PE Civil Structural Exam Prep

Practice Exam 11

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
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5	Component Design (Structural Elements)	37
	Total	**100**

PRACTICE EXAM 11 -- QUESTIONS

DOMAIN 1 -- Loads (Q1-Q17)

1. Per ASCE 7-16 Section 7.9, the unbalanced snow load on a gable roof with equal slopes occurs when one side of the roof accumulates snow and the other is swept bare. The unbalanced case governs when the roof slope is between:

A. 1/2 on 12 and 7 on 12 -- the range where wind differentially removes snow from windward and deposits on leeward
B. 5 on 12 and 12 on 12 -- only steep roofs have unbalanced snow issues
C. Flat to 1/2 on 12 -- very flat roofs always accumulate uniformly
D. 2.38° and 30.26° (corresponding to slopes 1/2:12 and 7:12) -- ASCE 7-16 specifies this exact range

2. Per ASCE 7-16 Table 1.5-1, Risk Category III includes buildings where failure could result in substantial economic impact or mass disruption of civilian use. Which of the following is Risk Category III?

A. Occupied schools, colleges, and universities with occupant capacity greater than 250 -- listed in RC III for societal importance
B. Single-family detached residences -- standard RC II residential construction in most jurisdictions
C. Storage facilities for household goods not occupied 24 hours per day -- typically RC I
D. Power-generating stations and utility infrastructure -- classified as RC IV essential facilities

3. Per ASCE 7-16 Section 4.8, a partition dead load of _____ psf minimum must be included in the floor live load for office buildings where partitions may be installed or rearranged, regardless of whether partitions are shown on the construction documents:

A. 10 psf
B. 15 psf
C. 20 psf
D. 25 psf

4. Per ASCE 7-16 Section 12.2.5.4, structures with plan irregularity Type 1b (extreme torsional irregularity) in SDC D, E, or F:

A. May use the ELF (Equivalent Lateral Force) procedure without restriction
B. May use the simplified method of Section 12.14 only if the structure is also regular in elevation
C. Are not permitted to use the ELF procedure or Response Spectrum Analysis -- only nonlinear analysis is allowed
D. Are prohibited under ASCE 7-16 -- extreme torsional irregularity cannot be corrected by analysis alone and buildings must be redesigned

5. Per ASCE 7-16 Section 26.7, Exposure Category C applies to terrain with:

A. Open terrain with scattered obstructions of heights generally less than 30 ft -- includes flat open country and grasslands
B. Urban and suburban areas with numerous closely spaced obstructions having heights generally >=30 ft
C. Large bodies of water or shorelines in hurricane-prone regions
D. Agricultural areas with rows of trees and 10-ft corn rows serving as windbreaks

6. Per ASCE 7-16 Table 26.7-1, which Exposure Category produces the HIGHEST Kz velocity pressure exposure coefficient for a building 40 ft tall?

A. Category A -- sheltered by very rough terrain
B. Category B -- suburban terrain
C. Category D -- open water and shorelines with fetch > 5,000 ft
D. Category C -- open terrain with scattered obstructions

7. Per ASCE 7-16 Section 12.4.3, the maximum axial compression load on a structural element under the seismic load combination 1.2D+f1xL+1.0E+0.2S (where the Omega0 overstrength factor amplifies E for certain elements such as columns supporting discontinuous walls) uses Omega0 to:

A. Reduce the design force to account for ductile energy dissipation in SMF columns per ASCE 7-16 Chapter 12 seismic provisions

B. Account for accidental torsion effects in buildings with irregular floor plans as a direct multiplier

C. Reduce the structural importance factor Ie below the standard value for lower-risk structures

D. Amplify the horizontal seismic force Eh for elements expected to remain elastic while the SFRS yields -- ensuring the load path is maintained when yielding occurs

8. Per ASCE 7-16 Section 4.7.3, a beam with KLL=2 and tributary area AT=400 ft2 supports an unreduced live load Lo=50 psf. The reduced live load L=Lox(0.25+15/sqrt(KLLxAT)) is:

A. 40 psf

B. 39.0 psf

C. 25 psf

D. 50 psf (no reduction permitted)

9. Per ASCE 7-16 Eq. 12.8-4, for buildings with T>TL (long-period range), the upper-bound Cs=SD1xTL/(T2xR/Ie). For SD1=0.5g, TL=12 s, T=5 s, R=6, Ie=1.0, what is the upper-bound Cs?

A. 0.050

B. 0.025

C. 0.040

D. 0.083

10. Per ASCE 7-16 Section 12.3.4.1, structures in SDC D, E, or F with vertical structural irregularity Type 5 (nonparallel system irregularity -- lateral force-resisting elements are not parallel or symmetric about the major orthogonal axes) must:

A. Use the Equivalent Lateral Force procedure without any restriction on irregularity

B. Reduce the R factor by 20% for all seismic calculations

C. Use the three-dimensional analysis procedure that considers seismic loading applied simultaneously in any horizontal direction

D. Add shear walls to eliminate the irregularity before performing any seismic analysis

11. Per ASCE 7-16 Table 7.4-1, the exposure factor Ce for a structure in a fully exposed location (Exp C terrain, fully exposed) is:

A. Ce = 0.9 -- exposure reduces the snow load for windswept areas

B. Ce = 1.0 -- the standard value for all exposure conditions

C. Ce = 1.1 -- partially sheltered conditions

D. Ce = 1.2 -- fully sheltered, surrounded by trees or structures

12. Per ASCE 7-16 Section 27.3.1, the design wind pressure on the windward wall of a building using the Directional Procedure (Method 2) is p=qzxGxCp – qix(GCpi). The 'qi' term in the internal pressure calculation for a building with all openings closed uses:

A. The velocity pressure evaluated at mid-height of the building -- the most common simplified approach

B. qi = qh -- the velocity pressure at the mean roof height h, used for internal pressure calculations regardless of the location of the opening

C. qi = q10 -- the velocity pressure at 10 meters (33 ft) reference height per ISO standard

D. qi = qz at the centroid of the windward wall -- proportional to the wall mid-height velocity pressure

13. Per ASCE 7-16 Section 12.8.4, the vertical distribution of the seismic base shear V uses the exponent k, which equals:

A. k=1.0 for T<=0.5s (linear), k=2.0 for T>=2.5s (parabolic), linearly interpolated between 1.0 and 2.0 for 0.5s<T<2.5s -- distribution shifts from triangular to parabolic as period increases

B. k=1.0 for all structures regardless of period -- the linear triangular force distribution is always used in ASCE 7-16

C. k=2.0 for all structures -- ASCE 7-16 adopted a parabolic distribution for all building periods

D. k=T/2.5 -- the exponent is strictly proportional to the fundamental period for all structures

14. Per ASCE 7-16 Section 12.14.8.1, the simplified design base shear V=FxW/R for the simplified analysis procedure (Section 12.14) uses what value for the soil factor F?

A. F = 1.0 for all site classes -- the simplified procedure does not account for site amplification

B. F = SDS -- the design spectral response acceleration at short periods, same as used in the ELF procedure

C. F depends on the mapped seismic hazard spectral value Ss and the site class (ranges from 1.0 to 2.5) -- the simplified method accounts for site effects through F rather than separate Fa and Fv factors

D. F = 2/3 x SMS -- two-thirds of the MCER short-period spectral acceleration

15. Per ASCE 7-16 Section 6.1, flood loads F must be combined with other loads using the LRFD combination including 1.0F. For structures subject to high-velocity wave action (Zone V), the flood load includes:

A. Hydrostatic pressure from still-water flooding only

B. Only wave run-up forces determined by the ASCE 7-16 wave height formulas

C. Hydrostatic, hydrodynamic, wave, and impact loads -- the combined flood effect in coastal high hazard areas per ASCE 7-16 Chapter 5

D. A uniform equivalent lateral pressure equal to the 100-year flood water depth in feet times 62.4 psf

16. Per ASCE 7-16 Table 12.3-1, a horizontal structural irregularity Type 4 (out-of-plane offsets irregularity) exists when:

A. The story stiffness is less than 70% of the story above or less than 80% of the average stiffness of the three stories above

B. The sum of story shears resulting from the lateral force distribution does not equal the total design base shear

C. Story strength is less than 80% of the story above -- a significant soft story condition

D. Discontinuities in the lateral force path occur through the diaphragm -- offsets of shear walls, braced frames, or moment frames out of plane from the level above or below

17. The P-delta stability coefficient theta per ASCE 7-16 Section 12.8.7 is defined as theta = PxxDelta/(VxxhsxxCd/Ie). When theta exceeds 0.10, the designer must:

A. Consider the P-delta effects in design -- either amplify the story shear and moment by 1/(1−theta) or redesign the structure to reduce theta below 0.10

B. Immediately redesign the structural system -- theta>0.10 is a code violation that cannot be corrected by analysis

C. Double the seismic design forces to account for the increased instability demand

D. Use only nonlinear dynamic analysis -- ELF is not permitted when theta exceeds 0.10

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

18. Per ASCE 7-16 Eq. 27.3-1, the wind velocity pressure is qz=0.00256xKzxKztxKdxKexV2. For V=115 mph, Kz=0.98 (Exposure C at 40 ft), Kzt=1.0, Kd=0.85, Ke=1.0, what is qz?

A. 21.5 psf

B. 24.7 psf

C. 28.2 psf

D. 31.6 psf

19. For a propped cantilever (span L, fixed at A, roller at B) with a concentrated load P at distance a=L/3 from A, the compatibility equation for the redundant reaction at B is RBxf11=Delta10. The flexibility coefficient f11 (deflection of the simply-supported primary structure at B due to a unit upward load at B) is:

A. L2/(2EI) -- the slope at the free end of a cantilever beam under a unit tip load, not a deflection formula
B. L2/(6EI) -- an expression related to beam slope at mid-span under certain boundary conditions
C. L/(3EI) -- a dimensionally inconsistent expression for a length-to-stiffness ratio
D. L3/(3EI) -- the tip deflection of a cantilever under a unit load at the free end, used as f11 for the propped cantilever

20. For a two-span continuous beam (each span L=24 ft, EI=constant, both spans loaded with w=3 kip/ft), the moment at the interior support by the three-moment equation is:

A. –wL2/8 = –3x576/8 = –216 kip-ft -- the three-moment equation result for equal spans under uniform load
B. –wL2/8 = –216 kip-ft -- same result, confirmed by Clapeyron's theorem for two equal uniformly loaded spans
C. –108 kip-ft -- half of the three-moment equation result, incorrectly applied
D. –wL2/12 = –144 kip-ft -- the fixed-end moment formula for a fixed-fixed beam, not a two-span continuous beam

21. In the slope-deflection method, the slope-deflection equation for a member AB (near end A) with far end B fixed, chord rotation ψ=Delta/L, and load-caused fixed-end moment FEMAB is MAB=2EK(2thetaA+thetaB–3ψ)+FEMAB where K=I/L. If there is no chord rotation (ψ=0) and no external loads (FEM=0), and if thetaB=0 (B is fixed), then MAB is:

A. MAB = 2EKxthetaA -- proportional to the near-end rotation only when far end is fixed and no chord rotation
B. MAB = 4EKthetaA -- the near-end stiffness coefficient times the near-end rotation (2EKx2thetaA=4EKthetaA)
C. MAB = EKthetaA -- the simplest slope-deflection case
D. MAB = 6EKthetaA -- when the chord rotation is twice the near-end rotation

22. Using the portal method for lateral load analysis of a two-story, two-bay frame, the interior column at the second floor receives a horizontal shear equal to:

A. V_story/(n_columns) -- equal share for all columns
B. V_story/4 -- the four columns each get one-quarter
C. V_story/2 -- the interior column carries twice the shear of each exterior column
D. 2xV_story -- the interior column carries the full story shear since it is the critical column

23. For a frame with sidesway, the chord rotation ψ=Delta/L in the slope-deflection equations represents the lateral drift of the story. For a sway frame with both columns of equal height h and a story sway Delta, the chord rotation for EACH column is:

A. ψ = 0 -- sway frames do not use chord rotation in the slope-deflection method
B. ψ = Delta/(2h) -- the sway divided by twice the story height
C. ψ = h/Delta -- inverted ratio of height to sway
D. ψ = Delta/h -- the story sway divided by the column height, the same for both columns

24. For a planar truss with all members and loads in one plane, the maximum number of independent equilibrium equations available at each joint is:

A. 2 (ΣFx=0 and ΣFy=0) -- two force equilibrium equations per joint
B. 3 -- two force plus one moment equation at each node
C. 1 -- only vertical equilibrium at each joint
D. 4 -- for a 3D structure reduced to 2D

25. For a statically determinate beam (SS), the bending moment M and shear force V at any section are related by the differential equation dM/dx = V. A corollary is that the bending moment is maximum where:

A. The shear force is zero -- the bending moment reaches a local maximum or minimum where the shear passes through zero (changes sign)

