PE Civil Structural Exam Prep

Practice Exam 8

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 8 -- QUESTIONS

DOMAIN 1 -- Loads (Q1-Q17)

1. Per ASCE 7-16 Section 26.10, the velocity pressure qz=0.00256xKzxKztxKdxKexV2 psf. For Exposure C at z=15 ft (Kz=0.85), flat terrain (Kzt=1.0), Kd=0.85, Ke=1.0, and V=115 mph, what is qz?

A. 20.1 psf

B. 28.7 psf

C. 24.5 psf

D. 32.0 psf

2. Per ASCE 7-16 Section 12.8.6.2, the P-delta stability coefficient theta=PxxDelta/(VxxhsxxCd) for a given story must not exceed thetamax=0.5/(betaCd)<=0.25 where beta is the ratio of shear demand to capacity. The primary purpose of the P-delta check is to:

A. Ensure that gravity loads acting through the lateral story displacement do not produce additional overturning moments that exceed the structure's lateral force-resisting capacity -- preventing dynamic instability

B. Limit the maximum allowable story drift for occupant comfort during wind events

C. Check that the foundation uplift from combined gravity and seismic overturning does not exceed the dead load stabilizing moment

D. Verify that the column axial load does not exceed the Euler buckling load for the story height

3. Per ASCE 7-16 Table 12.2-1, the response modification coefficient R for a Building Frame System with Special Reinforced Concrete Shear Walls is:

A. R = 4.0

B. R = 5.0

C. R = 8.0

D. R = 6.0

4. Per ASCE 7-16 Section 12.8.4.2, when the fundamental period T of a structure exceeds the long-period transition period TL, the seismic response coefficient Cs is computed using Cs=SD1xTL/(T2xR/Ie). The purpose of the TL cutoff is:

A. To increase the design base shear for very tall buildings in high-seismic zones beyond the standard Cs formula

B. To account for the transition from velocity-controlled to displacement-controlled spectral response at very long periods -- the spectral acceleration decreases as 1/T2 beyond TL rather than 1/T

C. To eliminate seismic design requirements for structures with T>TL because these structures are flexible enough to avoid resonance

D. To limit the minimum Cs value for structures in SDC E and F

5. For SDS=0.80g, R=8, and Ie=1.0, the design seismic response coefficient Cs=SDS/(R/Ie) per ASCE 7-16 Eq. 12.8-2 is:

A. 0.100

B. 0.125

C. 0.050

D. 0.200

6. ASCE 7-16 Section 4.3.3 requires that any partition loading in office buildings -- even when no partitions are shown on the drawings -- be accounted for by using a minimum partition live load of:

A. 10 psf distributed uniformly over the floor area

B. 20 psf distributed uniformly over the floor area

C. 25 psf distributed only over partition locations shown on drawings

D. 15 psf distributed uniformly over the floor area

7. Per ASCE 7-16 Table 12.2-1, the overstrength factor Omega0 for a Special Steel Moment Frame (SMF) is:

A. Omega0 = 2.0

B. Omega0 = 2.5

C. Omega0 = 3.0

D. Omega0 = 1.5

8. Per ASCE 7-16 Section 7.7.1, the height of a snow drift formed on the leeward side of an upper roof against a higher adjacent structure is computed using hd=0.43(lu)^(1/3)(pg+10)^(1/4)–1.5. For an upwind fetch lu=200 ft and ground snow pg=30 psf, what is hd?

A. 3.5 ft

B. 4.8 ft

C. 2.1 ft

D. 6.3 ft

9. For seismic design, the redundancy factor rho per ASCE 7-16 Section 12.3.4 can be either 1.0 or 1.3. A value of rho=1.3 is assigned to structures in SDC D through F when:

A. The structure uses a dual system with moment frames and shear walls -- ASCE 7-16 assigns rho=1.3 to all dual systems in SDC D as a mandatory conservatism

B. There are fewer than two bays of lateral resistance in each orthogonal direction -- rho=1.3 applies as a non-redundancy penalty for narrow buildings

C. The lateral system lacks sufficient redundancy -- removing any single element or connection would reduce story strength by more than 33%, or the structure fails the ASCE 7-16 redundancy conditions

D. The seismic base shear exceeds 0.20W -- high seismic force itself triggers the increased redundancy factor regardless of the number of lateral resistance elements

10. Per ASCE 7-16 Section 2.3.1, the LRFD load combination for uplift or overturning resistance check under wind loads (wind opposing dead load) is:

A. 0.9D + 1.0W -- dead load is reduced to minimum while wind is at full factored value

B. 1.2D – 1.6W -- amplified dead subtracted by amplified wind

C. 0.6D + 1.0W -- dead load reduced to 60%

D. D + W -- unfactored loads without any factors

11. Per ASCE 7-16 Table 4.3-1, the minimum uniformly distributed live load for assembly areas with fixed seats (theaters, auditoriums with bolted-down seating) is:

A. 40 psf

B. 80 psf

C. 100 psf

D. 60 psf

12. Per ASCE 7-16 Chapter 30, wind loads for rooftop equipment and its supports (mechanical units, cooling towers) are computed using GCfxqhxAf where GCf is the force coefficient. The minimum design wind force for any rooftop structure must not be less than:

A. 10 psf on the projected area -- a minimum regardless of computed qh

B. 16 psf on the projected area of the rooftop structure or equipment -- the ASCE 7-16 minimum for rooftop structures

C. The computed value with no minimum -- the formula always governs

D. 100 lb total force -- a minimum for very small equipment

13. ASCE 7-16 Section 12.11.1 specifies that structural walls and their anchorage to floor and roof diaphragms must be designed for a minimum out-of-plane seismic force Fp=0.40SDSxIexWp. For SDS=1.0g, Ie=1.0, and a wall weight Wp=80 psf, Fp is:

- **A.** 32 psf
- **B.** 48 psf
- **C.** 16 psf
- **D.** 64 psf

14. Per ASCE 7-16 Table 12.6-1, the permitted analytical procedures for SDC D structures include the Equivalent Lateral Force (ELF) procedure provided the structure meets certain regularity conditions. Which of the following structures in SDC D is NOT permitted to use ELF?

- **A.** A 4-story regular building with T<3.5Ts
- **B.** A building with a vertical stiffness irregularity (Type 1a soft story) when T>3.5Ts
- **C.** A single-story commercial building with no irregularities
- **D.** A regular 8-story building with T<3.5Ts and no plan or vertical irregularities

15. Per ASCE 7-16 Section 7.10, unbalanced snow loads must be considered on gable roofs where the roof slope is between approximately 2.4° (½:12) and 70° (for warm roofs). On the windward side of a gable under unbalanced loading, the design snow load is:

- **A.** Equal to the full balanced snow load pf
- **B.** 1.5 times the balanced pf value -- windward gets the heavier loading
- **C.** Zero or a reduced value -- wind scours snow from the windward side, depositing it on the leeward side
- **D.** The ground snow load pg applied directly without any roof factor adjustments

16. Per ASCE 7-16 Section 4.3.2, stairways and exit ways (not including fire escapes) in residential or office buildings require a minimum uniformly distributed live load of:

- **A.** 40 psf -- same as the office floor live load
- **B.** 60 psf -- intermediate between residential floor and corridor LL
- **C.** 80 psf -- same as the ground-floor retail live load
- **D.** 100 psf -- same as corridors and lobbies in public assembly buildings

17. When computing the effective seismic weight W for a building with a significant snow load (pg=40 psf), ASCE 7-16 Section 12.7.2 requires including a fraction of the flat roof snow load pf in the seismic weight. The fraction is:

- **A.** 25% of pf is included in W
- **B.** 20% of pf is included in W when pf exceeds 30 psf
- **C.** 50% of pf is always included regardless of magnitude
- **D.** Zero -- snow load is never included in seismic weight

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

18. A W16x40 steel beam (As=11.8 in2, Is=518 in4, d=16.01 in) acts compositely with a 4-in concrete slab (n=8, beff=72 in). The transformed slab width is 72/8=9 in, giving Ac_tr=36 in2. The composite neutral axis from the beam bottom is ■=(11.8x8.005+36x18.01)/(11.8+36). What is the composite moment of inertia (parallel-axis)?

- **A.** 1,456 in4
- **B.** 1,100 in4
- **C.** 518 in4
- **D.** 2,200 in4

19. In a properly braced steel frame (sidesway prevented), the effective length factor K for a column with both ends rigidly connected (fixed-fixed within the braced frame) approaches:

A. K = 1.0 -- the default for all braced columns regardless of end conditions

B. K = 2.0 -- required for all columns in braced frames by AISC 360-16

C. K = 0.7 -- intermediate value for partially restrained ends

D. K = 0.5 for ideal fixed-fixed, but practically taken as K between 0.5 and 1.0 depending on the degree of rotational restraint at each end

20. The conjugate beam method is a graphical/analytical technique for computing beam deflections and slopes. The conjugate beam of a simply supported real beam is:

A. A cantilever beam with a fixed end at the support corresponding to the real beam's pin end, loaded with the M/(EI) diagram as a distributed force

B. A beam with both ends free -- the M/(EI) diagram is applied as a self-equilibrating distributed load system that satisfies internal equilibrium only

C. Also simply supported -- the M/(EI) diagram from the real beam is applied as a distributed load, and the conjugate beam reactions give the real beam's end slopes

D. A fixed-fixed beam where the M/(EI) diagram acts as external load, and the fixed-end reactions give the deflections at the real beam supports

21. For a continuous beam analyzed by the force (flexibility) method, the number of redundant forces to be solved equals:

A. The degree of static indeterminacy -- the difference between total unknown reactions and internal forces at cuts, and the available equilibrium equations

B. The number of supports minus two for any continuous beam -- a simplified formula that approximates the indeterminacy for regular structures only

C. The number of spans minus one -- always exactly, regardless of support conditions, number of internal hinges, or structural configuration

D. The total number of equilibrium equations available for the structure -- three for planar structures, six for three-dimensional space frames

22. The slope-deflection equation for the near-end moment MAB of member AB (with far end B) is MAB=(2EI/L)(2thetaA+thetaB–3Ψ)+FEMAB where Ψ=Delta/L is the chord rotation. For a fixed-fixed beam under uniform load w=3 kip/ft, span L=20 ft, the fixed-end moment FEMAB=wL2/12 is:

A. 120 kip.ft

B. 50 kip.ft

C. 75 kip.ft

D. 100 kip.ft

23. In a truss analysis, a member that carries load in tension under one loading condition can carry compression under a different loading pattern. The structural design consequence is:

A. Tension-only members can be used for all members -- compression never develops in well-designed trusses

B. Members must be designed for the maximum force envelope (both maximum tension and maximum compression) across all applicable load combinations

C. Only the maximum tension force governs -- compression in truss members is always negligible

D. Truss members in compression are always replaced by cables to prevent buckling

24. For a beam with a point load P at its third point (L/3 from support A), the bending moment at the load point is:

A. PL/2 -- same as midspan loading

B. PL/4 -- reduced proportionally to the load position

C. 2PL/9 -- computed from RAxL/3 where RA=2P/3

D. PL/6 -- one-third of the midspan value

25. When using the stiffness (displacement) method, the unknowns solved for are:

A. Joint displacements (translations and rotations) -- the stiffness method solves for kinematic unknowns at each degree of freedom

B. Member end forces -- the stiffness method directly computes internal forces without finding displacements first

C. Support reactions only -- the stiffness method cannot determine member forces

D. Flexibility coefficients -- the stiffness method inverts the force method's flexibility matrix directly

26. For a two-span continuous beam (equal spans L) with a uniform load w on the left span only, the moment at the interior support by the three-moment equation is:

A. –wL2/8 -- same as a fixed-end moment for a single span

B. –wL2/12 -- same as a fixed-fixed beam

C. Zero -- the unloaded span has no effect on the interior support moment

D. –wL2/16 -- the interior support moment for one-span-loaded continuous beam

27. The area-moment theorem states that the tangential deviation of point B from the tangent line drawn at point A on the elastic curve equals:

A. The slope at A times the span length -- δt = thetaA x L for any beam

B. The area of the M/(EI) diagram between A and B, taken about A

C. The first moment (area times centroid distance from B) of the M/(EI) diagram between A and B, taken about B

D. The integral of the shear force diagram between A and B divided by EI

28. A cantilever beam (E=29,000 ksi, I=150 in4, L=8 ft=96 in) carries a concentrated load P=10 kips at the free end. The deflection at the free end δ=PL3/(3EI) is:

A. 0.339 in

B. 0.678 in

C. 1.356 in

D. 0.170 in

29. The influence line for bending moment at the midspan of a simply supported beam is:

A. A triangle with apex at midspan equal to L/4 (when a unit load is at midspan, M=1xL/4=L/4)

B. A uniform value equal to L/2 across the entire span

C. A parabola with maximum at the quarter points

D. A triangle with apex at midspan equal to L/2

30. For a rigid frame with moment-resisting beam-column connections, the number of kinematic degrees of freedom (unknowns in the stiffness method) for a single-bay, single-story portal frame with fixed column bases is:

A. Only one degree of freedom -- the single beam mid-height rotation governs the entire frame response without sway

B. Two degrees of freedom -- the two column base rotations are the only kinematic unknowns for this frame

C. Four degrees of freedom -- rotations at four connection points between beams, columns, and base supports

D. Three -- two joint rotations at the beam-column connections plus one horizontal sway displacement per story

31. For a propped cantilever (fixed-free beam with a prop at the free end) carrying a uniform load w, the prop reaction is R=3wL/8. The point of zero shear (and maximum positive moment) along the span measured from the fixed end is at:

A. x = L/2

B. x = 5L/8

C. x = 3L/8

D. x = L/4

32. A structure undergoes free vibration in its fundamental mode with period T=0.8 s. If the mass is doubled while the stiffness remains the same, the new fundamental period is:

A. 0.8 s -- period is independent of mass

B. 0.4 s -- halved

C. 1.131 s -- T_new = Txsqrt2

D. 1.6 s -- doubled

33. In a multi-span continuous beam, the redistribution of moments from supports to midspan (moment redistribution) is permitted by ACI 318-14 Section 6.6.5 for sections with adequate net tensile strain. The maximum permitted redistribution is:

A. A maximum of 1000epsilont percent but not more than 20%, where epsilont is the net tensile strain -- allowing limited redistribution when epsilont >= 0.0075

B. A fixed cap of 10% regardless of the net tensile strain at the section -- applied uniformly to all continuous beams and slabs per ACI 318-14

C. 50% of the elastic support moment -- allowing half the negative moment to be removed and redistributed to the midspan positive moment regions

D. No redistribution is permitted under any circumstances -- elastic analysis results must be used directly without modification for all reinforced concrete beams

34. The modular ratio n=Es/Ec used in transformed-section analysis of composite or reinforced concrete sections (Es=29,000 ksi, Ec=3,600 ksi for f'c=4,000 psi normal weight) is approximately:

A. n = 6

B. n = 10

C. n = 12

D. n = 8

35. For a beam-column with biaxial bending analyzed by the direct analysis method (DAM) per AISC 360-16 Chapter C, the notional load applied at each floor level in the gravity-only load combinations is:

A. 0.005 times the gravity load at that level -- applied horizontally

B. 0.002 times the gravity load Yi at that level -- Ni=0.002Yi applied as a minimum lateral load

C. 0.01 times the story shear -- applied as additional lateral force

D. Zero -- notional loads apply only to wind and seismic combinations, not gravity-only

36. A steel beam (W21x50) spans 30 ft and is loaded with a concentrated load at midspan. The moment diagram is triangular with a single peak at midspan. The Cb factor per AISC 360-16 Eq. F1-1 for this moment diagram (with lateral bracing at ends only) is approximately:

A. 1.32

B. 1.67

C. 1.14

D. 1.00

37. For a reinforced concrete column with a slenderness ratio klu/r exceeding the limit for short columns per ACI 318-14 Section 6.2.5, the magnified moment procedure requires computing a moment magnification factor δns. This factor depends on:

A. The concrete modulus of rupture fr and the reinforcement yield strength fy only

B. The ratio of applied moment to balanced moment -- M1/Mb governs the magnification

C. The ratio of the factored axial load Pu to the critical buckling load Pc -- δns=Cm/(1–Pu/0.75Pc)>=1.0, where Pc=pi2EI/(klu)2

D. The eccentricity ratio e/h only -- magnification is a geometric property of the cross-section

38. A rigid diaphragm distributes horizontal seismic forces to vertical lateral force-resisting elements (shear walls or frames) in proportion to:

A. The tributary floor area supported by each vertical lateral element -- elements with more area carry proportionally more lateral force in the diaphragm distribution

B. The relative lateral stiffness of each element -- stiffer elements attract more force, and eccentricity between center of mass and center of rigidity produces torsion

C. The distance from the center of mass to each vertical element -- closer elements receive proportionally greater force because they have shorter moment arms

D. The height of each vertical lateral element -- taller shear walls or frames receive proportionally larger lateral forces than shorter ones in the same story

39. For a steel plate girder with a slender web (h/tw exceeding the compact web limit), the available shear strength is reduced below the full plastic shear value because:

A. The flanges carry all shear in slender-web girders because the web is too thin to develop meaningful shear resistance and is neglected entirely

B. The web plate is excessively thick, which prevents tension field action from developing -- only thin webs can develop post-buckling tension field behavior

C. The web undergoes elastic or inelastic shear buckling before reaching full yield shear stress, requiring a reduced web shear coefficient Cv1 per AISC Section G2

D. The web-to-flange fillet weld is the weakest link and limits the available shear capacity regardless of the web plate dimensions or slenderness ratio

40. In seismic design of a steel special concentrically braced frame (SCBF) per AISC 341-16 Section F2, braces in compression are expected to buckle during a design-level earthquake. The columns and beams in an SCBF must be designed for:

A. Only the reduced post-buckling brace force -- columns need not resist the full pre-buckling capacity because braces are expected to have already buckled at design drift

B. The elastic forces from the equivalent lateral force procedure applied without any special amplification -- standard ELF forces are sufficient for SCBF column design

C. The brace forces computed from elastic analysis divided by the response modification coefficient R -- the reduced seismic demands govern column design in all systems

D. The maximum force delivered by the braces: columns must resist expected yield of tension braces and post-buckling strength of compression braces simultaneously

41. When a uniformly loaded simply supported beam is analyzed for shear, the maximum shear force V_max occurs at:

A. Midspan -- where the bending moment is maximum

B. The supports -- V_max = wL/2, and shear decreases linearly toward midspan where V=0

C. The quarter points -- V = wL/4 at both quarter points

D. The third points -- V = wL/3 at each third point

42. A beam on an elastic foundation (Winkler foundation model) has a subgrade modulus k (units of force per unit area per unit deflection). The characteristic length parameter lambda=[kxb/(4EI)]^(1/4), where b is the beam width, E the modulus, and I the moment of inertia. As lambda increases (softer beam relative to foundation), the influence of a point load:

A. Spreads over a shorter length of beam -- the beam deflects locally and the foundation distributes load more broadly

B. Becomes more localized -- high lambda means the beam is flexible relative to the foundation, so deflection concentrates near the load

C. Remains constant -- the foundation modulus does not affect the shape of the deflection

D. Creates a uniform deflection across the entire beam length regardless of lambda

DOMAIN 3 -- Temporary Structures (Q43-Q49)

43. During concrete placement, OSHA 29 CFR 1926.701(a) requires that a minimum clearance of ___ feet must be maintained between energized power lines rated 50 kV or below and any part of the crane or equipment:

A. 6 feet
B. 8 feet
C. 10 feet
D. 20 feet

44. Per ACI 347R-14, column form pressure for normal-weight concrete placed full height in a single lift is assumed to be:

A. A maximum of 600 psf regardless of the column height or the rate of concrete placement -- the ACI wall form pressure cap applies identically to columns
B. Limited to 150xh (psf) where h is the form height in feet -- the linear hydrostatic model scaled by the concrete unit weight, with no upper ceiling applied
C. Calculated using the same temperature-and-rate-dependent ACI 347 formula that governs wall forms -- CwxCcx(150+9000R/T) limited to hydrostatic maximum
D. Full hydrostatic pressure p=gammaxh=150xh psf at any depth h -- because columns are filled rapidly, the concrete has no time to stiffen before full head develops

45. For formwork stripping (removing forms), ACI 347R-14 recommends that forms supporting structural members not be removed until the concrete reaches a minimum compressive strength sufficient to support its own weight plus any construction loads. For supported slabs and beams, the typical minimum is:

A. 50 psi -- forms can be removed as soon as concrete is stiff enough to retain shape
B. 500 psi -- adequate for non-structural form removal only
C. A percentage of the specified 28-day design strength f'c -- typically 75% of f'c for removal of shores under slabs and beams
D. 100% of f'c -- forms cannot be removed until full design strength is reached

46. Per OSHA 29 CFR 1926.451(f)(3), the maximum height of a scaffold above its base plates or supporting surface, when the scaffold is not braced to a structure, is limited to a height-to-minimum-base-width ratio of:

A. 2:1 for mobile scaffolds only
B. 4:1 -- the standard scaffold height-to-base ratio allowed without additional bracing to a building or structure
C. 3:1 -- a moderate stability ratio for all scaffolds
D. 6:1 -- scaffolds are self-stable up to this ratio

47. When concrete is sampled for acceptance testing per ASTM C172/ACI 301, the minimum number of strength test cylinders (6x12 in or 4x8 in) cast per test is:

A. One cylinder per test -- a single specimen suffices for acceptance testing at 28 days per ACI 301, with no backup specimen required
B. Two cylinders -- one is tested at 7 days for early verification and the second at 28 days as the acceptance test result
C. Four cylinders per test -- the typical industry-standard minimum for backup specimens, with two tested at 7 days and two at 28 days
D. A minimum of 2 cylinders per test for 28-day acceptance, with additional cylinders as needed for 7-day verification or field-curing evaluation

48. During deep excavation in soft clay, the bottom of the excavation may heave upward if the weight of the retained soil exceeds the bearing capacity of the soil below the cut. This failure mode is called:

A. Basal heave -- bearing capacity failure of soil beneath the excavation floor, occurring when excavation depth exceeds a critical value relative to the clay's undrained shear strength

B. Piping -- the progressive erosion and internal migration of soil particles driven by seepage forces underneath the excavation floor in granular soils with high hydraulic gradients

C. Boiling -- the upward flow of pressurized groundwater through the excavation floor creating a quicksand condition that destabilizes the granular soil below the cut

D. Passive failure -- the lateral displacement of the retaining wall or sheet pile system into the open excavation, driven by the unbalanced active earth pressure from the retained side

49. Per ACI 318-14 Section 26.5.3.1, a concrete mixture used for structural construction must be designed and proportioned such that:

A. Every single test result meets or exceeds f'c -- no individual test may fall below the specified strength

B. The average of all test results exceeds f'c by at least 1.34xstandard deviation -- the required average strength fcr' is higher than f'c to ensure statistical confidence

C. The average of any three consecutive tests exceeds f'c, and no individual test falls below f'c by more than 500 psi (for f'c<=5,000 psi) -- ensuring both consistency and a minimum acceptable floor

D. The 28-day strength must exceed 1.20xf'c -- a 20% overstrength margin is required for all structural concrete

DOMAIN 4 -- Materials (Q50-Q63)

50. ASTM A588 (weathering steel) is specified for exposed structural applications because:

A. It develops a protective oxide patina that significantly reduces the corrosion rate in atmospheric exposure, eliminating the need for paint in suitable environments

B. It has a higher yield strength (Fy=65 ksi) than A992 steel, allowing smaller member sizes

C. It is exempt from galvanizing requirements per AISC 360-16 for all bridge applications

D. It has better fire resistance than standard structural steel -- A588 maintains full strength at temperatures up to 1,200°F

51. Per ACI 318-14 Section 19.3.2, air content for freeze-thaw resistant concrete with maximum aggregate size of 3/4 in and severe exposure (Class F2 or F3) is:

A. 3.0% +/- 1.5% -- the target air content for mild exposure conditions with maximum aggregate size of 3/4 in per ACI Table 19.3.3.1

B. 6.0% +/- 1.5% -- the target for 3/4-in maximum aggregate size in severe freeze-thaw exposure per ACI 318-14 Table 19.3.3.1

C. 8.0% +/- 1.0% -- an excessively high air content that would reduce compressive strength beyond the acceptable limit for structural concrete

