PRACTICE EXAM 19: PE CIVIL STRUCTURAL SIMULATION (100 QUESTIONS)

This practice exam contains 100 questions simulating the NCEES PE Civil – Structural CBT examination. You have 9 hours.

1. Per ASCE 7-16 Section 26.6, the wind directionality factor Kd for main wind force resisting systems of buildings is:

A. Kd = 0.90 — the standard value for building MWFRSs, reflecting the reduced probability that maximum wind speed and worst-case wind direction coincide simultaneously

B. Kd = 0.85 — for chimneys, tanks, and solid signs

C. Kd = 0.95 — for open buildings and lattice frameworks

D. Kd = 1.00 — wind directionality is ignored in the strength design

2. Per ASCE 7-16 Chapter 10, atmospheric ice loads (from freezing rain) must be considered for certain exposed structures. Ice loads are typically significant for which of the following structural elements:

A. Lattice towers, overhead electrical lines, antennas, and exposed slender members — icing multiplies their effective area and wind drag

B. All structures in temperate climates — ice loads govern regardless of building type or geometry

C. Only structures north of 40° latitude per ASCE 7-16 Figure 10.4-2 — an arbitrary latitude cutoff

D. Heavy steel frames only — lightweight structures shed ice accumulation before it builds up

3. Per ASCE 7-16 Section 7.6.1, roof drift loads from snow are required when a lower roof is adjacent to a higher structure (upper roof or wall). The drift load surcharge is shaped as a triangular cross-section with a maximum height hd at the roof-wall junction. The drift width w is taken as:

A. w = 6×hd — a fixed multiple of the drift height

B. w = 8×hd — an increased width for lower roof drifts

C. w = 4×hd — but not greater than 8 ft or the lower roof width

D. w = 4×hd — the ASCE 7-16 Section 7.6 leeward drift width, measured horizontally from the obstruction

4. Per ASCE 7-16 Section 2.3.1 (LRFD Combination 5), the controlling combination for wind with live and dead load is typically 1.2D + 1.0W + 1.0L + 0.5S. The 0.5 factor on snow S in this combination reflects:

A. The standard deviation of snow loads is 50% of the mean — a probabilistic calibration of the load factor

B. Snow is seasonal — the factor accounts for the fraction of the year during which snow load is possible

C. Statistical improbability of maximum wind and maximum snow occurring simultaneously — the 0.5 factor reflects this combined hazard reduction

D. The reduced tributary area effect when wind pressure acts alongside snow distribution

5. Per ASCE 7-16 Section 12.4.3 (Special Seismic Load Combinations), collector elements (drag struts) and their connections must be designed for a seismic force using the overstrength factor $\Omega 0$. The amplified collector force is $Emh=\Omega 0\times QE$ where QE is the seismic force from the lateral analysis. For a typical steel special moment frame building, $\Omega 0$ is:

A. $\Omega 0=3.0$ for special moment frames per ASCE 7-16 Table 12.2-1

B. $\Omega 0=2.0$ for ordinary concentrically braced frames and ordinary moment frames

C. Ω_0=2.5 for special concentrically braced frames and shear wall systems

D. Ω_0=2.5 for intermediate moment frames — a mid-level overstrength factor

6. Per ASCE 7-16 Section 12.3.4.1 and Table 12.3-2, structures with EXTREME torsional irregularity (Type 1b) in SDC E and F are:

A. Permitted with ρ=1.3 — the elevated redundancy factor is intended to compensate for this level of torsional weakness in the lateral system

B. Must use MRSA rather than ELF — the more accurate modal analysis is required but the structure itself remains permissible

C. Permitted without restriction as long as the diaphragm is fully rigid and the seismic force distribution is recomputed accordingly

D. Not permitted — ASCE 7-16 Table 12.3-2 prohibits Type 1b extreme torsional irregularity in SDC E or F

7. Per ASCE 7-16 Section 4.8.1, the minimum floor live load for operating room floors in hospitals is:

A. 40 psf — the minimum for residential and most general occupancy floors

B. 60 psf — for general hospital corridors and waiting areas on the first floor only

C. 80 psf — for hospital corridors above the first floor, but not operating rooms

D. 60 psf — the ASCE 7-16 minimum for operating room floors per Table 4.3-1

8. Per ASCE 7-16 Section 26.7, the surface roughness category used to define wind Exposure Category B requires that the terrain has surface roughness B (including suburban areas, wooded areas, or terrain with numerous small obstructions) extending in the upwind direction from the site for a minimum distance of:

A. 500 ft or 10h upwind — the minimum fetch for rough-terrain classification

B. 1,500 ft or 10h upwind (the larger) — the Exposure B minimum upwind surface roughness B fetch

C. 3,000 ft in the principal wind directions — for Exposure A (not used in ASCE 7-16 directly)

D. 1,000 ft in any direction — a simplified requirement for sites near urban cores

9. Per ASCE 7-16 Section 12.8.6.2, the story drift Δ for Seismic Design Category D (Risk Category II buildings) must not exceed which limit:

A. $0.010h_{sx}$ — for essential facilities (hospitals, fire stations) in Risk Category IV

B. $0.015h_{sx}$ — for Risk Category II structures that are shorter or stiffer than the general case

C. $0.020h_{sx}$ — for Risk Category III and IV structures with more restrictive drift limits

D. $0.025h_{sx}$ — the ASCE 7-16 maximum for Risk Category I and II, $h_n \leq 4$ stories or $T \leq 0.5$ s

10. Per ASCE 7-16 Section 7.3.4, the unbalanced snow load for a sloped gable roof is determined by applying the balanced load $C_s \times p_f$ to the leeward slope and _____ on the windward slope:

A. The full balanced load $C_s \times p_f$ — no reduction for the windward slope

B. 0.0 — the windward slope has no snow load due to drift and wind erosion

C. $0.3 \times C_s \times p_f$ — a 30% reduction from the balanced load on the windward slope

D. $0.5 \times C_s \times p_f$ — half the balanced load on the windward slope represents the typical unbalanced distribution for gable roofs per ASCE 7-16