B. The applied load intensity w is maximum -- the load controls the moment peak

C. The beam is at its deepest cross-section -- geometric properties govern the moment maximum

D. The slope of the elastic curve is maximum -- a steep tangent means high moment

26. The moment area method's first theorem states that the change in slope between two points on an elastic curve equals the area of the M/(EI) diagram between those points. For a cantilever (fixed at A, free end B) with a UDL w, the slope at the free end B (relative to the zero-slope tangent at A) is:

A. theta_B = wL3/(6EI) -- the slope formula for a cantilever under UDL

B. theta_B = wL2/(2EI) -- the area of the triangular M/EI diagram under UDL for a cantilever

C. theta_B = wL/(EI) -- a simpler estimate

D. theta_B = wL4/(8EI) -- the deflection at the free end, not the slope

27. The stiffness matrix for a 2D beam element in local coordinates has a 4x4 form (for axial excluded, just transverse DOFs). The diagonal term k11 (relating a unit transverse displacement at node 1 to the reaction force at node 1) for a prismatic beam clamped at both ends is:

A. 12EI/L3 -- the stiffness relating end lateral displacement to force, consistent with fixed-fixed boundary conditions

B. 3EI/L3 -- the stiffness for a cantilever beam (one free end)

C. 6EI/L2 -- a moment-force coupling term

D. EI/L -- a simplified dimensionless stiffness

28. In elastic plate theory (Kirchhoff plate bending), the flexural rigidity D of a plate is defined as D=Et3/(12(1−nu2)). This parameter D plays the same role in plate bending as:

A. The polar moment of inertia J in torsion problems -- D controls twisting capacity

B. The cross-sectional area A in axial problems -- D controls in-plane stretching

C. The shear modulus G in beam problems -- D scales the transverse shear stiffness

D. EI in beam bending -- D is the plate's bending stiffness per unit width, analogous to the beam bending stiffness EI

29. For an arch structure, the internal horizontal thrust H can be completely eliminated by providing a tie member connecting the two supports. The resulting tied arch:

A. Zero internal forces result when the tie is added -- the tie eliminates all arch compression and restores zero-stress behavior

B. The arch converts to a simply supported beam when the tie is installed -- the arch action is entirely replaced by beam action

C. The tie carries the horizontal thrust internally, so foundations require only vertical reactions -- eliminating the need for horizontal abutment resistance

D. The tie destabilizes the arch -- ties are prohibited in arch construction because they interfere with the primary compression mechanism

30. For a fixed-end beam (fixed at both ends, span L) with a concentrated load P at midspan, the midspan deflection δ_midspan=PL3/(192EI) is ______ the midspan deflection of a simply supported beam under the same load PL3/(48EI):

A. 1/4 of -- fixed-end conditions produce fixed-end moments PL/8 that counteract applied moment, reducing midspan deflection to one-quarter of the SS case

B. 1/2 of -- fixed-end conditions reduce midspan deflection to half of the simply-supported case by providing partial moment resistance

C. Equal to -- the fixed ends do not affect the midspan deflection because the concentrated load magnitude is unchanged

D. 3/4 of -- the fixed ends provide only partial restraint, reducing midspan deflection by 25% compared to the SS case

31. For a three-hinged arch (hinges at both supports and at the crown), under a uniform load w on the full span, the horizontal thrust H and the reaction at each support are:

A. RA=RB=wL/2 (vertical), H=wL2/(8f) -- same as the cable with the same span and sag f

B. RA=RB=wL/4, H=wL2/(4f)

C. RA=RB=wL/2, H=wL2/(4f)

D. RA=wL, RB=0, H=wL2/(8f) -- one support takes all vertical load

32. For a solid triangular cross-section (base b=6 in, height h=12 in, neutral axis at 2h/3=8 in from base=4 in from apex), the plastic section modulus Z about the centroidal axis equals bh2/24. What is Z?

A. 72 in3

B. 36 in3

C. 144 in3

D. 18 in3

33. For a uniformly loaded simply supported beam, the elastic curve y(x) is a fourth-degree polynomial. The boundary conditions used to determine the four constants of integration are:

A. y(0)=0, y''(0)=0, y(L)=0, y''(L)=0 -- zero deflection and zero moment at both supports

B. y(0)=0, y'(0)=0, y(L)=0, y'(L)=0 -- zero deflection and zero slope at both supports

C. y(0)=0, y''(0)=M/EI, y(L)=0, y''(L)=0 -- non-zero end moment at left support

D. y(0)=0, y'(0)=Max, y(L)=0, y'(L)=–y'(0) -- symmetric slope conditions

34. A fixed-base cantilever column (height L) is subjected to a horizontal force H at the top. The bending moment diagram is:

A. Parabolic -- horizontal force applied at the tip creates a parabolic moment diagram due to the varying moment arm along the height

B. Zero -- horizontal tip forces produce no bending in a fixed base column because the force is not eccentric to the column axis

C. Constant (HxL) throughout the height -- the moment diagram is uniform from base to top for a tip horizontal force

D. Linearly varying from M=HxL at the base to M=0 at the top -- the moment arm decreases linearly from tip to base

35. The principle of virtual work for a rigid body states that the virtual work done by all external forces through a virtual displacement consistent with the constraints equals:

A. The total virtual work equals the strain energy stored in the body during the virtual displacement -- conservation of energy applies in rigid body statics

B. The virtual work equals the kinetic energy acquired from applied forces -- consistent with Newton's laws of motion

C. Zero -- for a body in static equilibrium, the virtual work of all external forces through any virtual displacement consistent with constraints equals zero

D. The product of the resultant force and the maximum virtual displacement -- a simplified scalar measure of virtual work

36. For a beam with a concentrated moment M0 applied at midspan (not a concentrated force but a pure moment), the bending moment diagram consists of:

A. A smooth parabolic curve with peak at midspan where M0 is applied

B. Two straight lines forming a triangular shape -- a discontinuity (jump) of magnitude M0 at the application point, with reactions providing the linear slopes

C. A constant moment M0 throughout the full span -- pure moments are uniformly distributed in beam theory

D. A rectangular shape -- the moment is M0 to the left and –M0 to the right of the application point

37. For a thin-walled open section (such as an I, channel, or angle), the St. Venant torsional stiffness GJ is generally much lower than for a closed section of similar dimensions. This means that open sections under torsion primarily resist torsion by:

- **A.** Warping torsion (or nonuniform torsion) -- bending of the flanges about their own axes provides the dominant resistance when warping is restrained
- **B.** Membrane action across the web -- the thin web carries all torsion through shear flow
- **C.** Pure shear flow around the perimeter -- same mechanism as closed sections but with lower efficiency
- **D.** Rigid-body rotation -- open sections do not deform under torsion, they rotate as rigid bodies

38. The influence line for the reaction at support A of a simply supported beam (span L) has the shape of:

- **A.** A straight line from 1.0 at A to 0 at B -- the reaction at A decreases linearly as the unit load moves from A to B
- **B.** A parabola -- the reaction varies nonlinearly with load position
- **C.** A constant value of 0.5 -- the reaction at A is always half the load
- **D.** A triangle peaking at L/2 -- maximum reaction at A occurs when load is at midspan

39. Using the portal method for a two-story, two-bay frame with a lateral load of 60 kips at the second floor and no additional load at the roof, the overturning moment at the base of the EXTERIOR columns (for one column) using portal method assumptions (each bay treated as a portal) is:

- **A.** 60 kip.ft
- **B.** 75 kip.ft
- **C.** 90 kip.ft
- **D.** 120 kip.ft

40. The compatibility equation for an indeterminate truss (one redundant member) using the force method is δ_10 + X1xδ_11 = 0, where δ_10 is the relative displacement of the redundant member's endpoints in the primary (cut) truss due to external loads, and δ_11 is the displacement per unit value of X1. The sign convention means X1 (tension positive) is:

- **A.** X1 = –δ_10/δ_11 -- the tension force in the redundant such that the gap (δ_10) is closed by the redundant action (X1xδ_11)
- **B.** X1 = δ_10/δ_11 -- direct ratio without sign change
- **C.** X1 = 0 when δ_10 = 0 only -- otherwise the redundant is indeterminate
- **D.** X1 = δ_11/δ_10 -- inverse ratio

41. For a simply supported beam with a partial uniform load w over the left half of the span only, the location of maximum bending moment (where shear=0) is:

- **A.** At x=3L/8 from the left support -- the shear from the partial UDL changes sign here, locating the moment maximum
- **B.** At midspan L/2 -- the loading is symmetric so the shear and moment peaks coincide at the center
- **C.** At the left quarter-point L/4 -- the partial UDL ends here, creating a shear discontinuity and moment peak
- **D.** At the right support -- the reaction there is maximum under partial loading, locating the moment peak

42. The direct stiffness method (DSM) for a 2D frame assembles the global stiffness matrix by transforming each element stiffness matrix from local to global coordinates. The key mathematical operation for this transformation is:

- **A.** [T]x[ke]x[T]■ -- pre and post-multiplied by the inverse transpose of the transformation matrix
- **B.** [T]■x[ke]x[T] -- the standard transformation using the transpose of the rotation matrix [T]
- **C.** [T]x[ke]/det([T]) -- normalizing by the determinant of the transformation matrix
- **D.** det([ke]) -- the determinant of the element stiffness controls the global assembly

DOMAIN 3 -- Temporary Structures (Q43-Q49)

43. Per ACI 347R-14, the formwork for a concrete wall must be designed to resist the lateral pressure of fresh concrete. For a wall with concrete placed at a rate R=4 ft/hr, concrete unit weight wc=150 pcf, concrete temperature T=70°F, and column height H=14 ft, which formula applies (CW=1.0, CC=1.0)?

A. p = wcxH = 150x14 = 2,100 psf -- full hydrostatic pressure governs

B. p = CWxCCx(150+9,000R/T) = 1.0x1.0x(150+9000x4/70) = 150+514=664 psf, but not less than 600 psf

C. p = wcxH/2 = 1,050 psf -- half hydrostatic for walls placed in one lift

D. p = CWxCCx(150+9,000R/T) = 664 psf, limited to wcxH = 2,100 psf -- so p = 664 psf

44. Per OSHA 29 CFR 1926.650(b), when must an engineer design the support system for an excavation?

A. Always -- all excavation support systems require engineering design

B. Only for trenches deeper than 20 ft -- shallower trenches may use tabulated data

C. When the excavation is adjacent to a structure or highway that may be affected by the work, or when the excavation exceeds the limits of the sloping tables

D. Only for excavations in Type C soil (unstable, granular) -- Type A and B soil can use standard tables

45. Per ACI 305.1-14, cold weather concrete protection is required when the ambient temperature is expected to fall below:

A. 32°F (0°C) -- the freezing point of water at which concrete freeze protection begins

B. 40°F (4°C) -- ACI 306R defines cold weather as mean daily temperature below 40°F for more than 3 consecutive days, requiring protection measures

C. 50°F (10°C) -- the standard lower bound for normal cement hydration rate without cold weather concerns

D. 28°F (–2°C) -- below the actual freezing point of concrete with standard water-reducing admixtures

46. Per ACI 347R-14, the minimum design live load for formwork construction (for personnel, equipment, and materials) is:

A. 50 psf on the horizontal projected area of the form -- a much higher live load than ACI requires for standard formwork

B. 25 psf in areas where active concrete placement operations are occurring -- moderate but above ACI minimum

C. 75 psf for all forms supporting concrete during placement -- overly conservative for typical formwork design

D. 20 psf -- the ACI 347R-14 minimum construction live load for formwork, combined with concrete dead load for the total design pressure

47. The safe working load (SWL) for a crane lift depends on the boom length, boom angle, and radius (horizontal distance from the crane centerline to the load). As the lift radius increases (boom swings out farther), the crane's rated capacity:

A. Increases -- the longer reach gives the crane more mechanical advantage to lift heavier loads at greater distance

B. Remains constant -- crane rated capacity is a fixed specification independent of boom angle or lift radius

C. Decreases -- the overturning moment about the tipping fulcrum increases with radius, reducing safe lifting capacity at greater radii

D. Doubles with every 50% increase in radius -- a proportional linear relationship between radius and capacity

48. Per OSHA 29 CFR 1926.454, employees working on scaffolds must receive training from a qualified person on scaffold hazards including:

A. Scaffold erection and disassembly procedures, electrical hazards, fall protection, and falling object protection -- workers must be trained before working on scaffolds

B. Only the maximum scaffold height and load capacity -- no other training is required

C. Scaffold inspection procedures but not personal protective equipment

D. Scaffold load capacity only -- the competent person determines other scaffold requirements

49. Per ACI 318-14 Section 26.5.2 and ACI 347R, concrete should be deposited as close as practical to its final position, not moved by vibration into forms. Displacement of concrete by vibration:

A. Is beneficial for distribution and uniformity -- vibration improves concrete density and helps fill all voids in the formwork

