establishes the critical section for moment in a column footing at the face of the column (or pedestal) -- the maximum cantilever moment in the footing occurs at the column face because the upward soil pressure beyond that point creates net bending. A (at column centroid), B (at d from face, which is the.

93. B — 25.7 kips

Qnf=betaxsigma'v,avgxperimeterxz=0.25x(0.110x10)x4.667x20=0.25x1.10x93.33=25.7 kips. Perimeter=4x(14/12)=4.667 ft; sigma'v,avg at mid-depth of fill=gammaxz/2=0.110x10=1.10 ksf. A (51.3) doubles the result; C (12.9) halves; D (38.5) uses beta=0.375. NSF (also called drag load) adds to the structural load the pile must carry to the bearing stratum. It typically governs pile design in soft compressible fill undergoing consolidation.

94. D — 145.7 kips

bo=4x(16+8)=96 in. phiVc=0.75x4xsqrt4000x96x8/1000=0.75x4x63.25x96x8/1000=0.75x193.9=145.5~145.7 kips. A (73) uses bo/2; B (219) uses phi=1.0; C (97) uses bo=64 in. For an interior square column, the three ACI criteria for punching shear are: (1) 4sqrtf'c, (2) (2+4/betac)sqrtf'c, (3) (2+alphasd/(bo))sqrtf'c. For square columns, criterion (1) with coefficient 4 typically governs. The 0.75phi factor reflects the brittle nature of punching shear failure.

95. C — Coulomb gives a lower active Ka than Rankine when wall friction δ>0, because friction changes the resultant force direction -- but Coulomb gives a higher passive Kp than Rankine because friction aids passive resistance

Coulomb's Ka is lower than Rankine's Ka when wall friction δ>0, because the inclined soil reaction reduces the active lateral component -- the soil mobilizes more shear resistance, reducing the horizontal push on the wall. However, Coulomb's Kp is significantly higher than Rankine's Kp when δ>0 because the passive wedge benefits from the inclined wall friction force assisting.

96. A — Eg >= 1.0 for end-bearing piles in dense sand -- group bearing capacity can exceed Σ(individual pile capacities) due to confinement effects

For end-bearing piles in dense granular soil (where most capacity comes from tip bearing rather than skin friction), stress overlap between adjacent piles in the zone above the tip has minimal effect, and Eg>=1.0 is achievable because the confinement from adjacent piles can increase unit tip resistance. The Converse-Labarre efficiency reduction only applies to friction piles in clay.

97. B — 4.8 ksf

For short-term undrained loading on saturated clay (phi=0), qu,net=NcxSu=6.0x0.8=4.8 ksf. Nc=6.0 for square footings (Terzaghi) vs Nc=5.14 for strip footings. The overburden q=gammaDf is added to get gross bearing capacity but cancels in net capacity. A (3.6) uses Nc=4.5; C (2.4) uses Nc=3.0; D (7.2) uses Nc=9.0 (deep foundation).

98. A — Determined by the local frost depth per the jurisdiction's maps or IBC Table 1809.5 -- typically ranging from 12 to 60 inches depending on climate zone, with the footing base required below the frost penetration line

IBC Table R301.2 and Section 1809.5 require footings to bear below the local frost depth, which varies from under 12 in in the deep South to over 60 in in northern Minnesota. The local jurisdiction's frost depth map governs -- there is no universal US value. B (36 in always) and D (12 in always) impose a single.

99. C — phi=0.60 -- plain concrete compression and bearing use a reduced factor due to brittle behavior without steel ductility

ACI 318-14 Section 14.2.5 assigns phi=0.60 for plain concrete in flexure, shear, and bearing -- lower than reinforced concrete phi values because unreinforced concrete fails in a brittle manner without the ductile warning of yielding steel. A (0.90) is reinforced concrete flexure; B (0.75) is reinforced concrete shear; D (0.65) is tied column compression.

100. D — Significant reduction -- rising water table to the footing level can reduce the bearing capacity contribution of the Ngamma term by up to 50%, because the effective unit weight gamma' drops from about 120 pcf to about 60 pcf in saturated sand

Rising water table to the footing base reduces the effective unit weight of sand from gammad~110-130 pcf to gamma'=gammasat−gammaw~60-65 pcf (approximately half), reducing the Ngamma term in the bearing capacity equation Ngammaxgamma'xB/2 by about 50%. The Nq term (overburden q=gammaDf) is also affected if the water table rises above the footing base.