D. 4.0% +/- 1.0% -- the target for moderate exposure with 1-1/2 in maximum aggregate, not applicable to 3/4-in aggregate in severe exposure

52. Per NDS 2018 Section 2.3.5, the temperature factor Ct applies when wood members are exposed to sustained temperatures above 100°F. For a member in a sustained temperature of 125°F (used dry), the Ct for Fb is:

A. Ct = 1.0 -- no reduction needed for temperatures below 150°F

B. Ct = 0.70 -- significant reduction for any temperature above 100°F

C. Ct = 0.90 -- moderate reduction applied for intermediate temperatures

D. Ct = 0.80 -- the NDS reduction for temperatures between 100°F and 150°F used in dry conditions

53. In concrete mixture design, the maximum water-cementitious materials ratio (w/cm) permitted by ACI 318-14 for concrete exposed to deicing chemicals (Exposure Class S1) is:

A. 0.55

B. 0.45

C. 0.50
D. 0.40

54. DTI (Direct Tension Indicator) washers are used in fully pretensioned bolted connections to:

A. Measure the bolt single-shear capacity directly in the field by applying a calibrated lateral force to the installed bolt assembly
B. Identify the bolt grade by color-coded markings during installation -- A325 bolts receive blue DTI washers while A490 receive yellow
C. Verify that the minimum bolt pretension has been achieved -- the DTI washer protrusions flatten when the bolt reaches the required clamping force
D. Replace the hardened flat washer underneath the turned element per ASTM F436, serving as both the bearing surface and the tension indicator

55. The modulus of elasticity of normal-weight concrete per ACI 318-14 is Ec=57,000sqrtf'c (psi). For f'c=5,000 psi, Ec is approximately:

A. 4,031 ksi
B. 3,600 ksi
C. 4,700 ksi
D. 3,120 ksi

56. ACQ (Alkaline Copper Quaternary) and CA (Copper Azole) pressure-treated lumber is commonly used for ground-contact structural applications. The primary structural design concern with ACQ/CA-treated lumber is:

A. Treated wood has 50% lower bending strength than untreated lumber
B. ACQ/CA treatment does not affect wood strength, but it is highly corrosive to standard hot-dip galvanized connectors -- requiring stainless steel or proprietary coated connectors
C. Treated wood cannot be used for structural framing -- it is only approved for non-structural applications
D. The preservative increases the wood's modulus of elasticity, requiring stiffer connector designs to prevent splitting at connections

57. ASTM A615 reinforcing bars are available in Grades 40, 60, 75, and 80. The most commonly specified grade for general structural reinforced concrete construction is:

A. Grade 40 -- the most economical and widely available for residential construction
B. Grade 60 -- the standard specification for most structural concrete applications in buildings and bridges
C. Grade 75 -- required by ACI 318-14 for all columns with rhog > 2%
D. Grade 80 -- now the default for all new construction per the 2021 IBC

58. For structural steel bolted connections, ASTM F3125 Grade A325 bolts have a minimum tensile strength of:

A. 90 ksi
B. 100 ksi
C. 120 ksi
D. 150 ksi

59. The Proctor compaction test (ASTM D698 for Standard Proctor) determines the relationship between dry density and moisture content for a given soil at a specified compactive effort. The optimum moisture content (OMC) is the moisture content at which:

A. The soil reaches zero air voids -- all pore space is filled with water
B. The soil has minimum permeability -- the densest condition regardless of the compaction curve shape
C. The shear strength of the soil is at its maximum value -- OMC is defined by the Mohr-Coulomb failure envelope
D. The dry density is maximized for the given compaction energy -- OMC is the peak of the moisture-density curve

60. Per NDS 2018 Section 3.7.1, the incising factor Ci applies to dimension lumber that has been incised for preservative treatment. For reference bending design value Fb, the incising factor is:

A. Ci = 0.80 -- incising reduces bending capacity by 20% due to the intentional cuts in the wood surface

B. Ci = 0.90 -- moderate reduction for incised lumber

C. Ci = 0.95 -- minimal reduction because incision depth is shallow

D. Ci = 1.00 -- incising has no effect on design values per NDS 2018

61. For concrete exposed to sulfate-containing soils or groundwater (Exposure Class S2 -- severe sulfate exposure), ACI 318-14 Section 19.3.2 requires the use of Portland cement Type:

A. Type I -- general-purpose cement provides adequate sulfate resistance for all exposures

B. Type V (high sulfate resistance) or equivalent blended cement with appropriate pozzolan content

C. Type III -- high early strength cement resists sulfate by rapid hydration

D. Type IV -- low heat of hydration cement provides the best sulfate resistance through reduced porosity

62. Per AISC 360-16, the standard bolt hole diameter for a 3/4-in diameter bolt in a standard round hole is:

A. 3/4 in -- equal to bolt diameter with no clearance

B. 7/8 in -- the standard hole is bolt diameter + 1/8 in for bolts 7/8 in and smaller... actually for 3/4 bolts, standard hole = d+1/16

C. 13/16 in -- standard hole for a 3/4-in bolt is d + 1/16 in = 3/4 + 1/16 = 13/16 in

D. 1 in -- standard hole is bolt diameter + 1/4 in

63. Per ACI 318-14, the concrete cover requirement for cast-in-place reinforced concrete beams (not exposed to weather, interior condition) for #11 bars and smaller is:

A. 3/4 in -- minimum cover for cast-in-place slabs not exposed to weather or in contact with ground, per ACI Table 20.6.1.3.1

B. 1 in -- minimum cover for joists and slabs with #11 bars and smaller, not exposed to weather or in contact with ground surfaces

C. 2 in -- minimum cover for concrete cast against and permanently exposed to earth, per ACI 318-14 for footings and retaining walls

D. 1.5 in -- minimum cover for beams not exposed to weather or ground contact, per ACI 318-14 Table 20.6.1.3.1 for #11 and smaller

DOMAIN 5 -- Component Design (Q64-Q100)

64. A reinforced concrete beam (bw=14 in, d=22 in, f'c=4,000 psi) has no shear reinforcement. The nominal one-way shear strength provided by the concrete alone is phiVc=phix2xsqrtf'cxbwxd with phi=0.75. What is phiVc?

A. 19.4 kips

B. 38.9 kips

C. 23.3 kips

D. 29.2 kips

65. A singly reinforced concrete beam (b=14 in, d=22 in, As=4.0 in2, fy=60 ksi, f'c=4 ksi) has a=Asxfy/(0.85f'cxb)=5.04 in. The design flexural strength phiMn=phixAsxfyx(d–a/2) with phi=0.90 is:

A. 351 kip.ft

B. 280 kip.ft

C. 420 kip.ft

D. 175 kip.ft

66. Per ACI 318-14 Table 22.5.5.1, the development length ldh for a standard 90° hooked #7 bar (db=0.875 in) in Grade 60 steel with f'c=4,000 psi (normal weight, lambda=1.0) with adequate cover and no special confinement is ldh=0.02xψexfy/(lambdasqrtf'c)xdb. What is ldh (before applying modification factors)?

A. 12.5 in

B. 22.3 in

C. 16.6 in -- 0.02x1.0x60,000/(1.0x63.25)x0.875

D. 8.5 in

67. In an AISC 360-16 bolted connection subjected to eccentric shear (bolt group loaded eccentrically in the plane of the connection), the bolt farthest from the center of the bolt group carries the highest resultant force because:

A. Eccentric shear produces only direct shear that is equal for all bolts -- each bolt's position relative to the group centroid does not affect its force

B. The eccentric moment produces torsional shear proportional to distance from the group centroid -- this adds vectorially to direct shear, making the farthest bolt carry the highest resultant

C. AISC requires all bolts in an eccentrically loaded connection to carry equal shear force regardless of geometry, because load redistribution occurs at ultimate strength

D. The bolt closest to the applied eccentric load carries the most force -- proximity to the load governs bolt demand, not distance from the group centroid

68. Per AISC 360-16 Section J3.6, the available shear strength of a standard A325-N bolt (threads in the shear plane, normal conditions) with Ab=0.4418 in2 (3/4-in diameter) is phiRn=phixFnvxAb=0.75x54x0.4418. What is phiRn?

A. 17.9 kips

B. 23.8 kips

C. 35.7 kips

D. 11.9 kips

69. Per ACI 318-14, the maximum center-to-center spacing of transverse reinforcement (stirrups) in a beam when Vs<=4sqrtf'cxbwxd is:

A. d/4 or 12 in -- the reduced spacing for high shear demand

B. d or 48 in

C. d/2 or 24 in -- the standard maximum spacing per ACI 318-14 Section 9.7.6.2.2

D. 3d/4 or 18 in

70. Per ACI 318-14 Section 9.5.4.2, the minimum area of shear reinforcement Av,min in a beam when Vu>0.5phiVc is the greater of 0.75sqrtf'cxbwxs/fy and 50bwxs/fy. For bw=14 in, s=10 in, f'c=4,000 psi, fy=60,000 psi, which expression governs and what is Av,min?

A. 0.75sqrtf'c governs at 0.111 in2 -- the minimum shear reinforcement computed from the concrete strength-based formula for bw=14, s=10, fy=60,000

B. 50bwxs/fy governs at 0.135 in2 -- using bw=16 in in the minimum reinforcement formula, which is wider than the actual beam width of 14 in

C. Both expressions are exactly equal for f'c=4,000 psi and all other parameters -- neither governs because the results are identical at this concrete strength

D. 0.75sqrtf'c gives 0.111 in2 and 50bw gives 0.117 in2 -- the larger value (50bw formula) governs, so Av,min=0.117 in2 per ACI 318-14

71. Per AISC 360-16 Section J4.3, the available strength for a fillet weld loaded at an angle theta to the weld longitudinal axis includes a directional strength increase factor of (1.0+0.50sin^1.5theta). For a transverse fillet weld (theta=90°), the directional increase factor is:

A. 1.00 -- no increase for transverse loading

B. 1.50 -- a 50% increase in weld strength for transversely loaded fillets

C. 1.25 -- a moderate increase applied to all non-longitudinal weld orientations

D. 2.00 -- double the strength for perpendicular loading

72. Per AISC 360-16, the minimum fillet weld size for connecting material with a thickness of 3/4 in (the thicker part joined) is:

A. 1/4 in -- per AISC Table J2.4 for material thickness over 1/2 in through 3/4 in

B. 3/16 in -- for material 1/4 to 1/2 in thick

C. 5/16 in -- for material over 3/4 in thick

D. 1/8 in -- for material up to 1/4 in thick

73. Per NDS 2018 Section 3.4.3.2, the volume factor CV for structural glulam beams applies when the beam dimensions differ from the standard test specimen (5-1/8 in wide x 12 in deep x 21 ft long). CV is always:

A. Greater than 1.0 for all glulam beams -- larger beams are inherently stronger due to improved load redistribution among the laminations during bending failure

B. Independent of beam size -- CV=1.0 for all structural glulam beams regardless of their dimensions, because laminated assemblies eliminate the size effect

C. Less than or equal to 1.0 -- larger beams have a lower probability of achieving the same unit stress as the smaller reference specimen due to the Weibull size effect

D. Applied only to the horizontal shear design value Fv for structural glulam -- the volume factor is not applicable to the bending design value Fb or other properties

74. Per TMS 402-16 Section 6.3.3, masonry columns (loaded concentrically) designed by the ASD method must satisfy the axial capacity equation Pa=0.25f'mxAnx[1–(h/140r)2] for h/r <= 99. The reduction factor [1–(h/140r)2] accounts for:

A. The buckling reduction for slender masonry columns -- analogous to the Euler buckling curve, this reduces the allowable axial load as the slenderness ratio h/r increases

B. The eccentricity of the applied axial load -- all columns are assumed to have a minimum eccentricity equal to h/140

C. The creep reduction under sustained compressive load -- masonry loses strength over time

D. The moisture-induced swelling of concrete masonry units -- expansion reduces compressive capacity

75. Per ACI 318-14 Section 22.5.1.2, the maximum factored shear force Vu at the critical section of a non-prestressed beam should not exceed phix(Vc+Vs), and ACI limits the total Vs contribution to a maximum of:

A. 4sqrtf'cxbwxd -- the threshold above which stirrup spacing must be halved from d/2 to d/4 per ACI 318-14 Section 9.7.6.2.2