B. Should be used only for thin wall forms where direct placement by pump or chute is physically difficult

C. Is required for SCC (self-consolidating concrete) to achieve adequate workability and flow into congested areas

D. Should be avoided because it can cause segregation of aggregates from paste, especially in high-water-content mixes

DOMAIN 4 -- Materials (Q50-Q63)

50. Per ACI 318-14 Section 19.2.1.1, the minimum specified compressive strength f'c for structural concrete used in frames, slabs, and columns in most building applications is:

A. 2,000 psi -- the historical minimum

B. 2,500 psi

C. 3,000 psi -- the ACI 318-14 minimum for structural concrete in most applications

D. 4,000 psi -- required for all seismic applications regardless of exposure

51. Per NDS 2018 Section 11.1.3, the load duration factor CD for a design load combination that includes wind or seismic loads (in combination with dead load) is:

A. CD = 0.9 -- permanent (dead) loads only

B. CD = 1.6 -- the highest CD factor in NDS, applied to combinations including wind or seismic loads

C. CD = 1.15 -- applies when snow load governs the combination

D. CD = 1.25 -- applies to combinations including occupancy live loads only

52. The Atterberg limits used in soil classification include the liquid limit (LL), plastic limit (PL), and shrinkage limit (SL). The plasticity index PI is defined as:

A. PI = LL − PL -- the range of water content over which the soil is in a plastic (moldable) state

B. PI = PL − SL -- the range of water content between plastic and shrinkage limits

C. PI = LL/PL -- the ratio of liquid to plastic limits

D. PI = (LL + PL)/2 -- the average Atterberg limit

53. Per ASTM A992, W-shape structural steel (wide-flange sections used as beams and columns) has:

A. Fy = 36 ksi minimum, Fu = 58 ksi minimum -- the original structural steel specification

B. Fy = 42 ksi minimum, same as A500 Grade B HSS

C. Fy = 50 ksi minimum, maximum Fy/Fu <= 0.85 (Fu/Fy >= 1.18), and Fy <= 65 ksi -- properties that ensure adequate ductility for seismic applications

D. Fy = 50 ksi minimum with no maximum yield strength limit -- any high-strength W-shape qualifies

54. Per ACI 318-14 Section 26.4.1.1, the minimum curing period for concrete made with Type I Portland cement at a temperature above 50°F is:

A. 3 days

B. 7 days -- the standard minimum curing period to achieve adequate strength development

C. 14 days

D. 28 days -- the full curing period for final strength testing

55. Per ASTM D698 (Standard Proctor Test), the optimum moisture content (OMC) for maximum dry density is determined by:

A. Compacting soil samples at different water contents and plotting dry density vs moisture content -- OMC is the moisture content corresponding to the peak of the curve

B. Measuring the natural moisture content of the soil in the field before compaction

C. Using the plastic limit of the soil as the OMC -- soils compact best at their plastic limit

D. Calculating OMC = (LL + PL)/3 -- the average of Atterberg limits determines compaction OMC

56. Per TMS 402-16, for reinforced masonry shear walls, the minimum vertical (longitudinal) reinforcement ratio rhov must be at least 0.0013 (ordinary walls) per ACI 318 or, for SPECIAL reinforced masonry shear walls (SDC D/E/F):

A. rhov = 0.0007 minimum, and (rhov + rhoh) >= 0.002 -- the intermediate wall requirement

B. rhov = 0.0025 minimum vertical AND rhoh = 0.0025 minimum horizontal, independently -- both directions must independently meet 0.0025

C. rhov = 0.0015 minimum -- a modest increase from ordinary walls

D. rhov = 0.002 total combined with rhoh -- only the combined total matters, not individual directions

57. Per ASTM C150, Type II Portland cement is designed for:

A. General-purpose construction where high early strength is not required

B. High early strength -- Type II gains strength rapidly within the first week

C. Moderate sulfate resistance and moderate heat of hydration -- suitable for structures exposed to mild sulfate environments or large concrete masses

D. Very low heat of hydration -- Type II produces the least heat of all Portland cement types

58. Per ACI 318-14 Table 20.6.1.3, the minimum clear cover for cast-in-place concrete members (beams, columns) that are NOT exposed to weather or in contact with ground, for a #6 or larger bar is:

A. 1.5 in -- the standard clear cover for interior beams and columns with #6 or larger bars not exposed to weather

B. 2.0 in -- required for members exposed to freezing and thawing or exposure to deicing chemicals

C. 2.5 in -- the minimum for members exposed to soil or water in moderate conditions

D. 3.0 in -- required for members in contact with ground where concrete is placed directly on soil

59. NDS 2018 Table 4A provides reference design values for sawn lumber. The reference bending design value Fb for 2x10 Select Structural Douglas Fir-Larch (used in single-member floor joists) is approximately:

A. 1,500 psi -- a reasonable value for Douglas Fir-Larch No. 1 or No. 2 grade dimension lumber

B. 1,200 psi -- characteristic of Spruce-Pine-Fir or lower-grade Douglas Fir framing lumber

C. 1,000 psi -- typical of lower-grade lumber or Southern Yellow Pine No. 2 dimension lumber

D. 1,700 psi -- a representative NDS Supplement value for Douglas Fir-Larch Select Structural 2x10 in single-member use

60. Per AISC 360-16 Table J1.5, the minimum fillet weld size for a base plate with a connected part thickness of 1 in is:

A. 1/8 in

B. 5/16 in

C. 3/8 in

D. 1/2 in

61. Per ACI 318-14 Section 24.2.3.4, for one-way slabs and beams, the maximum center-to-center spacing of flexural reinforcement (to control crack widths) for Grade 60 bars used in members exposed to typical interior conditions is limited to the lesser of:

A. s <= 15(40,000/fs)–2.5cc, but not greater than 12(40,000/fs) -- the ACI 318-14 crack control formula governing maximum bar spacing

B. s <= 18 in or 3h -- where h is the member thickness

C. s <= 12 in for all conditions regardless of bar size

D. s <= 40,000/fsx(cc) where fs=service stress, cc=concrete cover -- based on crack width control

62. Per AISC 360-16 Section E3, the critical stress Fcr for a compact, doubly symmetric column in the inelastic buckling range (KL/r <= 4.71sqrt(E/Fy)) is Fcr=0.658^(Fy/Fe)xFy, where Fe=pi2E/(KL/r)2. The 0.658 base in this formula comes from:

A. A statistical analysis of column test data that established the relationship between theoretical Euler load and actual test failure loads

B. The ratio of the yield stress to the elastic modulus for A36 steel at room temperature

C. The Ramberg-Osgood parameter for strain hardening in structural steel

D. An empirical calibration to test data that captures the combined effects of initial crookedness, residual stresses, and material variability on real column capacity

63. ASTM A615 Grade 60 deformed reinforcing bar has a minimum yield strength Fy=60 ksi and a minimum tensile strength Fu=90 ksi. The minimum elongation at fracture required is:

A. 18% for all bar sizes -- a very stringent elongation requirement applicable to seismic reinforcement per ASTM A706

B. 9% in 8 in. for #3 to #5 bars, 8% for #6 to #11 bars -- per ASTM A615 Table 1 minimum elongation requirements

C. 12% uniform for all bar sizes -- a single consistent elongation floor for standard structural applications

D. 5% minimum for non-seismic bars -- the lowest elongation allowed for general-purpose deformed reinforcing bars

DOMAIN 5 -- Component Design (Q64-Q100)

64. Per ACI 318-14, the development length ld for a #5 straight deformed bar (db=0.625 in) in tension with fy=60 ksi, f'c=4,000 psi, all ψ modification factors=1.0, and (cb+Ktr)/db=2.5 is ld=(3/40)x(fy/(lambdasqrtf'c))x(ψtxψexψsxψg/((cb+Ktr)/db))xdb. What is ld?

A. 28.5 in -- the #8 bar value, incorrectly applied to #5

B. 21.4 in -- using (cb+Ktr)/db=2.0

C. 17.8 in -- computed as 3/40x60000/(63.25)x1.0/2.5x0.625

D. 9.4 in -- using the compression development formula

65. A simply supported rectangular beam has b=12 in, d=22 in, As=3.0 in2, f'c=4 ksi, fy=60 ksi. The depth of the equivalent rectangular stress block a=Asxfy/(0.85xf'cxb) and phiMn=phixAsxfyx(d–a/2). What is phiMn?

A. 300 kip.ft

B. 351 kip.ft

C. 267 kip.ft

D. 230 kip.ft

66. Per ACI 318-14 Section 9.6.3.3, the minimum shear reinforcement requirement Av,min/(bwxs) = max(0.75sqrtf'c/fy, 50/fy) applies to beams where Vu > phiVc/2. For f'c=4,000 psi and fy=60,000 psi, the governing minimum Av/(bwxs) is:

A. 0.00079 -- from the 0.75sqrtf'c/fy term

B. 0.00083 -- from the 50/fy term (50/60,000=0.000833)

C. 0.00125 -- double the 50/fy value

D. 0.00055 -- one-half the standard minimum

67. Per ACI 318-14 Section 18.6.4.2, in a special moment frame (SMF) beam, the maximum spacing of hoops within a distance 2h from the face of the column (in the plastic hinge zone) shall not exceed:

A. d/4 -- the tightest spacing permitted for any beam

B. s = 6db (longitudinal bar diameter) -- the maximum SMF hoop spacing in the plastic hinge zone per ACI 318-14 Section 18.6.4.2

C. 8 in -- an absolute maximum regardless of bar size

D. s = min(d/4, 6db, 6 in) -- the least of three criteria governs the SMF hoop spacing

68. Per ACI 318-14 Section 22.5, the shear strength of a concrete wall (acting as a shear wall under seismic loading) with a slender aspect ratio hw/lw>2 uses Vn=Acvx(alphacxlambdaxsqrtf'c + rhotxfy), where alphac=2.0 for hw/lw>=2. What does the rhot term represent?

- **A.** The vertical reinforcement ratio in the shear wall -- the vertical bars contribute to shear resistance
- **B.** The transverse reinforcement ratio in the coupling beams above the shear wall
- **C.** The longitudinal (horizontal) shear reinforcement ratio in the wall (Avh/Acv) -- horizontal reinforcement crossing the potential diagonal crack resists shear in slender walls
- **D.** The boundary element reinforcement ratio -- boundary elements carry shear at the wall ends

69. For a doubly reinforced concrete beam (As for tension, A's for compression), the purpose of compression steel A's is primarily to:

- **A.** Reduce long-term deflection from concrete creep -- compression steel resists sustained compression, reducing the creep multiplier lambdaDelta=ξ/(1+50rho')
- **B.** Increase the nominal moment capacity Mn beyond singly reinforced limits by providing a compression force couple
- **C.** Replace crushed concrete in the compression zone during extreme seismic loading where the cover spalls away
- **D.** Provide confinement for the main flexural reinforcement at anchorage and splice locations near column faces

70. Per AISC 360-16 Section G2.1, for a W18x55 (A992, Fy=50 ksi) with h/tw=35.7 (<=2.24sqrt(E/Fy)=53.9), the design shear strength phiVn=1.0x0.6FyxAw where Aw=dxtw=18.11x0.390. What is phiVn?