11. Per ASCE 7-16 Section 11.4.2, the mapped MCER spectral response acceleration parameters S_s and S_1 are obtained from USGS hazard maps or the ASCE 7-16 online tool. These values represent a probability of exceedance of approximately _____ in 50 years for most sites:

A. 2% in 50 years — the MCE (Maximum Considered Earthquake) hazard level used for Ss and S1

B. 10% in 50 years — the 475-year return period used in older codes

C. 50% in 50 years — the 72-year return period for routine construction

D. 2% in 50 years — per ASCE 7-16 Section 11.4.1, the MCE maps are based on 2% probability in 50 years (≈2475-year return period) to represent the maximum considered earthquake motion

12. Per ASCE 7-16 Section 26.7.2, wind Exposure Category B is defined for sites surrounded by terrain with numerous closely spaced obstructions (including suburban, urban, wooded areas). The threshold that determines B versus C for terrain in the upwind direction from the building is:

A. Average obstruction height ≥ 5 ft — any terrain with frequent 5-ft or taller obstacles qualifies for Exposure B

B. Surface roughness B extending ≥ 1,500 ft or 10h upwind in any direction from the site

C. Average surrounding building height ≥ 30 ft within 1,500 ft of the site

D. Tree canopy covering at least 70% of the surrounding land within 500 ft of the site

13. Per ASCE 7-16 Section 12.9.1 (Response Spectrum Analysis), when using RSA (modal response spectrum analysis) for buildings with irregular configurations in SDC D, E, or F, the number of modes included in the analysis must result in a combined modal mass participation of at least:

A. 80% — a common international standard

B. 85% — ASCE 7-16 minimum for regular structures

C. 90% — the minimum combined modal mass participation required for RSA per ASCE 7-16 Section 12.9.1

D. 95% — for structures with significant vertical irregularity

14. Per ASCE 7-16 Section 4.6, roof live loads for ordinary flat, pitched, and curved roofs are assigned a tributary area reduction factor. For a roof member supporting a tributary area A_T=400 ft², the reduced roof live load from the 20-psf minimum is $L_r = L_o \times R_1 \times R_2$ where $R_1 = 1.2 - 0.001 \times A_T$. For A_T=400 ft²:

A. Lr = 20 × 0.80 × 1.0 = 16 psf

B. Lr = 20 × 0.50 × 1.0 = 10 psf

C. Lr = 20 × 0.60 × 1.0 = 12 psf — the minimum permitted roof live load after reduction

D. Lr = 20 × 1.0 = 20 psf — no reduction for AT < 600 ft²

15. Per ASCE 7-16 Table 12.6-1, the Equivalent Lateral Force (ELF) procedure may NOT be used for certain irregular structures in SDC D, E, or F. Which of the following makes a structure ineligible for the ELF procedure in SDC D:

A. The structure has a soft story irregularity (Type 1a or 1b) AND is more than 2 stories above grade

B. The structure has more than 10 stories above grade — any structure over 10 stories must use RSA or time history

C. The building has a setback irregularity (Type 4) at the upper half of its height

D. The structure has a torsional irregularity (Type 1a) and a plan irregular configuration

16. Per ASCE 7-16 Section 12.14.3, the simplified design procedure for seismic base shear $V = F_s \times S_{DS} \times W/R$ where F_s=1.2 may only be applied when SDS does not exceed:

A. 0.30g — the simplified procedure is restricted to low-seismicity regions only, below this SDS threshold

B. 0.50g — the maximum spectral acceleration for applying the simplified base shear procedure

C. 0.60g — for SDC A and B sites where the design ground motion is modest enough to allow this simplified approach

D. No SDS cap — eligibility depends on height, regularity, and SDC, not a maximum SDS value

17. Per ASCE 7-16 Section 12.8.7, the design of structural members designated as collectors must use the load combinations that include the seismic overstrength factor $\Omega0$. A collector in an SDC D building (steel SMF, $\Omega0=3.0$) is designed for a seismic horizontal force QE=40 kips. The required collector force including overstrength is:

A. Emh = $\Omega0 \times QE = 3.0 \times 40 = 120$ kips — the amplified seismic force for collector design

B. Emh = $1.5 \times QE = 60$ kips — a reduced overstrength for routine collector design

C. Emh = QE = 40 kips — no overstrength required for SDC D below 5 stories

D. Emh = $2.0 \times QE = 80$ kips — using the $\Omega0=2.0$ factor for the collector connection only

18. Per the Müller-Breslau principle applied to the conjugate beam method, the slope at a point on a real beam equals the shear in the conjugate beam at the same point. For a simply supported beam (L=20 ft, EI=29,000×500 kip-in²) under a central concentrated load P=30 kips, the slope at the left support is:

A. $\theta = PL^2/(16EI)$ — from moment-area: area of M/EI triangle from the support to midspan

B. $\theta = PL^2/(12EI)$ — using an incorrect coefficient in the moment-area integration of the parabolic M/EI diagram

C. $\theta = PL^2/(16EI)$ — support slope = area of M/EI triangle from A to midspan per moment-area theorem

D. $\theta = 5PL^3/(48EI)$ — the midspan deflection formula for a central point load, not the support slope

19. For a fixed-end beam subjected to a support settlement Δ (downward settlement at B only, span L, EI constant), the fixed-end moments induced at A and B due to the settlement are:

A. $MA = +6EI\Delta/L^2$ (hogging), $MB = -6EI\Delta/L^2$ (sagging)

B. $MA = +3EI\Delta/L^2$ and $MB = -3EI\Delta/L^2$

C. $MA = -6EI\Delta/L^2$ and $MB = +6EI\Delta/L^2$

D. $MA = MB = 6EI\Delta/L^2$ — equal moments at both ends due to chord rotation

20. For a fixed-end beam (span L=10 ft, EI constant) under a linearly varying load (w■=2 kip/ft at A, w■=4 kip/ft at B), the fixed-end moment at A (in kip·ft) is:

A. MFAB = wL²/12 = 3×100/12 = 25.0 kip·ft using the uniform average load w=(w■+w■)/2

B. MFAB = L²(3w■+5w■)/60 = 100(6+20)/60 = 43.3 kip·ft — a wrong coefficient arrangement

C. MFAB = L²w■/12 = 100×2/12 = 16.7 kip·ft — using only w■ and ignoring the varying component

D. MFAB = +L²(7w■+3w■)/60 = 100(14+12)/60 = 43.3 kip·ft — the correct linearly-varying load FEM formula

21. For the collapse mechanism of a fixed-fixed beam (span L, plastic moment Mp) under a single concentrated load P at distance a from the left (b=L−a from the right), the upper bound plastic collapse load is:

A. $Pc = 2Mp(a+b)/(a \times b)$

B. $Pc = 2Mp/a$ — for loads at the third point

C. $Pc = Mp \times L/(a \times b) + Mp/a$ — combining both plastic hinges

D. $Pc = Mp \times (a+b)/(a \times b) + Mp/b$ — an incorrect combination formula

22. For a two-span continuous beam (equal spans L, EI constant, all supports simple), the influence line ordinate for the interior reaction RB at the midspan of the LEFT span (x=L/2 from A) is:

A. IL_RB = 0.313 at midspan — same numerical value as the correct 5/16 but without the derivation

B. IL_RB = 0.25 at midspan — from a simplified tributary area approximation rather than the influence line theorem

C. IL_RB = 0.188 at midspan — this ordinate applies to the quarter-span or third-span position, not midspan

D. IL_RB = 5/16 = 0.313 — the standard influence line ordinate for the interior support at midspan of the adjacent span

23. For a standard C-channel section (web height d=8 in, flange width b=4 in, uniform thickness t=0.25 in), the shear center is located at distance e from the centerline of the web toward the open side. The distance e is:

A. e = 0.75 in — using only the flange contribution without the web correction

B. e = 1.50 in — $e=3b^2tf/(6b\times tf+d\times tw)=12/8=1.50$ in from the web centerline

C. e = 2.00 in — a larger offset that would apply to a wider-flange channel

D. e = 1.00 in — an approximate value often used for preliminary design

24. Maxwell's Reciprocal Theorem states δ■■=δ■■ for a linearly elastic structure. The most direct engineering application of this theorem is:

A. Computing principal stresses from Mohr's circle — a separate elastic mechanics tool

B. Determining the column buckling load — stability requires a different analysis approach

C. Reducing flexibility coefficient computations in structural analysis — knowing δ■■=δ■■ saves work

D. Confirming stiffness matrix symmetry — Maxwell's theorem is the equilibrium basis for k■■=k■■ in structural matrices

25. For a parabolic two-hinged arch (span L=60 ft, rise f=12 ft) under a uniform load w=2 kip/ft along the full span, the horizontal thrust H at the supports is:

A. $H = wL^2/(8f) = 2\times3600/(8\times12) = 75$ kips — the standard two-hinged arch formula for uniform load

B. $H = wL/(4f) = 2\times60/(4\times12) = 2.5$ kips — using L instead of L^2 in the numerator

C. $H = w\times L/(2f) = 2\times60/(2\times12) = 5.0$ kips — using L/2f instead of $L^2/8f$

D. $H = wL^2/(4f) = 2\times3600/(4\times12) = 150$ kips — a factor-of-2 error

26. For the method of virtual work applied to a statically determinate truss, the displacement of a joint in a given direction due to external loads is computed as:

A. $\Delta = \Sigma FL/(AE)$ — the sum of elongations without the virtual unit force member forces u

B. $\Delta = \Sigma$(real force × virtual elongation) — an alternative, equivalent form of virtual work

C. $\Delta = \Sigma(F\times L\times u)/(AE)$ — the correct expression where u=member forces from a unit virtual load at the desired displacement point

D. $\Delta = \Sigma F^2L/(2AE)$ — the strain energy expression, not the virtual work displacement formula

27. For a beam-column under combined axial compression P and bi-axial bending (Mx and My), the AISC 360-16 H1 interaction equation uses the plastic moments Mpx and Mpy (not elastic moments Sx×Fy and Sy×Fy) because:

A. Plastic moments are always larger than elastic moments — using them makes the AISC H1 check less conservative for compact sections

B. AISC H1 uses M_{nx} and M_{ny} (equal to M_p for compact sections without LTB) — the plastic moment is the correct design limit per Section F2

C. Computing plastic moments is simpler — it avoids looking up elastic section moduli from steel tables for I-shaped sections

D. AISC H1 is based only on elastic section moduli — the interaction equations use S rather than Z for both bending directions

28. For the direct stiffness method, the stiffness matrix of an inclined bar element (inclined at angle θ from the horizontal) in global coordinates is obtained by transforming the local stiffness matrix through [K_global]=[T]■[k_local][T] where [T] is the transformation matrix. For a tension/compression bar element, [T] is:

A. $[\cos\theta, \sin\theta; -\sin\theta, \cos\theta]$ — the standard 2×2 rotation matrix

B. $[\cos\theta, \sin\theta, 0, 0; -\sin\theta, \cos\theta, 0, 0; 0, 0, \cos\theta, \sin\theta; 0, 0, -\sin\theta, \cos\theta]$ — the 4×4 transformation matrix relating local to global DOFs for a 2D truss element

C. The identity matrix [I] — transformation is not needed for inclined elements

D. $[\cos^2\theta, \sin\theta\cos\theta; \sin\theta\cos\theta, \sin^2\theta]$ — the direction cosine matrix directly

29. For a frame with sidesway, after applying the slope-deflection equations to all members and joint equilibrium conditions, the resulting system of equations is solved for the unknown rotations and chord rotations. The number of independent equations equals:

A. The total number of DOFs equals joints times DOFs per joint — one equation per DOF in the full system

B. The number of spans plus the number of independent sway cases — a partial count only

C. Member ends minus boundary conditions — an energy-based count not matching the slope-deflection approach

D. The number of unknown joint displacements (independent rotations plus independent sway DOFs) — exactly one equilibrium equation per unknown

30. For a cantilever beam (fixed at A, free at B, span L=10 ft) with a moment release introduced at distance a=6 ft from A, the structure becomes:

A. A fixed-fixed beam — the moment release stiffens the interior

B. Unstable — releasing a moment in a cantilever creates a mechanism

C. A stable structure with a pinned connection at 6 ft and a free end at B

D. An unstable mechanism — a moment release in a cantilever creates a locus where the moment is forced to zero but the cantilever already has zero moment at the free end; the release at 6 ft from A makes the 6-ft segment from A to the release a cantilever, and the 4-ft segment beyond the release is a free-body needing the release reaction to be zero

31. For modal superposition (RSA) of a 3-story shear building, the maximum story shear at the first floor from Mode 1 alone is 200 kips, and from Mode 2 alone is 50 kips. Using SRSS combination, the best estimate of the maximum first-floor story shear is:

A. $V_{SRSS} = \sqrt{(200^2+50^2)} = \sqrt{(40{,}000+2{,}500)} = \sqrt{42{,}500} \approx 206$ kips

B. V_{SRSS} = 200+50 = 250 kips — the absolute sum (conservative upper bound)

C. V_{SRSS} = 200 kips — the first mode dominates and the second mode is neglected

D. V_{SRSS} = 250×0.83 = 208 kips — using a rough SRSS approximation without the exact formula

32. For the computation of member end forces in a frame element using the direct stiffness method, after solving for global nodal displacements {U}, the element forces in local coordinates are:

A. f_local = [k_local]{u_local} — where {u_local} = [T]{U} transforms global to local element displacements

B. f_local = [K_global]{U} — no transformation needed

C. f_local = [T]■{F_external} — back-transform the applied forces

D. f_local = [k_local][T]{U} + {FEM} — including the fixed-end moments for distributed member loads not captured in the nodal load vector

33. For a propped cantilever (fixed at A, roller at B, uniform EI, span L) analyzed by moment distribution, the distribution factor at joint B (where only one member frames in) is:

A. DF_B = 0.5 — a standard distribution factor for two members of equal stiffness meeting at a joint

B. DF_B = 1.0 — the entire unbalanced moment at the roller end is released to the only member framing into that joint

C. DF_B = 0.0 — the roller provides no moment resistance and the distribution factor is zero

D. DF_B = 0.67 — for two-thirds distribution to the span member

34. The complementary energy for a linearly elastic structure under applied loads equals the strain energy. For a nonlinear elastic structure where the load-deflection relationship is P=kδ■ (n≠1), the complementary energy C* differs from strain energy U because:

A. C* = U only when n=1 (linear); for n≠1, C*=∫δdP while U=∫Pdδ — they give different integrals

B. C* = 0 for nonlinear structures — no energy is stored

C. C* = 2U — the complementary energy is always double the strain energy

D. C* is undefined for nonlinear structures — Castigliano's theorems cannot be applied

35. For a moving load traversing a simply supported beam (span L), the critical position for MAXIMUM bending moment (not maximum midspan moment) under a single concentrated load P occurs at:

A. x = L/4 from the left support — the quarter-span position

B. x = L/2 — midspan is always the critical position for maximum moment

C. Under the load itself — for a single concentrated load P, the maximum moment equals P×a×b/L where a is the distance from the left support to the load and b=L–a; this is maximized at a=b=L/2 (midspan)

D. At the fixed end — for a fixed-pinned beam only

36. For a Timoshenko beam element (accounting for shear deformation), the total midspan deflection of a simply supported beam under central load P is δ=δ_bending+δ_shear where δ_shear=PL/(κAG). The shear correction factor κ for a solid RECTANGULAR section is approximately:

A. κ = 0.833 — valid for rectangular cross-sections per Timoshenko's shear correction

B. κ = 0.750 — for wide-flange steel sections dominated by web shear

C. κ = 1.0 — shear deformation is uniform across the rectangular section

D. κ = 0.667 — the exact value for rectangular sections per the linear stress distribution assumption

37. For a LRFD design of a tension member splice (two 7/8-in A325 bolts, double shear), the nominal bolt shear strength per AISC Table J3.2 using the threads-excluded-from-shear-plane (A325X) condition gives Fnv=84 ksi. The design shear strength per bolt is:

A. $\phi Rn = 0.75\times84\times\pi(0.875)^2/4 = 37.9$ kips per bolt (single shear)

B. $\phi Rn = 0.65\times84\times2\times\pi(0.875)^2/4 = 65.7$ kips — using the wrong $\phi=0.65$

C. $\phi Rn = 0.90\times84\times\pi(0.875)^2/4 = 45.5$ kips per bolt

D. $\phi Rn = 0.75\times84\times2\times\pi(0.875)^2/4 = 75.8$ kips per bolt (double shear)

38. For a frame with pinned bases and a rigid beam (infinitely stiff girder) on top, the lateral stiffness of the frame (force per unit sway displacement Δ) is:

A. $k = \Sigma(12EI/h^3)$ — for fixed-base, fixed-top columns in a sway frame

B. $k = \Sigma(3EI/h^3)$ — each pinned-base column with rigid top beam has cantilever stiffness $3EI/h^3$

C. $k = \Sigma(EI/h^3) \times$ factor — a general expression without resolved boundary conditions

D. $k = EI/h^3$ — the single-column stiffness term before summing

39. For a two-span continuous beam with different span lengths L■ and L■ (EI constant, UDL w on each span), the moment at the interior support B from the three-moment equation is:

A. $MB = -wL\blacksquare^2/8$ if $L\blacksquare > L\blacksquare$

B. $MB = -w(L\blacksquare^2+L\blacksquare^2)/16$ — using the average of both spans

C. $MB = -wL\blacksquare L\blacksquare(L\blacksquare+L\blacksquare)/(4(L\blacksquare^2+L\blacksquare L\blacksquare+L\blacksquare^2))$ — an approximate result

D. $MB = -w(L\blacksquare^3+L\blacksquare^3)/(4(L\blacksquare+L\blacksquare))$ — the exact result from the three-moment equation for equal EI and unequal spans with UDL on both

40. For a structural system with proportional damping (Rayleigh damping), increasing the β coefficient (stiffness proportional damping) primarily increases the damping ratio ξ at:

A. Low frequencies (long periods) — β-damping adds damping proportional to stiffness, which is significant for flexible structures

B. All frequencies equally — Rayleigh damping is frequency-independent

C. High frequencies (short periods) — $\beta\times$stiffness damping increases with frequency ω, providing more damping at higher modes

D. The first natural frequency only — β-damping is tuned to the fundamental mode

41. For a truss structure, the virtual work expression for the deflection at a joint in the direction of an applied unit virtual load is $\Delta = \Sigma(F\times\delta\blacksquare L)/(AE)$ where $\delta\blacksquare$ represents the unit-force member forces. If the real loads cause a member to elongate by $\varepsilon=F\times L/(AE)$, then the virtual work equation is equivalent to:

A. $\Delta\Sigma = \Sigma$(virtual force × real elongation) — the principle of virtual work for trusses

B. $\Delta = \Sigma$(real force × virtual elongation) — an alternative form of the method

C. $\Delta = \Sigma F^2L/(AE)$ — the strain energy formula which is not the virtual work expression

D. $\Delta = \Sigma F/(AE)$ — missing the length term and confused with force, not work

42. In a multi-story building, the story stiffness ki of a floor is defined as the story shear Vi divided by the story drift Δi. A soft story exists per ASCE 7-16 Section 12.3.2 when the lateral stiffness of a story is less than _____ of the stiffness of the story above:

A. 70% of the story above (Type 1a soft) or 60% (Type 1b extreme soft) — the ASCE 7-16 thresholds

B. 70% and 60% — the same thresholds as A but stated in different order

C. 90% — a much stricter trigger that would flag most multi-story buildings with irregular stiffness

D. 75% and 65% — fictional thresholds not found in ASCE 7-16 Table 12.3-1

43. Per OSHA 29 CFR 1926 Subpart P, for a Type A soil excavation (cohesive, unconfined compressive strength qu > 1.5 tsf, not fissured), the maximum allowable slope for a cut with height greater than 4 ft is:

A. 3/4H:1V — for Type A soil this is the maximum allowable slope per OSHA Appendix B

B. 1H:1V — the OSHA maximum allowable slope for Type B soil (not Type A)

C. 1/2H:1V — steeper than 3/4H:1V, suggesting a different soil or engineer-designed cut

D. 3/4H:1V — same as option A, the correct Type A soil limit per OSHA 29 CFR 1926 Appendix B

44. Per ACI 347R-14 Section 4.3.1, the minimum design load for concrete formwork systems must include:

A. Dead loads from concrete and formwork plus a construction live load of at least 50 psf

B. Dead loads from concrete and formwork plus a minimum construction live load of 25 psf (or 50 psf if powered concrete buggies are used)

C. Full hydrostatic pressure of concrete plus a 25 psf construction live load

D. The weight of concrete plus an impact factor of 1.25 — no separate live load is needed

45. Per OSHA 29 CFR 1926.502, personal fall arrest systems for construction workers must be rigged so that an employee cannot fall more than _____ feet, and cannot contact a lower level:

A. 4 ft free-fall maximum — for workers on lightweight scaffolding systems with limited energy absorption

B. 6 ft maximum free-fall AND no contact with a lower level — the OSHA 29 CFR 1926.502 fall arrest system requirement

C. 8 ft free-fall limit — for heavy-duty systems designed for concrete placement and formwork operations

D. 12 ft free-fall limit — applicable only to systems with certified energy-absorbing lanyards and double-action hooks

46. Per ACI 347R-14 and construction practice, gang forms (large prefabricated wall form panels moved by crane) offer which primary advantage over hand-set forms for walls:

A. Reduced concrete pressure — gang forms are stiffer and limit hydrostatic load

B. Lower initial cost — gang form systems cost less than hand-set forms to fabricate

C. Faster set/strip, better surface quality, fewer form ties — craned as a unit, dramatically cutting labor

D. No need for form ties — gang forms are entirely self-supporting

47. Per ACI 347R-14 and structural engineering practice, re-shoring (installing new shores below a slab after its own form-shoring system has been removed) is required primarily when:

A. The slab is less than 28 days old — re-shoring is always required until 28 days

B. The upper stories' construction loads will be transferred through the newly placed slab before it reaches adequate strength to support those loads independently

C. The form pressure exceeds 500 psf — hydrostatic pressure requires reshoring

D. The concrete mix uses fly ash — pozzolanic cements require extended shoring

48. Per OSHA 29 CFR 1926.703(b), shoring and reshoring of multi-story concrete structures must be designed and installed under the supervision of:

A. A licensed general contractor's superintendent — OSHA defers to GC experience for temporary works

B. A licensed professional engineer — OSHA 29 CFR 1926.703(b) requires shoring be designed by a licensed PE or architect

C. An ACI Level 3 field concrete inspector with 5+ years multi-story experience

D. Any person with documented 5 years of concrete construction experience under a licensed contractor

49. Per ACI 347R-14 Section 6, when setting concrete column forms for a multi-story building where the column cage has been spliced above the construction joint, the form must be seated to prevent:

A. Concrete spillage through the form bottom — column forms must be sealed at the base to prevent grout leakage and aggregate segregation