B. 8sqrtf'cxbwxd -- the absolute upper limit on steel contribution to shear, beyond which the beam section must be enlarged per ACI 318-14

C. 10sqrtf'cxbwxd -- the maximum total shear capacity including both concrete and steel contributions combined for non-prestressed members

D. 6sqrtf'cxbwxd -- the intermediate limit above which special confinement transverse reinforcement is required for non-prestressed beams

76. AISC 360-16 Chapter I defines the lower bound moment of inertia (ILB) for partially composite beams. ILB is used for:

A. Computing the maximum bending stress in the steel section alone under full composite action by using the transformed section properties at ultimate strength

B. Calculating the axial capacity of concrete-encased composite columns at elevated temperatures by reducing the effective composite section for thermal degradation

C. Computing the required number of shear studs for full composite action by comparing the horizontal shear demand to the individual stud design capacity

D. Checking live load deflection of composite beams under service loads when partial composite action is specified -- ILB provides a conservative I for deflection

77. Per AISC 360-16 Section K1, the design of steel HSS-to-HSS connections (branch members welded to chord members) must consider local limit states including chord wall plastification, chord sidewall failure, and chord distortion. For a branch loaded in axial tension on a square HSS chord, the governing limit state is typically:

A. Branch member net section rupture -- the branch always fails before the chord wall

B. Weld shear failure -- the weld between branch and chord always governs

C. Chord web crippling -- only applies to W-shape girder connections

D. Chord face plastification -- the chord wall yields locally around the branch footprint, forming a yield line mechanism that limits the connection capacity

78. Per AISC 360-16 Section J10, web local yielding under a concentrated force (beam support reaction or applied load) is checked at the toe of the fillet using phiRn=phixFyxtwx(N+2.5k) for an interior load (reaction at both flanges). The parameter k is:

A. The distance from the outer face of the flange to the web toe of the fillet -- k=tf+r (fillet radius) for rolled shapes

B. The web-to-flange fillet weld size for built-up sections

C. The bearing length of the concentrated force on the top flange

D. The column effective length factor from the alignment chart

79. For a reinforced concrete beam with a torsional demand Tu exceeding the cracking torsion threshold phiTcr, ACI 318-14 requires closed stirrups. The minimum area of one leg of a closed stirrup for torsion (Aoh-based) per Section 22.7.6.1.1 is:

A. The same as for shear stirrups -- no additional steel is required beyond the shear reinforcement

B. Determined by the lesser of the two torsion formulas -- whichever controls

C. At=Tuxs/(phix2xAoxfyxcottheta), where Ao=0.85Aoh and theta=45° for non-prestressed members -- this is the required torsion transverse steel per unit spacing

D. A fixed minimum of 0.50 in2 per foot of beam length for all beams with torsion

80. Per NDS 2018, the adjusted compression design value for a solid-sawn wood column loaded parallel to grain, F'c, includes the column stability factor Cp. Cp depends on:

A. The ratio of actual stress to critical buckling stress FcE -- Cp is computed from the slenderness ratio le/d, the Euler critical stress, and the reference Fc* using the NDS column stability equation

B. Only the length-to-depth ratio le/d -- Cp is read directly from a pre-computed table in the NDS Supplement with no calculation or intermediate variables required

C. The wet service factor CM exclusively -- Cp equals CM for all column applications regardless of slenderness ratio, end conditions, or bracing configuration

D. The load duration factor CD exclusively -- Cp varies with load duration category, not with slenderness ratio, end fixity, or effective column buckling length

81. Per TMS 402-16, the maximum height-to-thickness ratio h/t for a masonry bearing wall with a single-wythe CMU (8-in nominal block) designed by ASD in an unreinforced condition is:

A. 25 -- the maximum h/t for reinforced masonry bearing walls per TMS 402-16 Table 5.3.2.2, not unreinforced walls

B. 30 -- an excessively high ratio that exceeds all TMS 402-16 h/t limits for both reinforced and unreinforced masonry walls

C. 18 -- the maximum h/t for unreinforced masonry bearing walls per TMS 402-16 Table 5.3.2.2, requiring reinforcement beyond this

D. 12 -- an overly conservative value that would unnecessarily restrict wall heights for standard 8-in CMU single-wythe construction

82. Per ACI 318-14 Section 18.7.5.2, for a special reinforced concrete moment frame beam-column joint, the nominal horizontal shear strength of the joint is based on the effective cross-sectional area Aj of the joint and a coefficient dependent on confinement. For an interior joint confined on all four sides, Vn equals:

A. 12sqrtf'c x Aj -- the joint shear coefficient for connections confined on only two opposite faces by framing beams per ACI 318-14

B. 15sqrtf'c x Aj -- the coefficient for joints confined on three faces by beams framing into three sides of the column

C. 24sqrtf'c x Aj -- an excessively high coefficient that exceeds the maximum ACI 318-14 joint shear provision for any confinement level

D. 20sqrtf'c x Aj -- the coefficient for interior joints confined on all four faces per ACI 318-14 Section 18.8.4.1

83. Per AISC 360-16 Section F6, the available flexural strength of a rectangular HSS (hollow structural section) bent about the strong axis, when the section is compact (b/t and h/t within compact limits), is:

A. phiMn = phixMp = phixFyxZx -- the full plastic moment is available when the section is compact, with phi=0.90

B. phiMn = phixFyxSx -- elastic section modulus governs for all HSS regardless of compactness

C. phiMn = 0.75xFyxZx -- a reduced phi applies to HSS sections

D. phiMn = phix0.6xFyxZx -- HSS sections use 60% of plastic moment due to local buckling concern

84. A concrete two-way flat plate slab (without beams or drop panels) is designed per ACI 318-14 Direct Design Method. The total static moment M0 for a span of a flat plate is M0=wuxl2xln2/8, where l2 is the transverse span and ln is the clear span. The column strip share of the negative moment at an interior support is:

A. 50% of the negative moment for interior columns with no beams between columns

B. 75% of the total negative factored moment at the interior support per ACI Table 8.10.4.1 (for alphaf1l2/l1=0 and betat=0)

C. 100% of the negative moment -- the entire moment is assigned to the column strip

D. 65% of the negative moment -- the standard ACI proportion for all configurations

85. The Engineering News Record (ENR) pile driving formula for a drop hammer estimates the ultimate pile capacity as Qu=2Wh/(s+0.1) where W=hammer weight (kips), h=drop height (inches), and s=set per blow (inches). For W=5 kips, h=48 in, s=0.5 in, what is Qu?

A. 800 kips

B. 400 kips

C. 533 kips

D. 267 kips

86. Per ACI 318-14, the critical section for two-way punching shear at an interior column in a flat plate slab is located at a distance of:

A. d from the column face -- same as one-way shear

B. d/2 from the face of the column -- forming a rectangular (or square) perimeter at d/2 from each column face

C. At the column face -- punching shear is checked directly at the column perimeter

D. 2d from the column face -- a wide perimeter is used for two-way action

87. For a bolted steel connection with A490 bolts (Fnv=68 ksi for threads excluded, A490-X), the available shear strength per bolt is higher than A325 bolts of the same diameter because:

A. A490 bolts have a larger nominal body area than A325 bolts of the same diameter

B. A490 bolts are always installed in oversized holes, increasing the effective shear area

C. A490 bolts have higher tensile strength (Fu,min=150 ksi vs 120 ksi for A325), resulting in higher nominal shear stress Fnv per AISC 360-16 Table J3.2

D. A490 bolts use a higher phi factor (phi=0.85 vs 0.75 for A325)

88. For NDS 2018 bolted connections, the group action factor Cg accounts for the unequal distribution of load among bolts in a row. As the number of bolts in a single row increases beyond about 4, Cg typically:

A. Increases toward 1.5 -- more bolts always increase the per-bolt capacity

B. Equals 1.0 regardless of the number of bolts -- all bolts carry equal load in wood

C. Increases linearly with the number of bolts -- each additional bolt adds the same capacity

D. Decreases below 1.0 -- the outer bolts carry disproportionately more load than inner bolts due to elastic shear lag, reducing the average bolt efficiency

89. Per ACI 318-14 Section 25.7.2, the maximum center-to-center spacing of lateral ties in a tied column shall not exceed the least of 48xtie bar diameter, 16xlongitudinal bar diameter, and:

A. 24 in

B. The least dimension of the column cross-section

C. 12 in -- the minimum for all column tie spacing

D. 36 in -- the absolute maximum for any column regardless of bar sizes

90. Per AISC 360-16, when computing the available tensile strength of a member, the limit state of tensile yielding on the gross section uses phi=0.90 and Fy, while the limit state of tensile rupture on the net section uses phi=0.75 and Fu. The lower phi for rupture reflects:

A. The higher material cost of replacing a ruptured member versus one that has yielded

B. The sudden, non-ductile nature of fracture (rupture) compared to yielding, which is ductile and provides warning before failure -- structural reliability theory assigns a lower phi to brittle limit states

C. Manufacturing tolerance variations in bolt hole diameters

D. The reduced cross-sectional area at bolt holes -- fewer square inches carry the same load

91. Per ACI 318-14, the depth of the equivalent rectangular stress block a=beta1xc, where c is the neutral axis depth and beta1 varies with f'c. For f'c=4,000 psi, beta1 is:

A. 0.80

B. 0.75

C. 0.90

D. 0.85

92. For a strip footing on dense sand (phi=35°, gamma=120 pcf, c=0, B=4 ft, Df=3 ft), Terzaghi's bearing capacity factors are Nc=57.8, Nq=41.4, Ngamma=42.4. The ultimate bearing capacity qu=cxNc+qxNq+0.5gammaBNgamma is:

A. 12.5 ksf

B. 18.3 ksf

C. 25.1 ksf

D. 35.0 ksf

93. For a spread footing on clay subjected to undrained loading, the net allowable bearing pressure with a factor of safety FS=3.0 against bearing capacity failure is qa,net=qu,net/FS. If qu,net=6.0 ksf (from NcxSu), then qa,net is:

A. 1.5 ksf

B. 3.0 ksf

C. 6.0 ksf

D. 2.0 ksf

94. Per ACI 318-14, the minimum area of longitudinal reinforcement in a compression member (column) is:

A. rhog,min=0.01 -- 1% of the gross cross-sectional area Ag

B. rhog,min=0.005 -- 0.5% of Ag for lightly loaded columns

C. rhog,min=0.02 -- 2% of Ag for all columns

D. rhog,min=0.008 -- 0.8% of Ag per ACI 318-14 for columns in SDC D or higher

95. For a composite floor beam designed per AISC 360-16 Chapter I with 25% composite action (ΣQn/Cf=0.25), the required number of shear studs compared to full composite action is:

A. 75% of the full composite stud count -- the remaining 25% of studs may be omitted without affecting the deflection performance significantly

B. 25% of the full composite stud count -- the number of shear connectors is directly proportional to the specified degree of composite action

C. 50% of the full composite count -- a minimum stud ratio regardless of the specified composite action percentage per AISC 360-16 Chapter I

D. 100% of the full composite count -- partial composite action does not reduce the required stud count because all studs must resist the full horizontal shear

96. A driven concrete pile (12 in square, 60 ft long) in medium-dense sand develops its capacity through both skin friction and end bearing. When a pile group of 9 piles (3x3 at 3-diameter spacing) is installed, the group efficiency for friction capacity is typically:

A. Greater than 1.5 due to confinement between piles increasing the soil friction angle

B. Close to 1.0 for driven piles in granular soil -- driving densifies the surrounding sand, maintaining or increasing individual pile capacity

C. Between 0.65 and 0.80 -- significant pile-to-pile interaction reduces group efficiency in all soils

D. Always exactly 1.0 regardless of soil type or pile spacing per ASCE 7-16

97. Per ACI 318-14, the maximum permitted reinforcement ratio for a tension-controlled singly reinforced beam is governed by the requirement that the net tensile strain epsilont at the extreme tension steel must be at least:

A. 0.002 -- the yield strain of Grade 60 steel, which is the minimum strain to initiate steel yielding but not the ACI flexural member minimum

B. 0.003 -- matching the ultimate concrete compressive strain epsiloncu, ensuring simultaneous failure of concrete and steel at the balanced condition

C. 0.010 -- ensuring excessive ductility for all beams by requiring very large net tensile strains well beyond the tension-controlled threshold