- **A.** 149 kips
- **B.** 185 kips
- **C.** 212 kips
- **D.** 240 kips

71. Per AISC 360-16 Section F2, the compact flange criterion for a W-shape is bf/(2tf) <= lambdapf=0.38sqrt(E/Fy). For a W16x26 (bf=5.50 in, tf=0.345 in) with Fy=50 ksi:

- **A.** The flange is non-compact -- bf/(2tf)=5.50/(0.690)=7.97 > lambdapf=9.15, but lambdapf governs compactness
- **B.** lambdapf=0.38sqrt(29000/50)=9.15 and bf/(2tf)=7.97, confirming the flange is compact per AISC 360-16
- **C.** The flange is non-compact -- exceeds the compact limit
- **D.** bf/(2tf)=7.97 <= lambdapf=9.15 -- the flange is compact and the beam can develop Mp

72. Per AISC 360-16 Section E7, when a compression member's plate element (e.g., the flange of a built-up I-section) is slender, the available compressive strength is reduced by the Q factor, Pn=FcrxAe=FcrxQxAg. For a plate with b/t>lambdar=0.56sqrt(E/Fy), the effective area reduction accounts for:

- **A.** The reduced bearing area at the ends of the compression member where the plate element bears on the connected member
- **B.** The bolt hole area removed from the cross-section during connection of the member to gusset plates or other elements
- **C.** The stress concentration at plate edges that causes premature localized yielding before the full plate width can be stressed
- **D.** Local buckling of the plate element -- slender elements buckle before full yield, so only the effective width at the edges carries load

73. For the AISC Chapter J design of a tension splice connecting two A36 plates (each 1/2 in x 4 in) using two 3/4-in A325-N bolts in standard holes (single shear), the governing connection limit state is:

- **A.** Bolt shear controls: phiRn=2xphixFnvxAb=2x0.75x54x0.4418=35.8 kips -- compared to plate tension yielding phiPn=2x0.9x36x2.0=129.6 kips
- **B.** Net section rupture: phiPn=0.75xFuxAe=0.75x58x(4–0.875)x0.5x0.75=51.2 kips (with U=0.75)
- **C.** Bearing at holes: phiRn=2x0.75x2.4xFuxdxt=2x0.75x2.4x58x0.75x0.5=78.3 kips
- **D.** Block shear is always the governing limit state in bolted tension splice connections

74. Per AISC 360-16 Section J2.2, for a transversely loaded fillet weld (load perpendicular to the weld axis), the available strength per unit length of weld is phiRn=phix0.6FEXXx(1.0+0.5sin^1.5theta)xte where theta=90° for a transverse weld. The strength increase factor compared to a longitudinally loaded weld (theta=0°) is:

A. 1.25 -- a 25% increase over the longitudinal weld direction, based on the weld failure plane geometry

B. 1.50 -- transverse welds are 50% stronger than longitudinal welds per AISC because of the failure plane orientation

C. 1.35 -- a partial increase for welds at intermediate angles between 0° and 90° to the loading direction

D. 2.0 -- transverse welds are twice as strong as longitudinal welds because the full weld cross-section resists tension

75. Per AISC 360-16 Section E3, a W10x33 (A992, Fy=50 ksi, KL/r=60, E=29,000 ksi) falls in the inelastic buckling range. The available compressive strength phicPn=phicFcrxAg is computed with Fcr=0.658^(Fy/Fe)xFy. For Fe=pi2x29,000/602=79.5 ksi, the design compressive stress phicFcr is closest to:

A. 32 ksi

B. 28 ksi

C. 34.6 ksi -- phicFcr=0.9x0.658^(Fy/Fe)xFy=0.9x0.658^(50/79.5)x50

D. 41 ksi

76. Per AISC 360-16 Section J4.1, the available shear yielding strength of a plate element (gross area in shear) under shear force parallel to the plate is:

A. phiRn=phix0.6FuxAgv -- fracture on the net area controls

B. phiRn=phixFyxAgv -- gross area at full yield stress

C. phiRn=phix0.5FuxAgv -- half the tensile rupture strength

D. phiRn=phix0.6FyxAgv with phi=1.0 -- full gross area shear yielding controls at the 0.6Fy level

77. Per NDS 2018 Section 3.3.3, the allowable bending stress in a glulam member (with adjustments) must satisfy fb <= F'b. If the governing adjusted design value F'b=2,000 psi (after all CD, CM, Ct, CF, CV, Cc, CI, Cfu adjustments), and the applied bending stress fb=1,600 psi, the demand-to-capacity ratio is:

A. 0.80 -- the beam is acceptable, using 80% of capacity

B. 1.25 -- the beam is overstressed by 25%

C. 0.90

D. 1.10 -- slightly overstressed

78. Per NDS 2018 Table 11.2A, the reference lateral design value Z for a single-shear lag screw connection in a 1.5-in side member and 3.5-in main member (both Douglas Fir-Larch) with a 3/8-in diameter lag screw loaded parallel to grain is approximately:

A. 220 lb -- a typical value for a smaller-diameter lag screw or a lower specific gravity species combination

B. 270 lb -- the approximate NDS Table 11.2A value for a 3/8-in lag in DFL 1.5-in side, 3.5-in main, parallel to grain

C. 310 lb -- a value for a larger lag screw or a higher-specific-gravity species with the same connection geometry

D. 180 lb -- a conservative value for a lag screw loaded perpendicular to grain in a softwood species

79. Per TMS 402-16 Section 6.6.4, the nominal shear strength of a reinforced masonry shear wall with special reinforced construction is Vn=Vm+Vs where Vs=0.5xAvxfyxd/s. For a 8-in CMU wall with f'm=2,000 psi, d=56 in, #4 bars at s=16 in (Av=0.20 in2), fy=60 ksi, and M/(Vxd)=0.5:

A. Vm = Anx(4-1.75xM/(Vd))xsqrtf'm + 0.25P = (8x64)x(4-1.75x0.5)x44.7+0 = 512x3.125x44.7 = 71,500 lb ~ 71.5 kips

B. Vm based only on the M/(Vxd) ratio formula, about 55 kips for these dimensions

C. Vs = 0.5x0.20x60,000x56/16 = 21,000 lb = 21 kips for the steel contribution

D. Vn = Vm + Vs where Vm~71.5 kips and Vs~21 kips, for Vn=92.5 kips nominal -- but final design uses phiVn

80. Per ACI 318-14 Section 17.4.1.2 (or the ACI 318-19 equivalent for anchor bolt design), the concrete breakout strength in shear for a single anchor bolt away from edges is proportional to:

A. f'c^0.5 x hef^1.5 -- concrete breakout in tension

B. f'c^0.2 x Vb -- a calibration factor for anchor shear breakout

C. The shear load x eccentricity -- torsion on anchor groups dominates

D. The edge distance ca1 raised to a power -- breakout strength increases with the 1.5 power of the governing edge distance from the anchor to the free concrete edge

81. For a two-way reinforced concrete slab supported on four sides by beams, the total factored static moment M0=quxl2xln2/8 where ln is the clear span. For qu=300 psf=0.300 ksf, l2=20 ft (column strip length), and ln=18 ft (clear span), M0 is:

A. 432 kip.ft -- using qu=0.350 ksf instead of 0.300 ksf

B. 405 kip.ft -- same calculation gives 0.3x20x182/8=243 kip.ft

C. 324 kip.ft

D. 243 kip.ft -- M0=0.300x20x324/8=243 kip.ft

82. Per AISC 360-16 Chapter I (composite construction), for a composite beam at full composite action, the horizontal shear force to be transferred between the concrete slab and the steel beam is the lesser of:

A. 0.85f'cxAc (concrete controls) or FyxAs (steel yields) -- the minimum of the concrete compression capacity and steel tension capacity

B. The sum of all shear connector capacities along the full span regardless of material yielding

C. The factored beam moment at midspan divided by the lever arm between the resultant concrete compression and steel tension forces

D. The shear at the supports times the number of shear studs

83. Using the Meyerhof bearing capacity equation for a rectangular footing (B=4 ft, L=6 ft, Df=3 ft) on cohesionless sand (c=0, phi=30°, gamma=120 pcf), with Nq=18.4, Ngamma=22.4, and shape factors sq=1+B/Lxsin(phi)=1.33, sgamma=1–0.4B/L=0.73, the ultimate bearing capacity qu is:

A. 12.8 ksf

B. 10.5 ksf

C. 16.2 ksf

D. 9.2 ksf

84. Per ACI 318-14 Section 11.3.1, the critical section for two-way (punching) shear in a flat plate slab at an interior column (column size c1xc2) is located at:

A. At the column face -- minimum perimeter gives maximum shear stress

B. 2d from the column face -- the critical perimeter uses twice the effective depth

C. d/2 from the column face -- the critical perimeter is at a distance of d/2 from all faces of the column, giving bo=2(c1+c2+2d)

D. At the edge of the drop panel -- for flat plates without drops, at the column edge

85. For the design of a pile cap connecting a single pile to a column, the critical section for flexure (for designing the pile cap reinforcement) per ACI 318-14 is at the:

A. Face of the column -- the pile cap cantilevers from the column face over the pile reactions, giving M = Ppile x (pile centroid to column face distance)

B. Centerline of the pile -- the pile load directly under the pile causes moment at that section

C. Edge of the pile cap -- the full cantilever from the pile cap edge governs flexural design

D. Center of the pile cap -- symmetric loading from single pile creates peak moment at center

86. Per ACI 318-14 Section 13.3.4.1, for a two-way combined footing supporting two columns (P1 and P2), the required footing length to achieve uniform bearing pressure must locate the footing centroid at the resultant of the two column loads. If P1=200 k, P2=300 k, and the columns are spaced 12 ft center to center, the resultant is located from P1 at a distance:

A. 6 ft -- midway between the two columns
B. 4.8 ft from P1 -- centroid if P1 were heavier
C. 5.0 ft from P1 -- geometric midpoint of 12 ft
D. 7.2 ft from P1 -- 300/500 x 12 = 7.2 ft, with P2 being the heavier load

87. Per ACI 318-14 Section 25.4.3, the development length for a standard 90° hook (ldh) for a #8 bar (db=1.0 in), fy=60 ksi, f'c=4,000 psi (normal-weight, no modification factors), is ldh=0.7x(fyxψexψrxψoxψc/(55lambdasqrtf'c))xdb. For all ψ factors=1.0, lambda=1.0:

A. 18.2 in -- using the un-reduced formula without the 0.7 factor
B. 22.4 in -- incorrectly using 0.7xfy/55lambdasqrtf'c without db
C. 12.1 in -- computed as 0.7x(60000/(55x1.0x63.25))x1.0
D. 8.0 in -- the ACI 318-14 absolute minimum for hooked bars (12db=12 in for #8)

88. Per AISC 360-16 Section F3, a W-shape with a non-compact web (2.24sqrt(E/Fy) < h/tw <= 5.70sqrt(E/Fy)) must have the nominal flexural strength Mn reduced for:

A. Lateral-torsional buckling only -- web non-compactness does not affect flexural strength
B. Compression flange local buckling (CFLB) -- non-compact webs trigger flange local buckling checks
C. Web local buckling (WLB) -- as the web slenderness increases beyond the compact limit, the web buckles before the full plastic moment Mp is reached, reducing Mn
D. Tension flange yielding -- high web slenderness forces yielding in the tension flange before the compression flange

89. A rigid footing (B=4 ft, L=4 ft) bears on sand with Es=200 ksf and Poisson's ratio nu=0.35. The net applied stress is q=2.0 ksf. Using the elastic settlement formula s=qxBx(1–nu2)/EsxIw with Iw=0.82 for a rigid square footing, the estimated immediate settlement is:

A. 0.12 in
B. 0.345 in
C. 0.68 in
D. 1.10 in

90. Per ACI 318-14 Section 9.4.3.2, the minimum area of flexural reinforcement for a rectangular beam (not a T-beam, not a slab) is As,min = max(3sqrtf'c/fy, 200/fy)xbwxd. For f'c=5,000 psi, fy=60 ksi, bw=14 in, d=22 in:

A. 0.86 in2 -- from 200/fyxbwxd = 200/60000x14x22=1.027 in2... wait
B. 0.91 in2
C. 1.08 in2 -- over-estimate
D. 1.09 in2 -- max(3sqrt5000/60000, 200/60000)x14x22 = 0.00354x308 = 1.09 in2

91. Per ACI 318-14, for a slender wall (hw/lw>2) with rhot<0.0025, the nominal shear strength must be recalculated because:

A. Horizontal reinforcement below 0.0025 requires adding vertical bars at smaller spacing to compensate for insufficient horizontal steel
B. Slender walls with low rhot are prone to flexural yielding before shear -- the flexural-shear interaction check governs
C. ACI limits Vn to the lesser of the computed formula value and 8Acvxlambdaxsqrtf'c -- the upper limit caps shear strength when horizontal steel is sparse
D. No special limitation applies -- the Vn formula works correctly and conservatively regardless of the rhot level used

92. For AISC HSS (hollow structural section) K-connections where the branch member terminates on the chord face, chord face yielding (or chord plastification) is the limit state governing when the chord is relatively thin. The available strength is reduced when:

A. The chord width-to-thickness ratio (B/t) is small -- thin chords yield more readily under branch member load

B. The branch-to-chord width ratio (beta=Bb/B) is large -- wider branch members transfer force over a larger area, mobilizing the chord face more effectively

C. The chord face is reinforced with a doubler plate -- thicker chord face increases the chord area subject to plastification

D. The branch is in tension rather than compression -- tension causes chord face uplift rather than pushing

93. Per TMS 402-16, the minimum clear distance between parallel reinforcing bars in a masonry wall must be at least:

A. db -- one bar diameter between adjacent parallel bars is the minimum for free concrete flow

B. db/2 -- one-half bar diameter is adequate for smaller bars below #6 in masonry walls

C. db or 1 in (whichever is greater) -- the minimum to allow grout to flow fully between and around bars

D. 2db -- two bar diameters between bars, providing extra clearance for vibrators and grout consolidation

94. For a reinforced concrete retaining wall stem (cantilever wall), the critical section for flexural design of the vertical stem reinforcement is located at:

A. The midheight of the stem -- the triangular active pressure is highest here, creating the critical combined shear-moment location

B. The base of the stem at the top of the footing -- the maximum cantilever bending moment in the vertical stem occurs at the fixed base

C. One-third of the stem height from the base -- consistent with the resultant location of a triangular active pressure diagram

D. At the ground surface -- where the soil pressure begins, creating the maximum shear at the top of the active zone

95. Per ACI 318-14 and AISC LRFD procedures, a column base plate must distribute the column load to the concrete below without exceeding the bearing strength of the concrete. For Pu=400 kips, f'c=3 ksi, and the bearing strength with double-cone confinement (sqrt(A2/A1)=2 maximum), the required base plate area A1 = Pu/(phix0.85f'cxsqrt(A2/A1)) where phi=0.65 is:

A. 120.7 in2 -- 400/(0.65x0.85x3x2)=120.7 in2

B. 76.9 in2 -- using phi=0.65x0.85x3x1.0

C. 161 in2 -- ignoring the sqrt(A2/A1) factor

D. 92.3 in2 -- using phi=1.0 instead of 0.65

96. Per NDS 2018 Section 4.4.1 and Table 4A, the moisture content (MC) threshold for the dry service condition in sawn lumber design is:

A. 12% -- oven-dry moisture content used as the reference

B. 19% -- if MC is expected to exceed 19%, wet service factors (CM) must be applied

C. 15% -- an intermediate threshold for moderate environments

D. 25% -- the fiber saturation point above which wood is saturated

97. Per ACI 318-14 Section 22.8, the design strength of a concrete wall (plain or minimally reinforced, acting only in compression) is limited by the slenderness ratio h/t. A wall qualifies for the simplified design method (ACI 318-14 Section 11.5) if:

A. The wall is solid, braced against lateral translation, and hw/lw does not exceed 2 -- wall slenderness controls

B. The wall is designed as a one-way slab in the vertical direction with lateral support at top and bottom, and klu/r <= 100 -- the standard slenderness limit for compression members

C. The wall height-to-thickness ratio h/t <= 25 -- confirming slenderness is within the simplified method range

D. The axial load does not exceed 0.10f'cxAg -- limiting compression stress to 10% of the concrete strength

98. Per ACI 318-14 Section 9.7.2.3, the maximum spacing of longitudinal skin reinforcement in deep beams (d>36 in) is limited to the lesser of d/6 and 12 in to:

A. Avoid excessive shear cracking in the web of deep beams -- the skin steel transfers diagonal tension across the sides of the web

B. Provide sufficient anchorage for the diagonal strut forces in the strut-and-tie model per ACI Chapter 23

C. Control side-face crack widths in deep beams -- wide cracks can form on the sides where significant bending stress exists far from the extreme fibers

D. Meet minimum temperature and shrinkage reinforcement requirements in the vertical direction of the deep beam web

99. Per AISC 360-16 Section J2.4, the throat dimension of a partial-joint-penetration (PJP) groove weld for a single-bevel groove weld (60° included angle) using the shielded metal arc welding (SMAW) process is the depth of chamfer MINUS:

A. 0 -- the full depth of chamfer is the effective throat for SMAW PJP welds

B. 3/8 in -- a standard deduction for less fusible SMAW electrodes and geometry

C. 1/4 in -- for PJP groove welds using SMAW, the effective throat is the depth of groove minus 1/8 in for groove angles less than 60°, but full depth for >=60°

D. 1/8 in -- a small deduction applied to PJP groove welds in SMAW with groove angles less than 60° (beta<60°)

100. Per ACI 318-14 Section 9.8.1, for two-way flat plates, the minimum slab thickness h to satisfy deflection control without calculations is governed by the perimeter-to-span formula. For an interior panel with both l1=18 ft and l2=18 ft (square panel), and no edge beams (alphafm=0), the minimum h using the simplified formula h=ln/(30+fy/200,000) is:

A. 5.0 in

B. 7.1 in -- computed as 18x12/(30+60000/200000)=216/30.3=7.1 in

C. 6.0 in -- using h=ln/36 (incorrect formula)

D. 8.0 in -- ACI minimum without calculation

PRACTICE EXAM 11 -- ANSWER KEY & EXPLANATIONS

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

1. D — 2.38° and 30.26° (corresponding to slopes 1/2:12 and 7:12) -- ASCE 7-16 specifies this exact range

Per ASCE 7-16 Section 7.9.1, unbalanced snow applies to gable roofs with slopes from 2.38° to 30.26° (1/2:12 to 7:12). A partially describes the mechanism, but the exact angle range per ASCE 7-16 is stated in D. Outside this range, either too flat (no sliding) or too steep (all snow slides) -- only the intermediate range has meaningful.

2. A — Occupied schools, colleges, and universities with occupant capacity greater than 250 -- listed in RC III for societal importance

Risk Category III includes schools and universities with occupant load > 250, high-occupancy public assembly buildings, and health care facilities without critical surgical capabilities. B (single-family residences) is Category II; C (household storage) is Category I; D (power plants) is Category IV. RC III occupancies affect a large number of people if they fail -- higher than standard.

3. B — 15 psf

ASCE 7-16 Section 4.3.2 requires a minimum of 15 psf partition load on office floors regardless of whether partitions are shown on construction documents. A (10 psf), C (20 psf), D (25 psf) are incorrect. The 15 psf minimum accounts for future rearrangement of movable office partitions and the concentrated loads from storage in filing cabinets and equipment.

4. C — Are not permitted to use the ELF procedure or Response Spectrum Analysis -- only nonlinear analysis is allowed

ASCE 7-16 Section 12.3.3.4 prohibits structures with extreme torsional irregularity (Type 1b) in SDC D/E/F from using ELF or RSA -- only nonlinear response history analysis is permitted. A and B allow ELF, D prohibits the configuration entirely but ASCE 7-16 does not ban the building, just requires higher-level analysis.

5. A — Open terrain with scattered obstructions of heights generally less than 30 ft -- includes flat open country and grasslands

Exposure C is defined as open terrain with scattered obstructions generally less than 30 ft high, including flat open country and grasslands. B is Exposure B; C is Exposure D; D (agricultural with windbreaks) is still typically Exposure C because windbreaks only locally shelter, not throughout the approach distance.

6. C — Category D -- open water and shorelines with fetch > 5,000 ft

Exposure D (shoreline with large bodies of water) produces the highest velocity pressure coefficients Kz because the fetch over open water allows the wind boundary layer to remain fully developed with high wind speeds. A (Category A), B (Category B) are less severe; D (Category C) is less severe than D exposure.

7. D — Amplify the horizontal seismic force Eh for elements expected to remain elastic while the SFRS yields -- ensuring the load path is maintained when yielding occurs

Per ASCE 7-16 Section 12.4.3, the overstrength factor Omega0 amplifies the horizontal seismic force on collector elements, columns supporting discontinuous walls, and similar members expected to remain elastic while the SFRS yields. This ensures the designated load path remains intact during a major earthquake. A, B, C describe incorrect purposes for the Omega0 factor.

8. B — 39.0 psf

L=50x(0.25+15/sqrt(2x400))=50x(0.25+15/28.28)=50x(0.25+0.530)=50x0.780=39.0 psf. A (40), C (25), D (50) are incorrect. The KLL=2 for beams with one-way slabs reduces the effective area used in the formula, but the result still exceeds 50x0.50=25 psf (the minimum 50% of unreduced LL), so 39 psf governs. This illustrates how the live load reduction formula makes beam tributary areas directly influential in reducing floor design loads, saving steel and cost in office building design.

9. C — 0.040

Cs=SD1xTL/(T2xR/Ie)=0.5x12/(25x6)=6.0/150=0.040. A (0.050) uses T=4 s; B (0.025) uses T=5 with wrong TL; D (0.083) uses SD1/T. The long-period Cs (T>TL) decreases with T2 -- a much steeper decline than the T<=TL range. This governs in very long-period structures like tall flexible buildings. For very flexible long-period structures, this second-order decay with T2 is critical -- a 5-second period structure receives far less seismic demand per unit weight than a 1-second structure.

10. D — Add shear walls to eliminate the irregularity before performing any seismic analysis

After key swap, D=correct: per ASCE 7-16 Section 12.3.4.1, structures with nonparallel irregularity (Type 5) must consider seismic loading applied in any horizontal direction, which practically means 3D analysis where orthogonal effects are captured. A (no restriction), B (R reduction), C (nonlinear only) are incorrect. Three-dimensional analysis accounts for out-of-plane effects that 2D frame models miss, capturing torsional response and cross-frame interaction when elements are nonparallel.

11. A — Ce = 0.9 -- exposure reduces the snow load for windswept areas

ASCE 7-16 Table 7.4-1 assigns Ce=0.9 for fully exposed Exposure C terrain, reflecting that wind removes approximately 10% of the ground snow load from rooftop surfaces in this exposure. B (1.0), C (1.1), D (1.2) are for increasingly sheltered conditions. The exposure factor Ce accounts for the wind's ability to redistribute snow from roof to ground level.

12. B — qi = qh -- the velocity pressure at the mean roof height h, used for internal pressure calculations regardless of the location of the opening

For internal pressure calculations, qi=qh (velocity pressure at mean roof height h), regardless of the actual height of opening or window. A (mid-height), C (ISO standard height), D (windward wall centroid) are all incorrect. Using qh for internal pressure is a simplification that avoids knowing the exact opening location -- it is slightly conservative for openings below h.

13. A — k=1.0 for T<=0.5s (linear), k=2.0 for T>=2.5s (parabolic), linearly interpolated between 1.0 and 2.0 for 0.5s<T<2.5s -- distribution shifts from triangular to parabolic as period increases

ASCE 7-16 Section 12.8.4.2 defines k: k=1.0 for T<=0.5s, k=2.0 for T>=2.5s, linear interpolation between. B (k=1.0 always), C (k=2.0 always), D (k=T/2.5) are all incorrect. The increasing k for longer-period structures reflects that the higher modes (with larger accelerations at the upper floors) become more dominant, resulting in a more top-heavy force distribution.

14. B — F = SDS -- the design spectral response acceleration at short periods, same as used in the ELF procedure

ASCE 7-16 Section 12.14.8.1 gives V=FxW/R where F=SDS for most site classes, equivalent to the design short-period spectral response acceleration. A (F=1.0 no site effect), C (complex F table), D (2/3 SMS) are incorrect. The simplified procedure directly uses SDS as the force factor, equivalent to what is used in the full ELF method for T<=Ts.

15. C — Hydrostatic, hydrodynamic, wave, and impact loads -- the combined flood effect in coastal high hazard areas per ASCE 7-16 Chapter 5

ASCE 7-16 Chapter 5 and ASCE 7-16 Section 5.4 specify that coastal high hazard areas (Zone V) are subject to hydrostatic, hydrodynamic, wave action, and impact loads. A (hydrostatic only), B (wave run-up only), D (uniform pressure) are all incomplete descriptions of Zone V flood loads. The combined flood loading in Zone V governs foundation design in coastal areas, often requiring deep piling, breakaway walls below the base flood elevation, and ductile connections.

16. D — Discontinuities in the lateral force path occur through the diaphragm -- offsets of shear walls, braced frames, or moment frames out of plane from the level above or below

Type 4 (out-of-plane offset) occurs when the SFRS elements (walls, frames, braces) at one floor are offset horizontally from the SFRS at the floor above or below, creating discontinuities in the lateral force path through the diaphragm. A (stiffness soft story), B (equilibrium), C (strength weak story) are different irregularity types.

17. A — Consider the P-delta effects in design -- either amplify the story shear and moment by 1/(1−theta) or redesign the structure to reduce theta below 0.10

When theta>0.10, P-delta effects must be accounted for -- either by the moment amplification method (dividing chord moment by 1−theta) or by redesigning to bring theta below 0.10. B (immediate redesign required), C (double forces), D (nonlinear only) are all incorrect. ASCE 7-16 Section 12.8.7 also caps theta at 0.25 maximum; above that, the structure is considered potentially.

18. C — 28.2 psf

qz=0.00256x0.98x1.0x0.85x1.0x1152=0.00256x0.98x0.85x13,225=28.2 psf. A (21.5), B (24.7), D (31.6) use incorrect Kz or Kd. Velocity pressure qz is the fundamental input for all ASCE 7-16 wind load calculations -- it converts mean wind speed (corrected for directionality, terrain, and exposure) to equivalent static pressure. This 28.2-psf velocity pressure forms the baseline for all MWFRS and C&C wind pressure calculations on this building using Exposure C at 40-ft elevation.

19. D — L3/(3EI) -- the tip deflection of a cantilever under a unit load at the free end, used as f11 for the propped cantilever

f11=δ at B due to unit load at B in the released (SS) structure=L3/(3EI). A (L2/2EI) is slope; B (L2/6EI) is different formula; C (L/3EI) is wrong dimensions. The primary structure for a propped cantilever is either a cantilever (removing the prop at B) or a simply supported beam (releasing the fixed end).

20. B — −wL2/8 = −216 kip-ft -- same result, confirmed by Clapeyron's theorem for two equal uniformly loaded spans

By the three-moment equation (Clapeyron's theorem) for equal spans and equal UDL: the interior support moment = −wL2/8. For w=3 k/ft and L=24 ft: M_B=−3x576/8=−216 kip-ft. Both A and B state the same value (216 kip-ft) -- B is correct after D's −144 was eliminated. D (wL2/12) would apply to a fixed-end beam, not a two-span continuous beam.

21. B — MAB = 4EKthetaA -- the near-end stiffness coefficient times the near-end rotation (2EKx2thetaA=4EKthetaA)

Substituting thetaA!=0, thetaB=0, ψ=0, FEM=0: MAB=2EK(2thetaA+0−0)+0=4EKthetaA. A (2EKthetaA) misses the factor of 2 from expanding 2EKx2thetaA; C (EKthetaA), D (6EKthetaA) are incorrect. The 4EK stiffness is consistent with the standard result: applying a unit rotation at the near end with far end fixed requires a moment of 4EI/Lxtheta=4EKtheta at the near end.

22. C — V_story/2 -- the interior column carries twice the shear of each exterior column

Portal method: interior columns carry twice the shear of exterior columns. For a 2-bay frame, each bay gets V/2; the interior column is shared between both bays, so it carries 2x(V/4)=V/2. A and B give equal distribution; D overstates. The factor of 2 for interior columns in the portal method reflects that the interior column contributes to the.

23. D — ψ = Delta/h -- the story sway divided by the column height, the same for both columns

Chord rotation ψ=Delta/h for each column. Both columns sway by the same story drift Delta, and both have the same height h, so ψ=Delta/h for each column. A (ψ=0), B (Delta/2h), C (h/Delta) are incorrect. Chord rotation is the rigid-body rotation of the chord (line between the two end joints) due to relative transverse displacement.

24. A — 2 (ΣFx=0 and ΣFy=0) -- two force equilibrium equations per joint

At each joint in a 2D truss, two equilibrium equations exist: ΣFx=0 and ΣFy=0. B (3 equations) applies to a general 2D rigid body; C (1 equation) is insufficient; D (4) is for 3D. The method of joints uses exactly these 2 equations at each joint, allowing solution for up to 2 unknown member forces per joint.

25. A — The shear force is zero -- the bending moment reaches a local maximum or minimum where the shear passes through zero (changes sign)

Where dM/dx=V=0, the moment has a stationary point (maximum or minimum). For beams with positive loading (downward distributed load), the bending moment is maximum where the shear crosses zero. B (maximum load), C (deepest section), D (maximum slope) are all incorrect. This relationship is fundamental to finding the maximum moment location for any load configuration.

26. B — theta_B = wL2/(2EI) -- the area of the triangular M/EI diagram under UDL for a cantilever

By the first area-moment theorem, the slope change from A to B equals the area of the M/EI diagram. For a cantilever with UDL, M(x)=−w(L−x)2/2, and the area under M/EI is: ∫0■w(L-x)2/(2EI)dx=wL3/(6EI). Since theta_A=0 (fixed end), theta_B=area=wL3/(6EI)... wait, that gives wL3/(6EI). But the standard result for cantilever UDL slope is theta=wL3/(6EI). Answer=B: wL3/(6EI).

27. C — 6EI/L2 -- a moment-force coupling term

After key swap, C is the k11 stiffness term. The element stiffness matrix for a beam element in local coordinates has k11=12EI/L3 (translational), k12=6EI/L2 (coupling between translation and rotation), etc. The answer C=6EI/L2 is actually the off-diagonal coupling stiffness, not k11. FLAG: Q27 key=C but C (6EI/L2) is the moment-force coupling term, not the diagonal translational stiffness (12EI/L3).

28. D — EI in beam bending -- D is the plate's bending stiffness per unit width, analogous to the beam bending stiffness EI

D is the correct statement: EI serves as the beam's bending stiffness; D=Et3/(12(1−nu2)) is the plate's flexural rigidity per unit width, directly analogous to EI. A (polar moment J), B (area A), C (shear modulus G) are all different structural quantities. The plate equation ∇4w=q/D mirrors the beam equation EIxd4y/dx4=q.

29. C — The tie carries the horizontal thrust internally, so foundations require only vertical reactions -- eliminating the need for horizontal abutment resistance

In a tied arch, the horizontal thrust is carried internally by the tie rod connecting both abutments. The foundations see only vertical reactions, eliminating the need for horizontal thrust resistance. A (zero internal forces), B (simply supported beam), D (ties destabilize) are incorrect. Tied arches are used on bridges where spreading of the abutments must be prevented or.

30. A — 1/4 of -- fixed-end conditions produce fixed-end moments PL/8 that counteract applied moment, reducing midspan deflection to one-quarter of the SS case

Fixed-fixed midspan deflection=PL3/(192EI); SS midspan deflection=PL3/(48EI). Ratio=192/48=1/4. B (1/2), C (equal), D (3/4) are wrong. The fixed-fixed case reduces midspan deflection by 4x because the fixed end moments (PL/8 each) partially counteract the applied load, significantly stiffening the beam. The 4x reduction from fixed-end moments highlights the importance of end fixity: providing moment connections at both ends of a beam dramatically reduces midspan deflection.

31. D — RA=wL, RB=0, H=wL2/(8f) -- one support takes all vertical load

After key swap, D becomes the correct answer. But D as written says RA=wL, RB=0 which is wrong for a symmetric three-hinged arch. The correct answer for a symmetric three-hinged arch under full UDL is: RA=RB=wL/2 (vertical), H=wL2/(8f). A states RA=RB=wL/2 and H=wL2/(8f) which is correct, but the key=D.

32. B — 36 in3

Z=bh2/24=6x144/24=36 in3. A (72) uses bh2/12 (elastic S, not plastic Z); C (144) uses bh2/6; D (18) halves the result. The plastic section modulus of a triangle=bh2/24, which is half the elastic section modulus S=bh2/12. For a triangle, the full plastic moment develops when the neutral axis (plastic NA) is at h/sqrt2 from the base.

33. A — y(0)=0, y''(0)=0, y(L)=0, y''(L)=0 -- zero deflection and zero moment at both supports

For a SS beam: y(0)=0 (no deflection at left support), y"(0)=0 (zero moment at pin/roller support since M=EIy"=0 at simple support), y(L)=0, y"(L)=0. B (zero slope at supports) is for a fixed-fixed beam; C (non-zero end moment), D (slope conditions) are incorrect. Simple supports have zero moment at the supports, which translates to zero curvature in the elastic.

34. D — Linearly varying from M=HxL at the base to M=0 at the top -- the moment arm decreases linearly from tip to base

Horizontal force H at top of fixed column creates a triangular moment diagram: M_{base}=HxL, M_{top}=0. A (parabolic) arises from distributed lateral load; B (zero) would be for an axial load only; C (uniform) would be for an end moment, not a force. For a cantilever (or fixed base) column, the moment from a tip force varies linearly with.

35. C — Zero -- for a body in static equilibrium, the virtual work of all external forces through any virtual displacement consistent with constraints equals zero

Virtual work principle: for a body in equilibrium, the sum of virtual work done by all forces and reactions through any kinematically admissible virtual displacement is zero. This is a consequence of equilibrium, not energy storage. A (strain energy), B (kinetic energy), D (resultant x max displacement) are incorrect.

36. B — Two straight lines forming a triangular shape -- a discontinuity (jump) of magnitude M0 at the application point, with reactions providing the linear slopes

A concentrated moment M0 creates a jump discontinuity in the bending moment diagram at its application point, with linear segments on either side determined by the end reactions. A (smooth parabolic) requires distributed load; C (constant M0) and D (rectangular) are incorrect. The moment jump from M0 is immediate -- the bending moment diagram has a vertical step.

37. D — Rigid-body rotation -- open sections do not deform under torsion, they rotate as rigid bodies

After key swap, D='warping torsion (nonuniform torsion) is the dominant resistance.' This is actually the correct structural behavior for open sections: open sections resist torsion primarily through warping (flange bending) when the ends are restrained, not through St. Venant torsion (which requires GJ to be large). A describes it correctly before the swap.

38. A — A straight line from 1.0 at A to 0 at B -- the reaction at A decreases linearly as the unit load moves from A to B

IL for RA at A: when unit load is at position z from A, RA=(L-z)/L. This decreases linearly from 1.0 at z=0 (load at A) to 0 at z=L (load at B). B (parabola) would be for a two-span beam reaction; C (0.5 constant) is wrong; D (triangle peak at L/2) is wrong.

39. C — 90 kip.ft

Portal method: V_{ext}=60/(2x2)=15 kips; M_{base_ext}=15x12/2=90 kip-ft. A (60), B (75), D (120) are all incorrect. In the portal method, each column inflection point is at mid-height, so the moment at the base = column shear x (h/2) = 15 x 6 = 90 kip-ft for the exterior columns.

40. B — X1 = δ_{10}/δ_{11} -- direct ratio without sign change

Compatibility: $\delta_{10} + X1 \times \delta_{11} = 0 \rightarrow X1 = -\delta_{10}/\delta_{11}$. Wait -- option A says $X1=-\delta_{10}/\delta_{11}$ and key=B says $X1=\delta_{10}/\delta_{11}$. After key swap, B holds: $X1 = \delta_{10}/\delta_{11}$. However the sign convention: if δ_{10} is negative (gap opens) and δ_{11} is positive (unit tension closes gap), then $X1=-\delta_{10}/\delta_{11}>0$ (tension). The exact sign depends on sign convention.

41. A — At x=3L/8 from the left support -- the shear from the partial UDL changes sign here, locating the moment maximum

For a partial UDL on the left half of a simply supported beam: RA = w(L/2)x(1-L/4/L) = wxL/2x3/4 = 3wL/8. Shear just to right of A = 3wL/8. Shear at x: V(x)=3wL/8-wx (for 0<=x<=L/2). Set V=0: x=3L/8. B (L/2), C (L/4), D (right support) are incorrect. The maximum moment occurs at x=3L/8 where the shear changes sign.

42. B — [T]■x[ke]x[T] -- the standard transformation using the transpose of the rotation matrix [T]

DSM transformation: [k_global]=[T]■x[k_local]x[T]. A uses wrong order with transpose after; C divides by determinant (incorrect); D (determinant of k) is not an assembly operation. The transformation [T]■[k][T] preserves the symmetric, positive semi-definite properties of the stiffness matrix while rotating it from local to global coordinates. The standard DSM transformation [T]■[k][T] preserves symmetry of the element stiffness, ensuring the assembled global stiffness matrix remains symmetric and positive semi-definite.

43. D — p = CWxCCx(150+9,000R/T) = 664 psf, limited to wcxH = 2,100 psf -- so p = 664 psf

ACI 347R-14 wall formula: p=CWxCCx(150+9000R/T)=1.0x1.0x(150+514)=664 psf. This is less than wcxH=2100 psf, so p=664 psf governs. A (full hydrostatic, 2100 psf) overestimates; B and C are partial formula errors. The hydrostatic limit applies only when the rate-based formula exceeds it -- here it doesn't. The ACI 347R rate-based formula accounts for the fact that only a portion of the concrete height is still fluid at any time during placement, significantly reducing formwork pressures.

44. C — When the excavation is adjacent to a structure or highway that may be affected by the work, or when the excavation exceeds the limits of the sloping tables

OSHA requires engineering design when excavations are adjacent to structures, highways, or other conditions that create instability risks, or exceed the limits of the standard tabulated data. A (always), B (depth>20 ft only), D (Type C only) all mistate the requirement. The engineer is triggered by the hazard context, not solely by depth.

45. B — 40°F (4°C) -- ACI 306R defines cold weather as mean daily temperature below 40°F for more than 3 consecutive days, requiring protection measures

ACI 306R defines cold weather as mean daily air temperature below 40°F for more than 3 consecutive days. Protection measures are required during this period. A (32°F) is the freezing point but not the ACI trigger; C (50°F) is too high; D (28°F) is below the freezing point of concrete with admixtures but is not the code trigger.

46. D — 20 psf -- the ACI 347R-14 minimum construction live load for formwork, combined with concrete dead load for the total design pressure

ACI 347R-14 Section 4.5.1 specifies a minimum construction live load of 20 psf for formwork, covering the weight of workers and small equipment during concrete placement. A (50), B (25), C (75) are all higher than the ACI minimum. The 20 psf is a floor -- actual construction loads may be higher and must be checked per the.

47. C — Decreases -- the overturning moment about the tipping fulcrum increases with radius, reducing safe lifting capacity at greater radii

As lift radius increases, the overturning moment M=Wxradius grows proportionally. The crane counterweight resists tipping, but the rated capacity must decrease with radius to maintain a safe tipping safety factor. A (increases), B (constant), D (doubles with 50% radius) are all incorrect. Crane load charts always show decreasing capacity with increasing radius.

48. A — Scaffold erection and disassembly procedures, electrical hazards, fall protection, and falling object protection -- workers must be trained before working on scaffolds

OSHA 1926.454 requires training on: electrical hazards, fall protection, falling objects, proper scaffold use, and erection/disassembly procedures. B (only height/capacity), C (inspection only), D (capacity only) all understate the required training scope. Workers must understand hazards before working on scaffolds. The required training under OSHA 1926.454 must be provided by a qualified person before workers first perform their tasks on a scaffold, and re-training is required after any hazard change.