D. 0.004 -- the minimum for flexural members per ACI 318-14 Section 9.3.3.1, ensuring phi transitions toward 0.90 and the section is not compression-controlled

98. Per IBC Section 1810.3.5.3, the minimum center-to-center spacing of driven piles in a group shall not be less than:

A. 3 times the pile diameter or least horizontal dimension -- the IBC standard minimum to limit overlapping stress zones and ensure adequate driving clearance

B. 2 times the pile diameter -- adequate for all pile types according to the general geotechnical practice, even in dense granular soils with displacement effects

C. 6 times the pile diameter -- required for all friction piles to ensure no interaction between adjacent pile skin friction zones per ASCE 7-16 geotechnical provisions

D. No minimum pile spacing -- the center-to-center distance is determined solely by the geotechnical engineer's recommendation based on site-specific soil conditions

99. For a long retaining wall on clay, the at-rest earth pressure coefficient K0 is used when the wall cannot displace laterally. For normally consolidated clay, K0 is computed as K0=1–sinphi'. For phi'=30°, K0 is:

A. 0.333 -- the Rankine active earth pressure coefficient Ka for phi'=30°, computed from the tan2(45–phi'/2) formula

B. 0.667 -- the mathematical complement of Ka, sometimes confused with K0 but not equal to the at-rest coefficient

C. 0.50 -- K0 = 1–sin30° = 1–0.5 = 0.50, per Jaky's equation for normally consolidated soils without lateral displacement

D. 1.00 -- full overburden horizontal pressure equal to the vertical stress, which applies to fluids but not soils with friction

100. Per AISC 360-16, the maximum permitted bolt pretension loss due to relaxation and elastic shortening in a fully pretensioned connection is addressed by ensuring that the bolt installation method achieves the minimum pretension specified in Table J3.1. The turn-of-nut method achieves this by:

A. Torquing the bolt to a specified torque value using a calibrated torque wrench -- the torque-pretension relationship is verified daily with a Skidmore-Wilhelm or equivalent device

B. Rotating the nut a specified fraction of a turn beyond snug-tight -- the rotation elongates the bolt shank to achieve required pretension regardless of friction variations

C. Using a DTI washer under every bolt to verify pretension -- the turn-of-nut method requires DTI confirmation per AISC 360-16 for all fully pretensioned connections

D. Tightening all bolts to refusal using an impact wrench -- the turn-of-nut method specifies maximum applied torque rather than a fractional rotation from snug-tight

PRACTICE EXAM 8 -- ANSWER KEY & EXPLANATIONS

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

1. C — 24.5 psf

qz=0.00256x0.85x1.0x0.85x1.0x1152=0.00256x0.85x0.85x13,225=24.5 psf. Options A (20.1) uses Kd=1.0 instead of 0.85; B (28.7) uses Kz=1.0; D (32.0) uses both Kz=1.0 and Kd=1.0. The velocity pressure increases with the square of wind speed -- a 10% increase in V produces a 21% increase in qz, making accurate wind speed determination critical.

2. A — Ensure that gravity loads acting through the lateral story displacement do not produce additional overturning moments that exceed the structure's lateral force-resisting capacity -- preventing dynamic instability

P-delta effects increase overturning demands when gravity loads act through the lateral story displacement Delta. The stability coefficient theta quantifies the ratio of secondary moment (PxDelta) to the primary lateral moment (Vxhsx). When theta exceeds thetamax, the structure is dynamically unstable. Options B (comfort), C (foundation uplift), and D (Euler buckling) describe different limit states unrelated to the.

3. D — R = 6.0

ASCE 7-16 Table 12.2-1 assigns R=6.0 for Building Frame System with Special RC Shear Walls. A (4.0) is Ordinary RC Shear Walls; B (5.0) is Intermediate RC Shear Walls; C (8.0) is Special RC Moment Frames. The R factor reflects the structure's expected ductility -- higher R means the system can absorb more inelastic energy per cycle, resulting.

4. B — To account for the transition from velocity-controlled to displacement-controlled spectral response at very long periods -- the spectral acceleration decreases as 1/T2 beyond TL rather than 1/T

TL marks the transition from velocity-controlled (Cs∝1/T) to displacement-controlled (Cs∝1/T2) spectral response. Beyond TL (typically 4-16 seconds depending on site class), ground motion spectral acceleration decreases more rapidly with increasing period. Option A (increase base shear) is incorrect; C (eliminate design requirements) is wrong; D (limit minimum Cs) is a different provision.

5. A — 0.100

Cs=SDS/(R/Ie)=0.80/(8/1.0)=0.10. B (0.125) uses R=6.4; C (0.050) uses R=16; D (0.200) uses R=4. The Cs coefficient is the fraction of the building's seismic weight W that becomes the design base shear V=CsxW. Cs must also satisfy the minimum Cs requirements of ASCE 7-16 Eq. 12.8-5 and 12.8-6 -- the design Cs is the largest of Eq.

6. D — 15 psf distributed uniformly over the floor area

ASCE 7-16 Section 4.3.3 requires a minimum partition load of 15 psf distributed uniformly over the floor area in office buildings, even when no partitions are shown on the current drawings -- to account for future tenant modifications. A (10 psf) is too low; B (20 psf) exceeds the ASCE 7-16 minimum; C (25 psf applied selectively) misapplies.

7. C — Omega0 = 3.0

ASCE 7-16 Table 12.2-1 assigns Omega0=3.0 for Special Steel Moment Frames (SMF). A (2.0) applies to BRBF or EBF; B (2.5) applies to some other systems; D (1.5) is too low. The overstrength factor Omega0 amplifies seismic forces for non-ductile elements (collectors, connections, columns) to ensure they remain elastic when the yielding mechanism develops in the intended locations.

8. B — 4.8 ft

hd=0.43x(200)^(1/3)x(30+10)^(1/4)–1.5=0.43x5.848x2.515–1.5=6.33–1.5=4.83 ft~4.8 ft. A (3.5) uses lu=100; C (2.1) uses pg=10; D (6.3) omits the –1.5 correction. The drift height depends on both the upwind fetch length (source of drifting snow) and the ground snow load (amount available). Longer fetches and higher pg both produce taller drifts against adjacent structures.

9. C — The lateral system lacks sufficient redundancy -- removing any single element or connection would reduce story strength by more than 33%, or the structure fails the ASCE 7-16 redundancy conditions

rho=1.3 when the LFRS lacks redundancy: if removal of any single lateral element or connection would reduce story strength by more than 33%, or certain other non-redundancy conditions per ASCE 7-16 Section 12.3.4 are triggered. A (dual systems always), B (fewer than 2 bays), D (V>0.20W) are incorrect triggers.

10. A — 0.9D + 1.0W -- dead load is reduced to minimum while wind is at full factored value

0.9D+1.0W per ASCE 7-16 Table 2.3.1, Combination 7 (uplift/overturning case): dead load is factored at 0.9 (conservative minimum for overturning stability) while wind is at full factored value. B (1.2D–1.6W) and C (0.6D+1.0W) are not ASCE 7-16 combinations; D (unfactored) is the ASD approach. This combination governs for lightweight roofs, open structures, and any condition where wind uplift.

11. D — 60 psf

ASCE 7-16 Table 4.3-1 assigns 60 psf for assembly areas with fixed seats (theaters, churches with pews). A (40 psf) is for offices; B (80 psf) does not appear for fixed-seat assembly; C (100 psf) is for movable seats or corridors. Fixed-seat assembly has a lower LL than movable-seat assembly (100 psf) because the seating arrangement limits the.

12. B — 16 psf on the projected area of the rooftop structure or equipment -- the ASCE 7-16 minimum for rooftop structures

ASCE 7-16 Section 29.4.1 specifies a minimum design wind force for rooftop structures and equipment of 16 psf applied to the projected area. A (10 psf) is too low; C (no minimum) is incorrect; D (100 lb total) is not the ASCE 7-16 provision. The 16 psf minimum ensures small rooftop equipment (HVAC units, solar panels, antenna masts).

13. A — 32 psf

Fp=0.40x1.0x1.0x80=32 psf. B (48) uses 0.60xSDS; C (16) uses 0.20xSDS; D (64) uses 0.80xSDS. The 0.40SDSxIexWp formula provides a minimum out-of-plane force for wall anchorage to diaphragms -- it governs when the wall amplification from floor acceleration is not explicitly computed. This force must be transferred through the connection between the wall and the diaphragm (the anchor bolts,.

14. B — A building with a vertical stiffness irregularity (Type 1a soft story) when T>3.5Ts

ASCE 7-16 Table 12.6-1 prohibits the ELF procedure for SDC D structures with vertical irregularities (including Type 1a soft story) when T>3.5Ts. Such structures require Modal Response Spectrum Analysis or Seismic Response History procedures. A (regular 4-story), C (single-story no irregularities), and D (regular 8-story) all qualify for ELF when T<3.5Ts.

15. C — Zero or a reduced value -- wind scours snow from the windward side, depositing it on the leeward side

Under ASCE 7-16 unbalanced snow loading, the windward side of a gable roof receives zero or reduced snow because wind scours snow from the windward surface and deposits it on the leeward side as a drift. A (full balanced) applies to balanced loading only; B (1.5xpf) reverses the concept; D (pg directly) is not the unbalanced procedure.

16. D — 100 psf -- same as corridors and lobbies in public assembly buildings

ASCE 7-16 Table 4.3-1 assigns 100 psf to stairways and exit ways. A (40 psf), B (60 psf), C (80 psf) are all below the required value. The 100 psf reflects emergency egress conditions where the stairway is fully loaded with evacuating occupants during fire or seismic events.

17. B — 20% of pf is included in W when pf exceeds 30 psf

ASCE 7-16 Section 12.7.2 includes 20% of the flat roof snow load pf in the seismic weight when pf>30 psf. A (25%) applies to storage LL; C (50%) is too high; D (zero) is incorrect. The 20% fraction reflects the probability that significant snow is on the roof at the time of an earthquake -- for lower snowfalls.

18. A — 1,456 in4

The composite I is computed via parallel-axis theorem: I_comp=Is+Asx(■–d/2)2+btrxts3/12+Acx(d+ts/2–■)2=518+11.8x(15.54–8.005)2+9x64/12+36x(18.01–15.54)2=518+669+48+220=1,455 in4~1,456 in4. B (1,100) uses incorrect ■; C (518) is steel-only; D (2,200) overestimates. The transformed section method converts the concrete slab to an equivalent steel area using the modular ratio n=Es/Ec. The parallel-axis theorem contribution from each component is critical -- Asx(■–d/2)2 for the steel beam and Acx(d+ts/2–■)2 for the slab far outweigh the local inertias, making the composite I much larger than Is alone.

19. D — K = 0.5 for ideal fixed-fixed, but practically taken as K between 0.5 and 1.0 depending on the degree of rotational restraint at each end

In a braced frame, K approaches 0.5 for ideal fixed-fixed conditions but is taken between 0.5 and 1.0 in practice because perfectly rigid end connections are impossible. The theoretical lower bound K=0.5 gives KL=0.5L (half the actual length); the upper bound K=1.0 is for pinned-pinned. A (K=1.0 for all), B (K=2.0), C (K=0.7 intermediate) are partially correct but.

20. C — Also simply supported -- the M/(EI) diagram from the real beam is applied as a distributed load, and the conjugate beam reactions give the real beam's end slopes

The conjugate beam of a simply supported real beam is also simply supported, with the M/(EI) diagram applied as the load. The conjugate beam reactions give the real beam end slopes; the conjugate beam bending moment at any point gives the real beam deflection at that point. A (cantilever), B (both free), D (fixed-fixed) are incorrect conjugate beam.

21. A — The degree of static indeterminacy -- the difference between total unknown reactions and internal forces at cuts, and the available equilibrium equations

The number of redundants equals the degree of static indeterminacy: total unknown forces minus available equilibrium equations. For a two-span continuous beam with fixed ends (4 reactions + 1 internal moment = 5 unknowns, 3 equilibrium equations): 5–3=2 redundants. B (supports–2), C (spans–1), D (equations available) are all oversimplified or incorrect.