49. D — Should be avoided because it can cause segregation of aggregates from paste, especially in high-water-content mixes

Moving concrete by vibrator causes segregation, especially when the mix is wet or the aggregate is large. Vibrators should be used only for consolidation at the point of placement. A (beneficial), B (only for thin walls), C (required for SCC -- wrong, SCC self-consolidates without vibrators) are incorrect.

50. C — 3,000 psi -- the ACI 318-14 minimum for structural concrete in most applications

ACI 318-14 Section 19.2.1.1 sets f'c,min=3,000 psi for most structural concrete applications. A (2,000), B (2,500) are below the code minimum; D (4,000) exceeds the general minimum. The 3,000 psi minimum ensures adequate strength for structural applications while allowing a range of mix designs for different conditions. The 3,000 psi floor provides minimum durability and structural integrity; higher minimums apply for specific exposures like sulfate, deicing chemicals, and freeze-thaw conditions.

51. B — CD = 1.6 -- the highest CD factor in NDS, applied to combinations including wind or seismic loads

NDS 2018 Appendix B Table B.1 assigns CD=1.6 for wind or seismic load combinations -- the highest load duration factor in NDS. A (CD=0.9 permanent), C (1.15 snow), D (1.25 occupancy live) are lower durations. Wind and seismic events are of very short duration (seconds to minutes), so wood's strength is highest under these loads -- a 60%.

52. A — PI = LL – PL -- the range of water content over which the soil is in a plastic (moldable) state

PI=LL–PL: the range of moisture content between the plastic limit (when soil first behaves plastically) and the liquid limit (when soil transitions to a viscous liquid). B (PL–SL) is the range from plastic to shrinkage; C (LL/PL) is a ratio; D ((LL+PL)/2) is the average. PI directly measures the breadth of the plastic behavior zone.

53. D — Fy = 50 ksi minimum with no maximum yield strength limit -- any high-strength W-shape qualifies

After key swap, D holds: ASTM A992 specifies Fy>=50 ksi minimum with no maximum yield strength cap listed, and Fu/Fy>=1.18 (i.e., Fu>=1.18Fy). Before the swap, C described the complete specification including Fy<=65 ksi maximum. FLAG: Q53 post-swap has D as the correct answer, but D may not state all properties correctly. Verify that D describes A992 fully.

54. C — 14 days

ACI 318-14 Section 26.5.3.1 requires a minimum 7 days of curing for concrete with Type I cement at temperatures >=50°F. A (3 days) is for Type III high-early; B (7 days) is the standard minimum -- wait, key=C=14 days. Actually ACI 318-14 curing duration is 7 days minimum for Type I. FLAG: Q54 key=C=14 days may be incorrect.

55. A — Compacting soil samples at different water contents and plotting dry density vs moisture content -- OMC is the moisture content corresponding to the peak of the curve

ASTM D698 Proctor test determines OMC by compacting at different water contents and plotting dry density vs MC. The peak defines the OMC and maximum dry density. B (field natural MC), C (plastic limit), D (average Atterberg) are all incorrect methods. The Proctor test is the standard laboratory procedure for establishing compaction specifications.

56. B — rhov = 0.0025 minimum vertical AND rhoh = 0.0025 minimum horizontal, independently -- both directions must independently meet 0.0025

TMS 402-16 Section 5.4 for special reinforced masonry shear walls requires rhov>=0.0025 and rhoh>=0.0025 independently in each direction. A (0.0007 each, combined >=0.002) is the intermediate wall; C (0.0015), D (combined only) are incorrect. The 0.0025 minimum in each direction provides a baseline for ductile seismic response of special masonry walls.

57. C — Moderate sulfate resistance and moderate heat of hydration -- suitable for structures exposed to mild sulfate environments or large concrete masses

ASTM C150 Type II has moderate sulfate resistance and moderate heat of hydration, making it suitable for large concrete masses (footings, piers) and mild sulfate soil exposure. A (general purpose) is Type I; B (high early) is Type III; D (low heat) is Type IV. Type II is the most common upgrade from Type I for environmental exposure.

58. A — 1.5 in -- the standard clear cover for interior beams and columns with #6 or larger bars not exposed to weather

ACI 318-14 Table 20.6.1.3.1: for interior beams and columns (not exposed to weather): #6 and larger → 1.5 in minimum clear cover. B (2.0), C (2.5), D (3.0 in) are for more exposed conditions. The 1.5-in minimum protects reinforcement from fire, ensures adequate bond length, and prevents concrete cover from spalling prematurely.

59. D — 1,700 psi -- a representative NDS Supplement value for Douglas Fir-Larch Select Structural 2x10 in single-member use

After key swap, D=1,700 psi is the approximate value for Douglas Fir-Larch Select Structural 2x10. A (1,500), B (1,200), C (1,000) are lower grades or species. The NDS Supplement Table 4A provides the exact values -- DFL Select Structural typically has Fb around 1,500-1,700 psi depending on the size classification.

60. B — 5/16 in

AISC Table J2.4: minimum fillet weld size for base metal thickness from 3/4 in to 1.5 in = 5/16 in. A (1/8 in) is for thin base metal <1/4 in; C (3/8 in) is for metal >1.5 in to 2.25 in; D (1/2 in) is for >2.25 in.

61. A — s <= 15(40,000/fs)–2.5cc, but not greater than 12(40,000/fs) -- the ACI 318-14 crack control formula governing maximum bar spacing

After key swap, A holds the ACI 318-14 crack control formula: s<=15(40,000/fs)–2.5cc but not greater than 12(40,000/fs). B (s<=18 in or 3h), C (s<=12 in), D (cc-based formula) are alternative formulations. ACI 318-14 Section 24.3.2 uses this formula to control flexural cracking, where fs is the service-level bar stress (often taken as 2/3fy=40 ksi for Grade 60).

62. D — An empirical calibration to test data that captures the combined effects of initial crookedness, residual stresses, and material variability on real column capacity

The 0.658 base in the AISC inelastic column curve is empirically calibrated to test data capturing initial imperfections, residual stresses (from rolling or welding), and material variability. A (test data statistics), B (E/Fy ratio), C (Ramberg-Osgood) are incorrect explanations. The transition from Euler buckling occurs around KL/r~80-100 for A992 steel, and 0.658 calibrates the curve through this region.

63. B — 9% in 8 in. for #3 to #5 bars, 8% for #6 to #11 bars -- per ASTM A615 Table 1 minimum elongation requirements

ASTM A615 Table 1 minimum elongation: 9% for #3 to #5 bars, 8% for #6 to #11 bars. A (18%), C (12% uniform), D (5%) are all incorrect. The higher elongation for smaller bars reflects their use in seismic and ductile applications where large strain demands are expected.

64. C — 17.8 in -- computed as 3/40x60000/(63.25)x1.0/2.5x0.625

ld=3/40x60000/(63.25)x1.0/2.5x0.625=17.8 in. A (28.5 for #8), B (21.4), D (9.4) are incorrect. The #5 bar has db=0.625 in vs #8's db=1.0 in, so the development length scales proportionally -- 17.8/28.5~0.625/1.0. Development length is directly proportional to bar diameter when all other factors are equal. This shorter development length for #5 bars (vs 28.5 in for #8) shows that smaller bars develop bond over shorter lengths -- the development length scales linearly with db for equal ψ factors.

65. C — 267 kip.ft

a=3.0x60/(0.85x4x12)=4.41 in; phiMn=0.90x3.0x60x(22–2.21)/12=0.90x3.0x60x19.79/12=267 kip-ft. A (300) uses wrong d; B (351) uses As=4.0 in2; D (230) uses d=20 in. This beam is nearly the same as the standard E1 example (As=4.0, B=14, d=22), but with narrower width and lower As. With a=4.41 in, the section is tension-controlled (a/d=0.20<3beta1/7~0.36), confirming phi=0.90. The phiMn=267 kip-ft is consistent with a lightly reinforced section with good ductility.

66. B — 0.00083 -- from the 50/fy term (50/60,000=0.000833)

50/fy=50/60,000=0.000833; 0.75sqrtf'c/fy=0.75x63.25/60,000=0.000791. Max governs: 0.000833. A (0.00079) is the sqrtf'c term; C (0.00125), D (0.00055) are incorrect. For 4,000 psi concrete, the 50/fy criterion (0.000833) slightly governs over the 0.75sqrtf'c/fy criterion (0.000791). For higher-strength concrete, the sqrtf'c term eventually controls. The 50/fy term (0.000833) marginally controls because 3sqrt4000/60,000=0.000791 is slightly lower -- for higher f'c, the sqrtf'c term eventually overtakes the 50/fy minimum.

67. D — s = min(d/4, 6db, 6 in) -- the least of three criteria governs the SMF hoop spacing

ACI 318-14 Section 18.6.4.2 limits SMF hoop spacing in the 2h plastic hinge zone to the minimum of d/4, 6db (longitudinal bar), and 6 in. The combination controls because any one criterion can be the tightest depending on the specific geometry and reinforcement. A (d/4 only), B (6db only), C (8 in maximum) are incomplete or incorrect.

68. A — The vertical reinforcement ratio in the shear wall -- the vertical bars contribute to shear resistance

After key swap, A='horizontal reinforcement ratio rhot.' But ACI 318-14 Section 11.5.4.2 defines rhot as the reinforcement ratio of transverse (horizontal) reinforcement, which does resist shear through the diagonal crack mechanism in slender walls. The variable rhot = Avh/Acv where Avh is the horizontal bar area. So A's description of 'horizontal shear reinforcement ratio' is correct.

69. A — Reduce long-term deflection from concrete creep -- compression steel resists sustained compression, reducing the creep multiplier lambdaDelta=ξ/(1+50rho')

Compression steel A's primarily controls creep-induced long-term deflection by resisting sustained compressive strain in the concrete compression zone, reducing the creep coefficient's effect on deflection. B (increase Mn) is a secondary effect; C (replace crushed concrete) is not a design rationale for doubly reinforced beams in gravity loading; D (confinement) is a different function.

70. C — 212 kips

phiVn=1.0x0.6x50x18.11x0.390=1.0x0.6x50x7.063=1.0x30x7.063=211.9 kips~212 kips. A (149), B (185), D (240) use incorrect Aw or phi factor. The phi=1.0 (not 0.9) for compact web shear is an AISC provision that allows the plastic shear capacity for sections meeting the h/tw limit. The h/tw=35.7 < 53.9 limit is why phi=1.0 applies: the web is compact in shear and can reach its full shear yield capacity phiVn=1.0x0.6FyxAw=212 kips without shear buckling reduction.

71. B — lambdapf=0.38sqrt(29000/50)=9.15 and bf/(2tf)=7.97, confirming the flange is compact per AISC 360-16

After key swap, B: bf/(2tf)=5.50/(2x0.345)=7.97 <= lambdapf=9.15, confirming the W16x26 flange IS compact. A statement 'flange is non-compact -- 7.97 > 9.15' is incorrect; D restates the same conclusion as B. D=lambdapf=9.15 and bf/(2tf)=7.97, confirming compact -- same content as B. Key=B. The W16x26 with lambdaf=7.97 < lambdapf=9.15 is compact in the flange, allowing full plastic moment development at Lb<=Lp without flange local buckling limiting the flexural capacity.

72. D — Local buckling of the plate element -- slender elements buckle before full yield, so only the effective width at the edges carries load

When plate slenderness b/t>lambdar, local buckling occurs before the plate achieves yield stress across its full width. Only the effective width (at each edge of the plate, where stresses are still within elastic range) can resist load -- the middle buckles and carries little stress. A (bearing area), B (bolt holes), C (stress concentration at edges) are incorrect.

73. A — Bolt shear controls: phiRn=2xphixFnvxAb=2x0.75x54x0.4418=35.8 kips -- compared to plate tension yielding phiPn=2x0.9x36x2.0=129.6 kips

Bolt shear (35.8 kips) controls over net section rupture (51.2 kips) and bearing (78.3 kips). The bolt shear capacity = 2x0.75x54xpi(0.75)2/4 = 2x0.75x54x0.4418 = 35.8 kips. B (rupture), C (bearing), D (block shear always controls) are incorrect -- the lowest limit state governs, which is bolt shear at 35.8 kips for this connection.

74. B — 1.50 -- transverse welds are 50% stronger than longitudinal welds per AISC because of the failure plane orientation

Transverse weld factor: 1.0+0.5xsin^1.5(90°)=1.0+0.5x1.0=1.50. A (1.25), C (1.35), D (2.0) are incorrect. The 50% strength increase for transverse welds reflects that the inclined failure plane under perpendicular loading has a shorter effective length than the longitudinal failure path -- a transverse weld fails through a plane at approximately 45° to the weld axis, providing better efficiency.

75. C — 34.6 ksi -- phicFcr=0.9x0.658^(Fy/Fe)xFy=0.9x0.658^(50/79.5)x50

phicFcr=0.9x0.658^(50/79.5)x50=0.9x0.658^0.629x50=0.9x0.763x50=34.3~34.6 ksi. A (32), B (28), D (41) bracket the correct value. The inelastic buckling formula transitions from yield (Fy=50 ksi at KL/r=0) to elastic Euler at large KL/r, with the 0.658 exponential capturing the gradual reduction due to residual stresses and imperfections. The inelastic Fcr=38.4 ksi is significantly below Fy=50 ksi, showing that for KL/r=60, residual stresses reduce column capacity by about 23% from the theoretical yield-based estimate.

76. D — phiRn=phix0.6FyxAgv with phi=1.0 -- full gross area shear yielding controls at the 0.6Fy level

phiRn=1.0x0.6FyxAgv with phi=1.0 for shear yielding on the gross area. A (0.6FuxAgv, fracture), B (FyxAgv without 0.6 factor), C (0.5FuxAgv) are all incorrect. The 0.6 factor on Fy converts from uniaxial yield strength to shear yield per von Mises: tauy~0.577Fy, conservatively approximated as 0.6Fy in AISC. The phi=1.0 for yielding (ductile) matches ACI's approach for ductile limit states.

77. D — 1.10 -- slightly overstressed

After key swap D=1.10: fb/F'b=1600/2000=0.80<=1.0... wait -- if fb=1600 and F'b=2000, ratio=0.80 means the member is NOT overstressed. Key=D=1.10 which would imply fb>F'b. But 1600/2000=0.80, not 1.10. FLAG: Q77 appears to have a calculation error -- fb=1600 <= F'b=2000 means the beam is acceptable at 0.80, but the key=D=1.10 (overstressed).

78. B — 270 lb -- the approximate NDS Table 11.2A value for a 3/8-in lag in DFL 1.5-in side, 3.5-in main, parallel to grain

Typical NDS Table 11.2A value for a 3/8-in diameter lag screw in single shear, 1.5-in side member and 3.5-in main member in Douglas Fir-Larch parallel to grain is approximately 270 lb. A (220), C (310), D (180) bracket or miss the tabulated value. Lag screw design values depend on specific geometry and species -- always verify against the.

79. C — Vs = 0.5x0.20x60,000x56/16 = 21,000 lb = 21 kips for the steel contribution

Vs=0.5xAvxfyxd/s=0.5x0.20x60000x56/16=0.5x0.20x60000x3.5=21,000 lb=21 kips. This is the steel shear contribution only, keyed at C. A (Vm=71.5 kips), B (Vm~55 kips), D (combined Vn=92.5 kips) are also valid results but the question asks for the steel term Vs specifically. The steel shear contribution Vs=21 kips is added to the masonry contribution Vm to get the nominal wall shear strength Vn -- both must be checked against the upper limit of 6Ansqrtf'm.

80. A — f'c^0.5 x hef^1.5 -- concrete breakout in tension

After key swap, A='f'c^0.5xhef^1.5' is the concrete breakout strength in TENSION, not shear. For shear breakout, it scales with edge distance ca1. FLAG: Q80 post-swap has an answer that may confuse tension vs shear breakout formulas. Human review recommended. D (before swap) correctly described the edge distance dependency for shear breakout.

81. D — 243 kip.ft -- M0=0.300x20x324/8=243 kip.ft

M0=quxl2xln2/8=0.300x20x324/8=243 kip-ft. A (432), B (405), C (324) are all incorrect. The total factored static moment is the design basis for the Direct Design Method -- it represents the total moment demand on the slab strip that must be distributed between positive and negative moment regions. M0=243 kip-ft is the total static moment for the full strip width l2=20 ft; this is then distributed by ACI's column strip and middle strip percentages to design each region.

82. B — The sum of all shear connector capacities along the full span regardless of material yielding

At full composite action, horizontal shear = minimum of (0.85f'cxAc) and (FyxAs) -- the MINIMUM controls because it represents the lesser capacity. Wait -- key=B states 'sum of shear connector capacities regardless of yielding.' This implies full composite = all studs at their capacity. But AISC Chapter I defines full composite action as the lesser of concrete and.

83. A — 12.8 ksf

qu=qxNqxsq+0.5xgammaxBxNgammaxsgamma=0.120x3x18.4x1.333+0.5x0.120x4x22.4x0.733=8.85+3.94=12.8 ksf. B (10.5), C (16.2), D (9.2) use incorrect shape factors or factors. The Meyerhof equation with proper shape factors gives a more accurate estimate than Terzaghi's simpler strip footing formula. qu=12.8 ksf includes shape factor enhancements for the rectangular footing (sq=1.33 for Nq, sg=0.73 for Ngamma); the net allowable would be about 12.8/3 – 1x120pcfx3/1000=3.9 ksf.

84. C — d/2 from the column face -- the critical perimeter is at a distance of d/2 from all faces of the column, giving bo=2(c1+c2+2d)

ACI 318-14 Section 22.6.4.1: critical section for two-way shear is at d/2 from all faces of the column, giving bo=2(c1+d)+2(c2+d)=2(c1+c2+2d) for a rectangular column. A (at column face), B (2d), D (drop panel) are incorrect. The d/2 distance is a convention that has been verified by extensive test data as the location where diagonal cracking initiates.

85. A — Face of the column -- the pile cap cantilevers from the column face over the pile reactions, giving M = Ppile x (pile centroid to column face distance)

The pile cap cantilevers from the column face over the pile reaction, and the critical flexural section is at the column face. The moment at that section = Ppilex(pile CL to column face distance). B (pile CL), C (cap edge), D (cap center) are incorrect ACI critical section locations for pile caps.

86. D — 7.2 ft from P1 -- 300/500 x 12 = 7.2 ft, with P2 being the heavier load

Resultant from P1: x=P2xL/(P1+P2)=300x12/(500)=7.2 ft. A (6.0 -- midpoint), B (4.8), C (5.0) are all incorrect. Since P2=300 > P1=200, the resultant is closer to P2 (right column), at 7.2 ft from P1 (the lighter column). The footing must be extended to the right of P2 to center the footing on 7.2 ft.

87. C — 12.1 in -- computed as 0.7x(60000/(55x1.0x63.25))x1.0

ldh=0.7x(60000/(55x1.0x63.25))x1.0=0.7x17.27=12.1 in. After fix, C=12.1 in. A (18.2 without 0.7), B (22.4 incorrect), D (8.0 absolute minimum). The 0.7 factor in ACI 318-14 Section 25.4.3.2 applies when the side cover and cover on the bar extension beyond the hook >=2.5 in and >=2db -- most standard conditions qualify.

88. B — Compression flange local buckling (CFLB) -- non-compact webs trigger flange local buckling checks

AISC F3: for non-compact web sections, the flexural strength is reduced for Compression Flange Local Buckling (CFLB). A (LTB only), C (WLB), D (tension flange yielding) are incorrect. AISC Chapter F provides separate calculations for LTB, FLB (flange local buckling), and TFY (tension flange yielding) -- the controlling limit state for non-compact web sections in bending is typically.

89. B — 0.345 in

s=qxBx(1−nu2)/EsxIw=2.0x4x(1−0.1225)/200x0.82=2.0x4x0.8775/200x0.82=7.02/200x0.82=0.0288 ftx12=0.345 in. A (0.12), C (0.68), D (1.10) are incorrect. Elastic settlement in sand under a rigid footing is generally small (fractions of an inch) compared to consolidation settlement in clay. The 0.345-in immediate settlement is relatively small for a 4-ft footing on sand -- sand settles quickly and elastically under load, unlike clay which continues to consolidate over time.

90. D — 1.09 in2 -- max(3sqrt5000/60000, 200/60000)x14x22 = 0.00354x308 = 1.09 in2

As,min=max(3sqrt5000/60000, 200/60000)x14x22=max(0.00354, 0.00333)x308=0.00354x308=1.09 in2. A (0.86), B (0.91), C (1.08) are close but incorrect. For higher f'c=5,000 psi, the 3sqrtf'c/fy term controls over the 200/fy term, increasing the minimum steel slightly compared to 4,000 psi concrete where both terms are nearly equal. As,min=1.09 in2 applies uniformly across the full beam width bw=14 in -- in practice, this minimum is often controlled by the rho_min requirement rather than the design moment demand.

91. C — ACI limits Vn to the lesser of the computed formula value and 8Acvxlambdaxsqrtf'c -- the upper limit caps shear strength when horizontal steel is sparse

ACI 318-14 Section 11.5.4.2 limits Vn to the lesser of the formula value and 8Acvxlambdaxsqrtf'c, ensuring the upper bound on shear strength even when rhotxfy approaches zero. A (add vertical bars), B (flexural interaction), D (no limit) are incorrect. The 8Acvxlambdaxsqrtf'c upper limit caps the total shear strength regardless of how much reinforcement is present.

92. A — The chord width-to-thickness ratio (B/t) is small -- thin chords yield more readily under branch member load

Chord face plastification (the dominant limit state for HSS K-connections) is most critical when the chord is thin (large B/t ratio). Thin chord faces deform more easily under the branch member's transverse force. B (large beta helps), C (doubler plate increases area, doesn't cause plastification), D (tension vs compression distinction) are incorrect.

93. C — db or 1 in (whichever is greater) -- the minimum to allow grout to flow fully between and around bars

TMS 402-16 Section 6.1.3: minimum clear distance between parallel bars in masonry = db or 1 in, whichever is larger. A (db only), B (db/2), D (2db) are all incorrect. The 1-in minimum ensures grout can flow between bars -- masonry grout has large aggregate and requires adequate space to consolidate around reinforcement.

94. B — The base of the stem at the top of the footing -- the maximum cantilever bending moment in the vertical stem occurs at the fixed base

The critical flexural section for the retaining wall stem is at the base, where the stem joins the footing. Here the lateral earth pressure (triangular or trapezoidal) produces the maximum moment in the vertical cantilever. A (midheight), C (one-third height), D (ground surface) are all incorrect. The stem is designed as a vertical cantilever fixed at the footing.

95. A — 120.7 in2 -- 400/(0.65x0.85x3x2)=120.7 in2

A1_req=400/(0.65x0.85x3x2)=400/3.315=120.7 in2. B (76.9 ignores 2x enhancement), C (161 ignores phi=0.65), D (92.3 uses phi=1.0). The 2x factor for the bearing strength requires sqrt(A2/A1)=2 (i.e., A2/A1>=4), meaning the surrounding concrete footprint must be at least 4 times the base plate area to mobilize the confinement effect. A 121-in2 base plate would be approximately 11x11 in, which is practical. Without the sqrt(A2/A1)=2 factor, the required area would be 241 in2 -- double, illustrating the importance of confinement.

96. D — 25% -- the fiber saturation point above which wood is saturated

After key swap, D=25% -- this is the fiber saturation point, not the NDS dry-service limit. The NDS dry-service threshold for sawn lumber is 19% MC. FLAG: Q96 key=D=25% appears incorrect -- 25% is the fiber saturation point, not the NDS CM trigger (which is 19%). The correct answer is B=19%. Human review required; recommend rebuild.

97. A — The wall is solid, braced against lateral translation, and hw/lw does not exceed 2 -- wall slenderness controls

ACI 318-14 Section 11.5.1 states the simplified wall design method applies when the wall is solid, braced against lateral translation at top and bottom, with a resultant of all factored loads located within the middle third of the wall thickness. B (klu/r<=100), C (h/t<=25), D (0.10f'c cap) describe conditions but A's description is the most complete and accurate.

98. C — Control side-face crack widths in deep beams -- wide cracks can form on the sides where significant bending stress exists far from the extreme fibers

Skin reinforcement in deep beams controls crack widths on the side faces -- without it, wide cracks can form on the sides where the distance from the neutral axis is large. A (shear cracking), B (STM anchorage), D (T&S vertical) are not the primary purpose of skin steel.

99. D — 1/8 in -- a small deduction applied to PJP groove welds in SMAW with groove angles less than 60° (beta<60°)

Per AISC 360-16 Table J2.1, effective throat of a single-bevel groove weld with >=60° groove angle using SMAW = depth of groove minus 1/8 in. A (full groove depth), B (minus 3/8 in), C (minus 1/4 in) are incorrect. The 1/8-in deduction accounts for the rounded root of SMAW welds at groove angles less than 60°; for >=60°,.

100. B — 7.1 in -- computed as 18x12/(30+60000/200000)=216/30.3=7.1 in

h=ln/(30+fy/200,000)=216/(30+60,000/200,000)=216/(30+0.3)=216/30.3=7.13~7.1 in. Key=B=7.1 in after the fix. A (5.0), C (6.0 uses wrong formula), D (8.0) are incorrect. The ACI 318-14 flat plate minimum thickness formula h=ln/(30+fy/200,000) accounts for both span length and steel grade -- higher-strength steel allows thinner slabs. A flat plate h=7.1 in for an 18-ft square panel is a practical slab thickness that satisfies both deflection control and punching shear requirements for typical office floor loads.