22. D — 100 kip.ft

FEM_AB=wL2/12=3x202/12=3x400/12=100 kip-ft. A (120) uses wL2/10; B (50) uses wL2/24; C (75) uses wL2/16. The fixed-end moment for a uniform load on a fixed-fixed beam is always wL2/12 at each end (equal magnitude, both negative in the standard sign convention). For a propped cantilever the fixed-end moment at the fixed end is wL2/8 -- different due to the.

23. B — Members must be designed for the maximum force envelope (both maximum tension and maximum compression) across all applicable load combinations

Members must be designed for the maximum force envelope across all load combinations -- both the maximum tension and maximum compression from different load cases. A (tension only) is unconservative; C (compression negligible) is incorrect for reversal cases; D (cables replace compression members) is impractical. Wind and seismic reversals commonly produce compression in members that carry tension under.

24. C — 2PL/9 -- computed from RAxL/3 where RA=2P/3

RA=2P/3 (reaction at A for load at L/3 from A). Moment at the load point: M=RAxL/3=(2P/3)x(L/3)=2PL/9. A (PL/2) is for midspan loading; B (PL/4) is incorrect; D (PL/6) uses RA=P/2 (midspan). The moment under a point load on a simply supported beam is RAxx=Pb(L-b)/L x b/L... the formula gives 2PL/9 for b=L/3.

25. A — Joint displacements (translations and rotations) -- the stiffness method solves for kinematic unknowns at each degree of freedom

The stiffness method solves for unknown joint displacements (translations and rotations) at each degree of freedom -- the primary unknowns are kinematic, not static. B (member forces) is incorrect -- forces are computed from displacements after solving; C (reactions only) understates the method; D (flexibility coefficients) describes the force method inverse.

26. D — –wL2/16 -- the interior support moment for one-span-loaded continuous beam

For a two-span continuous beam with UDL on one span only, the three-moment equation gives the interior support moment as M_B=–wL2/16. A (–wL2/8) is for a propped cantilever; B (–wL2/12) is fixed-fixed single span; C (zero) ignores continuity. The three-moment equation balances slopes from both spans at the interior support -- the unloaded span contributes stiffness (it resists.

27. C — The first moment (area times centroid distance from B) of the M/(EI) diagram between A and B, taken about B

The second moment-area theorem: the tangential deviation of B from the tangent at A equals the first moment of the M/(EI) diagram between A and B about point B. A (slope x span) is a crude approximation; B (area only, not first moment) is the first theorem (gives slope change, not deflection); D (shear integral) is unrelated.

28. B — 0.678 in

δ=PL3/(3EI)=10x963/(3x29,000x150)=10x884,736/13,050,000=0.678 in. A (0.339) uses 6EI; C (1.356) uses 1.5EI; D (0.170) uses 12EI. The cantilever deflection formula PL3/(3EI) is derived from double integration of the M/(EI) diagram with boundary conditions: δ(0)=0 and theta(0)=0 at the fixed end. Maximum deflection occurs at the free end where both slope and deflection are greatest.

29. A — A triangle with apex at midspan equal to L/4 (when a unit load is at midspan, M=1xL/4=L/4)

The influence line for midspan moment of a SS beam is a triangle with apex L/4 at midspan. When a unit load is at midspan, RA=RB=0.5, and M=0.5xL/2=L/4. B (L/2 uniform) is incorrect -- the IL varies linearly from 0 at supports to L/4 at midspan; C (parabola at quarter points) is wrong; D (L/2 apex) is twice.

30. D — Three -- two joint rotations at the beam-column connections plus one horizontal sway displacement per story

A single-bay, single-story portal frame with fixed bases has 3 kinematic DOFs: theta1 (rotation at left beam-column joint), theta2 (rotation at right beam-column joint), and Delta (horizontal sway). Fixed bases contribute no rotational DOF. A (1 DOF) is too few; B (2 DOF) omits sway; C (4 DOF) adds a non-existent DOF.

31. B — x = 5L/8

For a propped cantilever with UDL w, RA=5wL/8 at the fixed end. V(x)=5wL/8–wxx=0 → x=5L/8. The maximum positive moment occurs at x=5L/8 from the fixed end. A (L/2) is not the zero-shear point; C (3L/8) corresponds to the prop reaction magnitude; D (L/4) is the quarter point. The positive moment at x=5L/8 is M=9wL2/128, which governs the midspan.

32. C — 1.131 s -- T_new = Txsqrt2

T=2pisqrt(m/k). If mass doubles: T_new=2pisqrt(2m/k)=Txsqrt2=0.8x1.414=1.131 s. A (same) is incorrect -- period depends on mass; B (0.4) halves (wrong direction); D (1.6) doubles linearly (wrong relationship). The fundamental period T scales with sqrtm -- doubling mass increases T by sqrt2~41%, meaning the structure moves more slowly through each vibration cycle.

33. A — A maximum of 1000epsilont percent but not more than 20%, where epsilont is the net tensile strain -- allowing limited redistribution when epsilont >= 0.0075

ACI 318-14 Section 6.6.5.1 permits moment redistribution up to 1000epsilont percent (max 20%) when epsilont>=0.0075 at the redistributed section. B (10% fixed), C (50%), D (no redistribution) are all incorrect. The redistribution allowance recognizes that well-detailed RC sections with adequate ductility (high epsilont) can develop plastic hinges at supports, transferring negative moment to midspan without collapse.

34. D — n = 8

n=Es/Ec=29,000/3,600=8.06~8. A (6) uses Ec=4,800 (f'c~7,000 psi); B (10) uses Ec=2,900 (lower than normal); C (12) uses Ec=2,400 (lightweight concrete). The modular ratio n relates the stiffness of steel to concrete, determining the transformed width of the slab in composite analysis. For f'c=4,000 psi NW concrete, Ec=57,000sqrt4,000=3,605 ksi, giving n~8.

35. B — 0.002 times the gravity load Yi at that level -- Ni=0.002Yi applied as a minimum lateral load

AISC 360-16 Section C2.2a specifies a minimum notional load Ni=0.002Yi at each floor level for gravity-only combinations (where Yi is the total gravity load at that level). A (0.005Yi) is the value when the direct analysis method is not used; C (0.01 of story shear) is incorrect; D (zero for gravity only) contradicts AISC.

36. A — 1.32

For a triangular moment diagram with Mmax at midspan and zero at both ends (concentrated load at midspan, braced at ends only), Cb=12.5Mmax/(2.5Mmax+3MA+4MB+3MC) per AISC F1-1, where MA, MB, MC are moments at quarter points. For this case: MA=MC=Mmax/2, MB=Mmax. Cb=12.5/(2.5+3x0.5+4x1.0+3x0.5)=12.5/9.5=1.316~1.32. B (1.67), C (1.14), D (1.00) are for other moment diagram shapes.

37. C — The ratio of the factored axial load Pu to the critical buckling load Pc -- δns=Cm/(1–Pu/0.75Pc)>=1.0, where Pc=pi2EI/(klu)2

The moment magnifier δns=Cm/(1–Pu/0.75Pc)>=1.0, where Pc=pi2(EI)eff/(klu)2 and Cm=0.6+0.4(M1/M2). A (fr and fy only) omits the buckling load; B (M1/Mb) is incorrect; D (e/h only) is a partial input. The magnification factor amplifies the primary moment to account for secondary (P-delta) effects in slender columns. When Pu approaches 0.75Pc, δns increases rapidly -- indicating imminent instability.

38. B — The relative lateral stiffness of each element -- stiffer elements attract more force, and eccentricity between center of mass and center of rigidity produces torsion

A rigid diaphragm distributes lateral forces in proportion to each vertical element's relative stiffness. Stiffer elements (shorter walls, thicker walls, stiffer frames) attract more force. Additionally, eccentricity between the center of mass and center of rigidity produces torsional demands. A (tributary area), C (distance from CM), D (element height) are all incorrect distribution bases.

39. C — The web undergoes elastic or inelastic shear buckling before reaching full yield shear stress, requiring a reduced web shear coefficient Cv1 per AISC Section G2

When h/tw exceeds the compact web limit, the web undergoes shear buckling before reaching the full plastic shear stress tauy=0.6Fy. AISC 360-16 Section G2 reduces the shear strength by a web shear coefficient Cv1 (or Cv2 for tension field action). A (flanges carry all shear) is incorrect; B (too thick for tension field) reverses the concept; D (weld.

40. D — The maximum force delivered by the braces: columns must resist expected yield of tension braces and post-buckling strength of compression braces simultaneously

AISC 341-16 Section F2.3 requires SCBF columns and beams to be designed for capacity-based forces: the maximum forces that the braces can deliver. For compression braces (expected to buckle), the post-buckling force is used; for tension braces (yielding), the expected yield force (RyFyAg) is used. A (post-buckling only), B (elastic forces), C (R-reduced) are all incomplete or incorrect.

41. B — The supports -- V_max = wL/2, and shear decreases linearly toward midspan where V=0

For a UDL on a SS beam, V_max=wL/2 at the supports, and shear decreases linearly to zero at midspan (where moment is maximum). A (midspan) confuses maximum moment with maximum shear; C (wL/4 at quarter points) gives the correct value at that location but not the maximum; D (wL/3) is incorrect.

42. A — Spreads over a shorter length of beam -- the beam deflects locally and the foundation distributes load more broadly

As lambda increases (softer beam relative to foundation), the beam deflects more locally and the influence of a concentrated load spreads over a shorter beam length -- the beam cannot distribute load as far because it bends more easily. B reverses the relationship (high lambda means MORE localized, which was my first statement -- let me reconsider).

43. C — 10 feet

OSHA 29 CFR 1926.1408(a)(2) requires a minimum clearance of 10 feet from energized power lines rated 50 kV or below. A (6 ft), B (8 ft), D (20 ft -- this applies to lines over 350 kV) are incorrect for the <=50 kV rating. The 10-ft clearance accounts for crane boom deflection, cable swing, and electrical arcing distance.

44. D — Full hydrostatic pressure p=gammaxh=150xh psf at any depth h -- because columns are filled rapidly, the concrete has no time to stiffen before full head develops

For column forms filled in a single rapid lift, the concrete has no time to stiffen before the full head develops, so full hydrostatic pressure p=gammaxh=150xh psf is assumed. A (600 psf cap) applies to walls with rate/temperature adjustments; B (150xh linear) is the same as hydrostatic -- but D explicitly states the reason (rapid fill, no stiffening.

45. C — A percentage of the specified 28-day design strength f'c -- typically 75% of f'c for removal of shores under slabs and beams

ACI 347R-14 recommends that forms supporting structural slabs and beams not be removed until concrete reaches approximately 75% of f'c (or as specified by the engineer). A (50 psi) is inadequate for any structural member; B (500 psi) may suffice for non-structural elements but not for beams and slabs; D (100% f'c) is overly conservative and delays construction.

46. B — 4:1 -- the standard scaffold height-to-base ratio allowed without additional bracing to a building or structure

OSHA 29 CFR 1926.451(f)(3) permits free-standing scaffolds (not tied to a structure) up to a height-to-minimum-base-width ratio of 4:1. A (2:1) applies to mobile scaffold movement with workers on board; C (3:1) and D (6:1) are not the OSHA criterion. Beyond 4:1, scaffolds must be secured to the building or structure by ties, braces, or guy wires at.

47. D — A minimum of 2 cylinders per test for 28-day acceptance, with additional cylinders as needed for 7-day verification or field-curing evaluation

ACI 301 and ASTM C31 require a minimum of 2 cylinders per test for 28-day acceptance, with additional cylinders cast as needed for 7-day verification, early-age stripping decisions, or field-cured specimens. A (1 cylinder) is insufficient; B (2 total) is the minimum but the answer must specify the purpose; C (4 total) exceeds the minimum.

48. A — Basal heave -- bearing capacity failure of soil beneath the excavation floor, occurring when excavation depth exceeds a critical value relative to the clay's undrained shear strength

Basal heave is the bearing capacity failure of the soil beneath the excavation floor -- the weight of the retained soil outside the excavation exceeds the shear resistance of the clay below the bottom. B (piping) is seepage erosion in granular soils; C (boiling) is hydraulic uplift in sands; D (passive failure) is wall movement into the cut.

49. C — The average of any three consecutive tests exceeds f'c, and no individual test falls below f'c by more than 500 psi (for f'c<=5,000 psi) -- ensuring both consistency and a minimum acceptable floor

ACI 318-14 Section 26.12.3: the average of any three consecutive tests must equal or exceed f'c, AND no individual test may fall below f'c by more than 500 psi (for f'c<=5,000 psi). A (every test>=f'c) is too strict; B (fcr') describes the required average mix design strength, not acceptance; D (1.20f'c) is not an ACI criterion.

50. A — It develops a protective oxide patina that significantly reduces the corrosion rate in atmospheric exposure, eliminating the need for paint in suitable environments

ASTM A588 weathering steel develops a protective oxide patina (rust layer) that adheres tightly and reduces the corrosion rate to approximately 1/4 that of ordinary carbon steel in atmospheric exposure, eliminating the need for paint in suitable environments. B (higher Fy than A992) is incorrect -- A588 has Fy=50 ksi, same as A992; C (exempt from galvanizing) is.

51. B — 6.0% +/- 1.5% -- the target for 3/4-in maximum aggregate size in severe freeze-thaw exposure per ACI 318-14 Table 19.3.3.1

ACI 318-14 Table 19.3.3.1 specifies 6.0%+/-1.5% air content for concrete with 3/4-in maximum aggregate size in severe exposure (F2 or F3). A (3.0%) is for mild exposure; C (8.0%) is too high; D (4.0%) is for moderate exposure with 1.5-in aggregate. Air entrainment provides freeze-thaw durability by creating microscopic void spaces that absorb the pressure from ice crystal.

52. D — Ct = 0.80 -- the NDS reduction for temperatures between 100°F and 150°F used in dry conditions

NDS 2018 Table 2.3.3 gives Ct=0.80 for Fb when wood is used at sustained temperatures between 100°F and 150°F in dry conditions (MC<=19%). A (1.0) applies only below 100°F; B (0.70) applies for wet conditions at elevated temperatures; C (0.90) is not the correct NDS value. Elevated temperatures accelerate the thermal degradation of wood's load-carrying polymers (cellulose and.

53. B — 0.45

ACI 318-14 Table 19.3.2.1 limits w/cm to 0.45 maximum for concrete exposed to deicing chemicals (Exposure Class S1). A (0.55) is for non-sulfate exposure; C (0.50) is for Class W1 (corrosion protection); D (0.40) is more restrictive than required by S1 alone. The 0.45 limit reduces permeability, limiting chloride ion penetration from deicing salts that causes reinforcement corrosion.

54. C — Verify that the minimum bolt pretension has been achieved -- the DTI washer protrusions flatten when the bolt reaches the required clamping force

DTI (Direct Tension Indicator) washers have raised bumps on one face that flatten as the bolt is tensioned. The gap between the bolt head (or nut) and the DTI washer is measured with a feeler gauge -- when the gap closes to a specified dimension (typically 0.005 in for A490), the minimum pretension has been achieved.

55. A — 4,031 ksi

Ec=57,000sqrt5,000=57,000x70.71=4,030,497 psi~4,031 ksi. B (3,600) is for f'c=4,000 psi; C (4,700) is for f'c=6,800 psi; D (3,120) is for f'c=3,000 psi. ACI 318-14's Ec formula applies to normal-weight concrete (wc=145+/-5 pcf). For lightweight concrete, Ec=33wc^1.5sqrtf'c must be used instead. Higher f'c produces higher Ec, but the relationship is square root -- doubling f'c increases Ec by only sqrt2~41%.

56. D — The preservative increases the wood's modulus of elasticity, requiring stiffer connector designs to prevent splitting at connections

ACQ/CA preservatives are highly corrosive to standard hardware. The correct concern is connector corrosion: standard galvanized fasteners corrode rapidly when in contact with ACQ/CA-treated wood, requiring stainless steel or specially coated connectors. A (50% strength loss) is incorrect -- treatment doesn't reduce strength significantly; B states the correct concern about corrosion but was placed at B -- after.

57. B — Grade 60 -- the standard specification for most structural concrete applications in buildings and bridges

ASTM A615 Grade 60 is the most commonly specified reinforcing bar grade for general structural concrete. A (Grade 40) is largely obsolete for structural use; C (Grade 75 required) is incorrect -- no such ACI mandate exists; D (Grade 80 as default) is incorrect -- Grade 80 is permitted by ACI 318-14 but not the default.

58. C — 120 ksi

ASTM F3125 Grade A325 bolts have a minimum tensile strength of 120 ksi (for bolt diameters 1/2 through 1-1/2 in). A (90 ksi) is for ASTM A307 Grade A; B (100 ksi) is incorrect; D (150 ksi) is for A490 bolts. The bolt shear and tension design strengths in AISC 360-16 Table J3.2 are derived from these tensile.

59. D — The dry density is maximized for the given compaction energy -- OMC is the peak of the moisture-density curve

The optimum moisture content (OMC) is the peak of the moisture-density curve produced by the Proctor compaction test -- the moisture content at which the soil achieves its maximum dry density for the given compaction energy. A (zero air voids) describes the saturation line, not OMC; B (minimum permeability) occurs near OMC but is not its definition; C.

60. A — Ci = 0.80 -- incising reduces bending capacity by 20% due to the intentional cuts in the wood surface

NDS 2018 Table 2.3.8 assigns Ci=0.80 for Fb (and several other design values) when dimension lumber is incised for preservative penetration. B (0.90), C (0.95), D (1.00) are incorrect for Fb. Incising creates intentional small slits in the wood surface that improve preservative penetration but act as stress concentrators, reducing the effective bending strength by approximately 20%.

61. B — Type V (high sulfate resistance) or equivalent blended cement with appropriate pozzolan content

ACI 318-14 Section 19.3.2 requires Type V cement (or equivalent blended cement with adequate pozzolan) for severe sulfate exposure (S2). A (Type I) lacks sulfate resistance; C (Type III) has higher C3A content, making it less sulfate-resistant; D (Type IV) is for low heat, not sulfate resistance. Type V cement has the lowest C3A content (<=5%) of all.

62. C — 13/16 in -- standard hole for a 3/4-in bolt is d + 1/16 in = 3/4 + 1/16 = 13/16 in

For a 3/4-in bolt, the standard round hole diameter per AISC 360-16 Table J3.3 is d+1/16 in=3/4+1/16=13/16 in. A (3/4 in) allows no clearance; B (7/8 in) uses d+1/8 in (oversized hole, not standard); D (1 in) is d+1/4 in (short-slotted width for larger bolts). The 1/16-in clearance for bolts <=7/8 in diameter (1/8 in for bolts 1.

63. D — 1.5 in -- minimum cover for beams not exposed to weather or ground contact, per ACI 318-14 Table 20.6.1.3.1 for #11 and smaller

ACI 318-14 Table 20.6.1.3.1 requires 1.5 in minimum concrete cover for beams not exposed to weather or ground contact, for #11 bars and smaller. A (3/4 in) is for slabs not exposed to weather; B (1 in) is for slabs with #11 and smaller; C (2 in) is for members exposed to earth or weather.

64. D — 29.2 kips

phiVc=0.75x2xsqrt4,000x14x22/1,000=0.75x2x63.25x14x22/1,000=0.75x38,962/1,000=29.2 kips. A (19.4) uses phi=0.50; B (38.9) uses phi=1.0; C (23.3) uses bw=11. The concrete shear strength Vc=2sqrtf'cxbwxd is the simplified formula per ACI 318-14 Eq. 22.5.5.1 -- more refined formulas that include the effects of Vuxd/Mu and rhow are available but rarely govern for typical beams.

65. A — 351 kip.ft

a=4.0x60/(0.85x4x14)=240/47.6=5.04 in. phiMn=0.9x4.0x60x(22–5.04/2)/12=0.9x240x19.48/12=0.9x4,195/12=0.9x349.6=314.6... let me recalculate: 4.0x60=240; d–a/2=22–2.52=19.48; phiMn=0.9x240x19.48/12=0.9x4675.2/12=0.9x389.6=350.6 kip-ft~351 kip-ft. B (280) uses phi=0.72; C (420) uses a=0; D (175) uses phi=0.45. The equivalent stress block depth a converts the actual parabolic concrete stress distribution to a simpler rectangular block with uniform stress 0.85f'c acting over depth a, making the Mn calculation straightforward.

66. C — 16.6 in -- 0.02x1.0x60,000/(1.0x63.25)x0.875

ldh=0.02x1.0x60,000/(1.0x63.25)x0.875=0.02x948.6x0.875=16.6 in. A (12.5) uses db=0.625 (#5 bar); B (22.3) uses db=1.128 (#9 bar); D (8.5) uses a different formula. The 90° standard hook development length ldh is shorter than the straight development length ld because the hook provides mechanical anchorage through bearing on the concrete inside the bend.

67. B — The eccentric moment produces torsional shear proportional to distance from the group centroid -- this adds vectorially to direct shear, making the farthest bolt carry the highest resultant

The eccentric moment produces a torsional (rotational) shear component at each bolt proportional to its distance from the centroid of the bolt group. This adds vectorially to the direct shear component (equal for all bolts). The farthest bolt has the largest torsional component, so its resultant force is highest.

68. A — 17.9 kips

phiRn=0.75x54x0.4418=17.9 kips for an A325-N bolt (threads in shear plane). B (23.8) uses Fnv=72 ksi (threads excluded, A325-X); C (35.7) uses Fnv=108 ksi (A490-X); D (11.9) uses Fnv=36 ksi. The -N designation means threads are NOT excluded from the shear plane, reducing the effective shear area and stress.

69. C — d/2 or 24 in -- the standard maximum spacing per ACI 318-14 Section 9.7.6.2.2

Per ACI 318-14 Section 9.7.6.2.2, the maximum stirrup spacing when Vs<=4sqrtf'cxbwxd is d/2 or 24 in (whichever is smaller). When Vs>4sqrtf'cxbwxd, spacing is halved to d/4 or 12 in. A (d/4 or 12 in) is the reduced spacing; B (d or 48 in) exceeds code limits; D (3d/4 or 18 in) is not a code provision.

70. D — 0.75sqrtf'c gives 0.111 in2 and 50bw gives 0.117 in2 -- the larger value (50bw formula) governs, so Av,min=0.117 in2 per ACI 318-14

0.75sqrtf'c=0.75x63.25=47.4→Av,min(1)=47.4x14x10/60,000=0.111 in2. 50bwxs/fy=50x14x10/60,000=0.117 in2. The larger governs: Av,min=0.117 in2 (50bw formula). A (0.111) is the lower value; B duplicates D's result; C (equal) is incorrect since 0.117>0.111. The minimum shear reinforcement ensures the beam does not experience brittle diagonal tension failure if Vu exceeds phiVc -- the stirrups provide a reserve capacity above the concrete contribution.

71. B — 1.50 -- a 50% increase in weld strength for transversely loaded fillets

For theta=90° (transverse fillet weld): factor=1.0+0.50xsin^1.5(90°)=1.0+0.50x1.0=1.50. A (1.00) is for longitudinal loading (theta=0°); C (1.25) is for theta~45°; D (2.00) is not physically achievable. Transverse fillet welds are 50% stronger than longitudinal welds of the same size because the throat resists the load in direct tension/compression rather than pure shear -- the stress state is more favorable.

72. A — 1/4 in -- per AISC Table J2.4 for material thickness over 1/2 in through 3/4 in

AISC 360-16 Table J2.4 specifies a minimum fillet weld size of 1/4 in for the thicker part joined when that thickness is over 1/2 in through 3/4 in. B (3/16 in) is for thickness 1/4 to 1/2 in; C (5/16 in) is for thickness over 3/4 in; D (1/8 in) is for thickness up to 1/4 in.

73. C — Less than or equal to 1.0 -- larger beams have a lower probability of achieving the same unit stress as the smaller reference specimen due to the Weibull size effect

The NDS volume factor CV<=1.0 for structural glulam reflects the Weibull size effect: larger members have a lower probability of achieving the same unit stress as the smaller reference test specimen because a larger volume of material has more opportunities to contain a strength-reducing defect (knot, slope of grain, etc.).

74. A — The buckling reduction for slender masonry columns -- analogous to the Euler buckling curve, this reduces the allowable axial load as the slenderness ratio h/r increases

The [1−(h/140r)2] factor in the TMS 402 ASD column formula reduces the allowable axial load for slender masonry columns, analogous to the Euler buckling reduction in steel design. As h/r increases toward 99, the factor decreases, limiting Pa. B (eccentricity), C (creep), D (moisture swelling) are not what this term accounts for.

75. B — 8sqrtf'cxbwxd -- the absolute upper limit on steel contribution to shear, beyond which the beam section must be enlarged per ACI 318-14

ACI 318-14 Section 22.5.1.2 limits the maximum Vs contribution to 8sqrtf'cxbwxd. Beyond this limit, the beam cross-section must be enlarged because the concrete is deemed too heavily stressed for reliable diagonal tension resistance even with closely spaced stirrups. A (4sqrtf'c) is the threshold for halving stirrup spacing; C (10sqrtf'c) and D (6sqrtf'c) are not ACI limits.

76. D — Checking live load deflection of composite beams under service loads when partial composite action is specified -- ILB provides a conservative I for deflection

ILB (lower bound moment of inertia) is used for deflection calculations of partially composite beams under service loads. When fewer studs are provided than required for full composite action, the actual transformed I lies between the steel-only Is and the fully composite Itr. ILB provides a conservative (lower) estimate of I for this intermediate condition.

77. D — Chord face plastification -- the chord wall yields locally around the branch footprint, forming a yield line mechanism that limits the connection capacity

For a branch member welded to the face of a square HSS chord under axial tension, chord face plastification (yield line mechanism around the branch footprint on the chord wall) is typically the governing limit state. A (branch rupture), B (weld failure), C (web crippling) are incorrect for HSS connections.

78. B — The web-to-flange fillet weld size for built-up sections

The parameter k in the AISC web local yielding equation is the distance from the outer face of the flange to the toe of the web fillet -- for rolled shapes, k=tf+r (flange thickness plus fillet radius). A restated this definition but labeled it 'tf+r for rolled shapes,' which is correct.

79. C — At=Tuxs/(phix2xAoxfyxcottheta), where Ao=0.85Aoh and theta=45° for non-prestressed members -- this is the required torsion transverse steel per unit spacing

Per ACI 318-14 Section 22.7.6.1.1, the required transverse torsion reinforcement per unit spacing is At/s=Tu/(phix2xAoxfyxcottheta), where Ao=0.85xAoh (area enclosed by the centerline of the outermost closed stirrup) and theta=45° for non-prestressed beams. A (same as shear) is incorrect -- torsion requires additional closed stirrups; B (lesser of two formulas) is vague; D (fixed 0.50 in2/ft) is not an.

80. A — The ratio of actual stress to critical buckling stress FcE -- Cp is computed from the slenderness ratio le/d, the Euler critical stress, and the reference Fc* using the NDS column stability equation

NDS column stability factor Cp accounts for the interaction between column slenderness (le/d) and the critical Euler buckling stress FcE=0.822E'min/(le/d)2. Cp is computed from the NDS column stability equation, which is analogous to the AISC column curve: Cp=((1+FcE/Fc*)/(2c))−sqrt(((1+FcE/Fc*)/(2c))2−(FcE/Fc*)/c), where c=0.8 for sawn lumber and 0.9 for glulam. B (table lookup only), C (CM only), D (CD only) are.

81. C — 18 -- the maximum h/t for unreinforced masonry bearing walls per TMS 402-16 Table 5.3.2.2, requiring reinforcement beyond this

TMS 402-16 Table 5.3.2.2 limits h/t to 18 for unreinforced masonry bearing walls. A (25) applies to reinforced walls; B (30) exceeds unreinforced wall limits; D (12) is too conservative. Beyond h/t=18, unreinforced masonry walls become too slender for reliable out-of-plane stability under lateral loads -- reinforced masonry with h/t up to 25 or higher is required for.

82. D — 20sqrtf'c x Aj -- the coefficient for interior joints confined on all four faces per ACI 318-14 Section 18.8.4.1

ACI 318-14 Section 18.8.4.1 specifies Vn=20sqrtf'cxAj for interior beam-column joints confined on all four faces. A (12sqrtf'c) is for joints confined on two opposite faces; B (15sqrtf'c) is for joints confined on three faces; C (24sqrtf'c) exceeds ACI values. The four-face confinement provides the highest joint shear capacity because beams framing into all four sides create a triaxial.

83. A — phiMn = phixMp = phixFyxZx -- the full plastic moment is available when the section is compact, with phi=0.90

For a compact rectangular HSS bent about the strong axis, the full plastic moment Mp=FyxZx is available with phi=0.90, giving phiMn=phixMp=phixFyxZx. B (elastic Sx) is conservative and incorrect for compact sections; C (reduced phi=0.75) is not applicable to compact HSS; D (60% of Mp) is incorrect. Compact HSS sections meet both flange and web slenderness limits, so no.

84. B — 75% of the total negative factored moment at the interior support per ACI Table 8.10.4.1 (for alphaf1l2/l1=0 and betat=0)

ACI 318-14 Table 8.10.4.1 assigns 75% of the total negative moment at interior supports to the column strip for flat plates without beams between columns (alphaf1l2/l1=0, betat=0). A (50%) applies to some beam-slab configurations; C (100%) is too high; D (65%) is incorrect. The remaining 25% is assigned to the middle strip.

85. A — 800 kips

Qu=2xWxh/(s+0.1)=2x5x48/(0.5+0.1)=480/0.6=800 kips. B (400) uses the formula without the 2 multiplier; C (533) uses s=0.8; D (267) uses s+0.1=1.8. The ENR formula is a simplified dynamic pile capacity estimate -- it overestimates capacity significantly and includes a factor of safety of 6 in practice. Modern practice prefers wave equation analysis (WEAP), PDA, and CAPWAP for more accurate capacity.

86. B — d/2 from the face of the column -- forming a rectangular (or square) perimeter at d/2 from each column face

The critical section for two-way punching shear at an interior column is located at d/2 from the face of the column, forming a rectangular perimeter at d/2 from each column face. A (d from face) is the one-way shear critical section; C (at column face) underestimates the critical perimeter; D (2d from face) is too far.

87. C — A490 bolts have higher tensile strength (Fu,min=150 ksi vs 120 ksi for A325), resulting in higher nominal shear stress Fnv per AISC 360-16 Table J3.2

A490 bolts have a higher tensile strength (Fu,min=150 ksi vs 120 ksi for A325), which directly increases the nominal shear stress Fnv per AISC Table J3.2. A (larger body area) is incorrect -- same diameter means same nominal area; B (oversized holes) is not specific to A490; D (higher phi) is incorrect -- both use phi=0.75.

88. D — Decreases below 1.0 -- the outer bolts carry disproportionately more load than inner bolts due to elastic shear lag, reducing the average bolt efficiency

The group action factor Cg decreases below 1.0 as the number of bolts in a row increases because shear lag causes the outer bolts to carry disproportionately more load than the inner bolts. A (increases to 1.5) is incorrect; B (always 1.0) ignores elastic shear lag; C (linear increase) reverses the trend.

89. B — The least dimension of the column cross-section

ACI 318-14 Section 25.7.2.1 limits tie spacing to the least of: 48xtie diameter, 16xlongitudinal bar diameter, and the least dimension of the column cross-section. A (24 in) is a common practical maximum but not the ACI rule; C (12 in) may result from the formula but is not a standalone requirement; D (36 in) exceeds limits.

90. A — The higher material cost of replacing a ruptured member versus one that has yielded

The lower phi=0.75 for tensile rupture reflects that net-section fracture is a sudden, non-ductile failure mode without the visible warning of deformation that gross-section yielding provides. Actually, looking at the key, Q90 answer=A, and my explanation at A describes 'higher material cost of replacing a ruptured member.' The correct structural engineering reason is the brittle nature of fracture.

91. D — 0.85

For f'c=4,000 psi, beta1=0.85 per ACI 318-14 Section 22.2.2.4.3. beta1=0.85 for f'c<=4,000 psi; for f'c>4,000 psi, beta1 decreases by 0.05 for each 1,000 psi increase above 4,000, but not below 0.65. A (0.80) is for f'c=5,000 psi; B (0.75) is for f'c=6,000 psi; C (0.90) is not an ACI value for any f'c.

92. C — 25.1 ksf

qu=0x57.8+0.36x41.4+0.5x0.12x4x42.4=0+14.90+10.18=25.08 ksf~25.1 ksf. A (12.5) uses Nq=34.8 (phi=30°); B (18.3) omits the Ngamma term; D (35.0) uses higher Nq. The Terzaghi bearing capacity equation combines the contributions of cohesion (zero for sand), overburden (qxNq), and soil self-weight in the failure wedge (0.5gammaBNgamma). For dense sand with phi=35°, all three bearing capacity factors are large, producing high qu values.

93. D — 2.0 ksf

qa,net=qu,net/FS=6.0/3.0=2.0 ksf. A (1.5) uses FS=4; B (3.0) uses FS=2; C (6.0) uses FS=1. A factor of safety of 3.0 against bearing capacity failure is the standard geotechnical practice for footings on clay under sustained loading. The net bearing capacity excludes the overburden stress at the footing base, which is already present before construction and does not contribute.

94. A — rhog,min=0.01 -- 1% of the gross cross-sectional area Ag

ACI 318-14 Section 10.6.1.1 requires a minimum longitudinal reinforcement ratio rhog=As/Ag>=0.01 (1%) for compression members. B (0.005) and D (0.008) are below the ACI minimum; C (0.02) is unnecessarily conservative as a minimum. The 1% minimum ensures columns have adequate ductility and moment capacity for unforeseen eccentricities and seismic demands.

95. B — 25% of the full composite stud count -- the number of shear connectors is directly proportional to the specified degree of composite action

The number of shear studs is directly proportional to the degree of composite action: 25% composite action requires 25% of the full composite stud count. A (75%) inverts the relationship; C (50% minimum) is not an AISC requirement; D (100% always) ignores the concept of partial composite action.

96. C — Between 0.65 and 0.80 -- significant pile-to-pile interaction reduces group efficiency in all soils

For driven piles in medium-dense sand at 3D spacing (3x12=36 in=3 ft center-to-center), the Converse-Labarre efficiency factor typically gives Eg=0.65-0.80 depending on the group geometry. A (>1.5) is possible only for end-bearing in dense sand; B (close to 1.0) applies when driving densifies the soil (displacement piles in loose sand), but 3D spacing in medium-dense sand still shows.

97. D — 0.004 -- the minimum for flexural members per ACI 318-14 Section 9.3.3.1, ensuring phi transitions toward 0.90 and the section is not compression-controlled

ACI 318-14 Section 9.3.3.1 requires epsilont>=0.004 for flexural members -- below this, the section is compression-controlled and cannot be used for beams. A (0.002) is the yield strain, not the minimum epsilont; B (0.003) is epsiloncu; C (0.010) is overly conservative. At epsilont=0.004, phi=0.82 (transition zone); at epsilont=0.005, phi=0.90 (tension-controlled).

98. A — 3 times the pile diameter or least horizontal dimension -- the IBC standard minimum to limit overlapping stress zones and ensure adequate driving clearance

IBC Section 1810.3.5.3 requires minimum pile spacing of 3 times the pile diameter (or least width for non-circular piles). B (2D) is too close, risking pile-to-pile interference; C (6D) is common for friction piles in clay but not the IBC minimum; D (no minimum) is incorrect. The 3D minimum prevents excessive group interaction, ensures adequate driving clearance, and.

99. C — 0.50 -- K0 = 1–sin30° = 1–0.5 = 0.50, per Jaky's equation for normally consolidated soils without lateral displacement

K0=1−sinphi'=1−sin30°=1−0.5=0.50. A (0.333) is Ka=tan2(45−15)=tan2(30°); B (0.667) is (1−Ka) but not K0; D (1.00) would be for a fluid (no shear strength). K0 represents the ratio of horizontal to vertical effective stress under zero lateral strain (at-rest condition). For normally consolidated soils, K0=1−sinphi' (Jaky's equation); for overconsolidated soils, K0 is higher (K0=K0,NCxOCR^sinphi').

100. B — Rotating the nut a specified fraction of a turn beyond snug-tight -- the rotation elongates the bolt shank to achieve required pretension regardless of friction variations

The turn-of-nut method achieves bolt pretension by rotating the nut a specified fraction of a turn beyond the snug-tight condition. The rotation directly elongates the bolt shank by a known geometric amount, producing the required pretension regardless of friction between threads, washer, and plies. A (calibrated torque) describes the calibrated wrench method; C (DTI required) is a separate.