B. Hydrostatic pressure exceeding 3,000 psf — pressure governs column form design

C. Vibrator damage — the vibrator frequency can break loose improperly seated forms

D. Thermal expansion — temperature differentials between fresh concrete and the form can unseat the column form panels

50. Per ASTM A572 Grade 50, the minimum mechanical properties that must be met for structural steel shapes (angles, channels, W-sections, and plates) are Fy=50 ksi minimum and Fu=65 ksi minimum. Compared to A36 (Fy=36 ksi, Fu=58 ksi), the A572 Grade 50 provides:

A. Higher yield strength, which allows lighter sections for strength-governed designs where the member yield governs over fracture or stiffness

B. Better corrosion resistance — A572 has improved weathering properties compared to A36

C. Greater elongation at rupture — A572 is more ductile than A36

D. Lower weldability — A572 Grade 50 requires more preheat than A36

51. Per ACI 318-14 Section 26.4.2.2, for concrete with a specified compressive strength f'c > 5,000 psi, ACI requires that the mix design be based on field experience or trial batches. The average compressive strength f'cr used for proportioning must exceed f'c by a margin that accounts for:

A. The statistical variability of concrete production (standard deviation s) — f'cr = f'c + 1.34s or f'c + 2.33s – 500 psi, whichever is greater, ensuring a low probability of field strengths falling below f'c

B. A fixed 10% safety margin — f'cr = 1.10×f'c for all concrete above 5,000 psi

C. The loss of strength due to temperature — f'cr adjusts for cold-weather concrete placement

D. The difference between cylinder and core strengths — cores are always 15% weaker

52. Per ASTM C150 and ACI 318-14, Type V Portland cement (high sulfate resistance) is specified when concrete will be exposed to soil or water with sulfate concentrations exceeding:

A. 0.10% by mass in soil or 150 ppm SO■ in water — the threshold for Class S2 exposure in ACI 318-14

B. 0.20% by mass in soil or 1,500 ppm in water — the moderate sulfate threshold for Type II cement

C. 0.60% by mass in soil or 1,500 ppm in water — the severe sulfate threshold requiring Type V cement with w/cm ≤ 0.45

D. 0.20% soil or 150 ppm water — the lower limits for Type II cement use

53. Per NDS 2018 Section 4.3.1, the Wet Service Factor CM for sawn lumber under GREEN (unseasoned, MC>19%) moisture conditions is less than 1.0 for most properties. Which NDS design value is NOT reduced by CM (CM=1.0) for sawn lumber in wet service?

A. Fv — shear parallel to grain — always reduced by CM=0.97

B. E_min (minimum modulus of elasticity for stability calculations) — CM=1.0 — the modulus used for stability (Emin) is not reduced for green conditions in sawn lumber

C. Fb — bending design value — CM=0.85 for most sizes

D. Fc⊥ — compression perpendicular to grain — CM=0.67

54. Per ASTM A36, the maximum carbon content limitation for plates greater than 3/4 in thick compared to plates 3/4 in or less reflects:

A. Thicker plates have lower carbon content — A36 reduces C% with increasing thickness to maintain weldability as restraint increases

B. Thicker plates have higher carbon content — the strength can be maintained at lower alloy content with thicker sections

C. Carbon content is fixed at 0.26% maximum for all A36 thicknesses — no variation with plate thickness

D. Thicker A36 plates require higher carbon for the yield strength minimum of 36 ksi, since slower cooling rates reduce grain refinement effects

55. Per NDS 2018, Southern Yellow Pine (SYP) typically has higher published design values for bending (Fb) than Douglas Fir-Larch (DFL) for the same visual grade and size. This is because:

A. SYP has a higher average specific gravity (≈0.55 vs ≈0.50 for DFL) and grows in a denser grain pattern, both of which contribute to higher mechanical properties

B. SYP is more widely tested — more data points give higher confidence intervals

C. SYP is seasoned at a higher temperature — heat treatment increases strength

D. SYP grows faster than DFL — faster growth produces stiffer wood

56. Per ASTM A490 structural bolts (compared to A325), the nominal tensile strength Fu is:

A. Fu = 120 ksi — the same as A325 bolts (incorrect, A325 is lower)

B. Fu = 150 ksi — the minimum nominal tensile strength for A490 bolts per ASTM F3125

C. Fu = 105 ksi — less than A325, which would make A490 weaker

D. Fu = 120–130 ksi range — the same as A325 depending on bolt diameter

57. Per ACI 301-16 and ACI 308R-16, for concrete in dry-in, wet-out exposure (concrete exposed to drying while in contact with moisture on one face), the required minimum curing period for concrete with Type I cement at 50°F ambient temperature is:

A. 3 days — the absolute minimum for accelerated-cure or warm-weather placements

B. 5 days — adequate for ambient temperatures above 70°F with Type I cement

C. 7 days — the ACI 308R minimum curing period for Type I cement at 50°F ambient

D. 14 days — the minimum for high-performance concrete with f'c above 5,000 psi

58. Per ASTM D2487 (USCS), the boundary between GRAVEL (G) and SAND (S) in the Unified Soil Classification System is:

A. The #4 sieve (4.75 mm) — particles retained on the #4 sieve are gravel, particles passing are sand

B. The #8 sieve (2.36 mm) — a finer distinction than the AASHTO system

C. The #10 sieve (2.00 mm) — USCS uses the #10 as the gravel/sand boundary

D. The 4.75-mm opening, which corresponds to the #4 sieve — the USCS standard gravel-sand boundary

59. Per ACI 318-14 Section 14.5.3, for a structural plain concrete footing (unreinforced, limited to SDC A, B only), the minimum permitted thickness is:

A. 6 in — a practical minimum for slab-on-grade applications

B. 8 in — for structural plain concrete footings on grade with light loads

C. 10 in — a conservative designer choice for stiff soils at greater depth

D. 10 in — the ACI 318-14 Section 14.5.3 minimum for structural plain concrete footings

60. Per ASTM C989 and ACI 233R, the performance advantage of using ground granulated blast-furnace slag (GGBFS) as a cement replacement in marine concrete structures is:

A. Increased rate of chloride diffusion — GGBFS accelerates chloride transport

B. Reduced permeability and refined pore structure — GGBFS reacts with Ca(OH)■ to form additional C-S-H gel that densifies the paste, significantly reducing chloride and sulfate penetration

C. Higher early strength — GGBFS accelerates the early hydration rate

D. Reduced unit weight — GGBFS has a much lower specific gravity than Portland cement

61. Per NDS 2018 Chapter 3 and Section 4.4.2, the timbers classification (5-in × 5-in and larger, width not more than 2 in greater than thickness) for sawn lumber is used for structural members where:

A. High appearance grades are required — timbers are aesthetically superior to dimension lumber

B. Only Select Structural or No. 1 grades are available — smaller grades don't exist for timbers

C. The knot and defect limitations differ from dimension lumber — timber grades (Select Structural, No. 1, No. 2) account for large knots relative to section size, reflecting loading in bending or compression rather than the light framing loads of dimension lumber

D. Fire resistance is required — timbers have inherent fire resistance due to their mass and char rate

62. Per ASTM A53 Grade B (welded or seamless steel pipe used structurally), the minimum yield strength is:

A. $F_u = 30$ ksi — the minimum for ASTM A53 Grade A pipe

B. $F_y = 35$ ksi — the minimum yield strength for A53 Grade B pipe per ASTM A53

C. $F_y = 42$ ksi — above A53, typical of A500 Grade B round HSS

D. $F_y = 25$ ksi — below the Grade A minimum, not found in ASTM A53

63. Per ACI 318-14 Section 26.4.1, the fundamental rule for proportioning concrete mixes is that the target average compressive strength f'cr (used for mix design) must exceed the specified f'c by a margin that depends on the standard deviation s of the concrete producer's quality control. If a producer has s=400 psi, the larger of the ACI requirements for f'cr (for f'c=4,000 psi) is:

A. $f'_{cr} = 4{,}000 + 1.34\times400 = 4{,}536$ psi

B. $f'_{cr} = 4{,}000 + 2.33\times400 - 500 = 4{,}432$ psi

C. $f'_{cr} = 1.10\times4{,}000 = 4{,}400$ psi — a flat 10% overage

D. $f'_{cr} = 4{,}000 + 1.65\times400 = 4{,}660$ psi — using a 1.65 factor instead of 1.34

64. Per ACI 318-14 Section 16.5 (Corbels), the required area of shear-friction reinforcement Avf for a corbel carrying a factored shear Vu=50 kips (cast monolithically, λ=1.0, fy=60 ksi, μ=1.4) is:

A. $A_{vf} = V_u/(\phi\times\mu\times f_y) = 50/(0.75\times1.4\times60) = 0.794$ in² for monolithic cast corbel shear friction

B. $A_{vf} = 0.5$ in² — for a much smaller corbel with $V_u \approx 19$ kips

C. $A_{vf} = 1.2$ in² — for the combined shear plus horizontal tensile force case

D. $A_{vf} = 1.5$ in² — an overestimate for corbels with large tensile demands

65. Per ACI 318-14 Section 22.7, the torsional threshold moment above which ACI requires torsion reinforcement (for a non-prestressed solid rectangular beam, b=12 in, h=24 in, f'c=4 ksi) is:

A. $T_{u\_threshold} = \phi\times0.083\lambda\sqrt{f'_c}\times(A_{cp}^2/p_{cp}) = 0.75\times0.083\times1.0\times63.25\times(288^2/72)/12 = 0.38$ kip-ft

B. $T_{u\_threshold} = 0.50$ kip-ft — a rounded design value

C. $T_{u\_threshold} = 1.5$ kip-ft — for larger sections

D. $T_{u\_threshold} = 0.75\times0.083\times\sqrt{4000}\times288^2/(72\times12{,}000) \approx 0.38$ kip-ft — this is the ACI minimum threshold; torsion below this can be neglected

66. Per ACI 318-14 Section 18.6.5, the maximum hoop spacing within the special moment frame beam plastic hinge region is the SMALLEST of:

A. $s \le d/4$, 8×longitudinal bar diameter, 24×hoop bar diameter, and 12 in

B. $s \le d/4$, 6×longitudinal bar diameter, 6 in — the three governing criteria for the SMF beam within Lo of the column face

C. $s \le d/2$, 8×longitudinal bar diameter, 24×hoop bar diameter

D. $s \le d/4$, 6×longitudinal bar diameter, 150 mm (6 in) — the ACI 318-14 Section 18.6.4.4 criteria for SMF beam hoops in the plastic hinge region

67. Per ACI 318-14 Section 11.5 (Wall Design), the effective length factor k for a wall between lateral supports that is not braced against sidesway equals:

A. k = 1.0 — for all walls per a simplified assumption

B. k = 0.8 — for walls with two horizontal supports and restrained rotation

C. k = 2.0 — for cantilever walls (one support at the base only)

D. k = 0.8 or 1.0 depending on support conditions — per ACI 318-14 Table 6.2.2, for walls braced against lateral movement at both ends k=0.8; for walls with one end free k=2.0; k=1.0 for pinned-pinned

68. Per AISC 360-16 Section F1, the limit state governing the nominal flexural capacity of a compact, doubly symmetric I-shaped member at a braced length Lb=0 is:

A. Lateral-torsional buckling — LTB always governs for any unbraced length

B. Web local buckling — the slender web limits the capacity even at Lb=0

C. Yielding — Mn=Mp=Fy×Z, the plastic moment capacity, for compact sections with Lb≤Lp

D. Flange local buckling — compact flanges still limit the capacity slightly below Mp

69. Per ACI 318-14 Section 22.7.5, the minimum area of closed torsion stirrups At/s for a non-prestressed beam in a region requiring torsion reinforcement is:

A. At/s = Vu×cot²θ/(ϕ×fy×2Ao) — the torsion design equation for required closed stirrup area

B. At/s ≥ 0.175bw/fy — the ACI minimum for torsion reinforcement when Tu exceeds the threshold

C. At/s = 0.85Vu/(ϕ×fy×jd) — the beam shear formula incorrectly adapted for torsion

D. At/s = 0.75√f'c×bw/fy — the shear minimum formula, applied here in the wrong context

70. Per AISC 360-16 Section J2, the throat area of a fillet weld is used to compute the nominal strength of the weld. For a 3/8-in fillet weld, the effective throat dimension is:

A. 0.265 in — the throat = 0.707×weld size = 0.707×3/8 = 0.265 in for a 45° fillet

B. 3/8 in — the full weld leg size

C. 0.375 in — using the full weld size for the effective area

D. 0.188 in — using half the weld size as a conservative estimate

71. Per ACI 318-14 Section 25.4.9, for headed deformed bars (ASTM A970), the development length ldh for a headed bar in tension compared to a standard straight bar ld is:

A. ldh = 0.25×fy×db/(λ×√f'c) for headed bars — significantly shorter than straight bar ld because the head provides bearing resistance at the embedded end

B. ldh = 1.5×ld — the head adds friction that requires extra development length

C. ldh = 0.5×ld — headed bars develop in half the length of straight bars

D. ldh = ld — headed bars develop the same length as straight bars with no benefit

72. Per ACI 318-14 Section 17.4.5, for a cast-in-place anchor bolt (headed anchor, concrete breakout failure mode, tension loading), increasing the effective embedment depth hef increases the breakout capacity by a factor of:

A. hef^1.0 — breakout capacity increases proportionally with embedment depth (linear)

B. hef^2.0 — the breakout failure cone surface area increases as the square of embedment depth

C. hef^1.5 — per ACI 318-14 Section 17.4.2.2, reflecting combined area and stress distribution effects

D. hef^2.5 — for deeply embedded anchors subject to confining effects from surrounding reinforcement

73. Per TMS 402-16 Section 8.1.6, the minimum distance between reinforcing bars in a masonry wall (center-to-center spacing) must be at least:

A. db or 1 in — a somewhat relaxed masonry requirement

B. 1.5db or 1.5 in — the larger of one and one-half bar diameters or 1.5 inches

C. db — one bar diameter minimum

D. d_b + clear space of 1 in — ensuring paste can flow between bars

74. Per ACI 318-14 Section 13.5, the critical section for punching shear in a flat slab with an interior column on grade (mat foundation) must be verified at:

A. d from the column face — the one-way shear critical section (not punching)

B. d/2 from the column face — the standard punching perimeter for two-way shear

C. The column face — the ACI critical section is right at the column-slab interface

D. bo = perimeter at d/2 from the column face is always used for punching regardless of grade or suspended slab — the d/2 offset critical perimeter is the ACI standard for all two-way flat slab punching

75. Per AISC 360-16 and the AISC Design Guide for shear connections, a single-plate shear connection (shear tab) with three 3/4-in A325N bolts in single shear has a bolt shear design strength per bolt of ϕRn=ϕ×0.48×Fu×Ab. For A325N in single shear (Fnv=54 ksi per AISC Table J3.2):

A. $\phi Rn = 0.75\times48\times\pi(0.75)^2/4 = 15.9$ kips — using Fnv=48 ksi (older A325N table value)

B. $\phi Rn = 0.75\times54\times\pi(0.75)^2/4 = 17.9$ kips — the design shear strength per bolt for A325N in single shear

C. $\phi Rn = 0.75\times72\times\pi(0.75)^2/4 = 23.8$ kips — using an overly high Fnv=72 ksi

D. $\phi Rn = 0.90\times54\times\pi(0.75)^2/4 = 21.5$ kips — using the wrong ϕ factor for bolt shear

76. Per AISC 360-16 Section H2 (non-symmetric sections), for an angle used as a beam under a bending moment causing bending about an axis that is NOT one of the principal axes, the maximum bending stress is:

A. fb = M/Sx — using only the x-axis section modulus regardless of load direction

B. fb = My/Ix — for moments about the y-axis only

C. The flexural formula fb = M×c/I must use the appropriate principal axis moments of inertia, accounting for bi-axial bending about both principal axes — for angles, bending about the geometric axis x-x or y-y induces biaxial bending about both principal axes simultaneously

D. Angle sections cannot be used as beams — AISC 360-16 prohibits unsymmetric sections in bending

77. Per ACI 318-14 Section 11.7.2 (Shear Wall Design), for a special structural wall with hw/lw < 2 (squat shear wall where Vn is controlled by shear), the nominal shear strength in the web is:

A. $Vn = Acv\times(\alpha c\times\lambda\times\sqrt{f'c} + \rho t\times fy)$ — where $\alpha c=3.0$ for $hw/lw\leq1.5$ and linearly decreases to 2.0 for $hw/lw\geq2.0$

B. $Vn = 0.2\times f'c\times Acv$ — a simplified upper limit

C. $Vn = Vc + Vs = 2\lambda\sqrt{f'c}\times bw\times d + Av\times fy\times d/s$ — the standard beam shear formula

D. $Vn = 4\lambda\sqrt{f'c}\times Acv$ — a simple upper bound for squat walls without reinforcement terms

78. Per AISC 360-16 Section G2.1, for a wide-flange member with a compact web ($h/t_w \le 2.24\sqrt{(E/F_y)}$) where shear yielding governs, the nominal shear strength is:

A. $V_n = 0.6F_y \times A_w \times C_v = 0.6F_y \times A_w \times 1.0$ — with the full web area $A_w = d \times t_w$

B. $V_n = 0.5F_y \times A_w$ — using the average shear stress at yield

C. $V_n = 0.36F_y \times A_w$ — applying a 0.6 factor to the web area

D. $V_n = F_y \times A_w/\sqrt{3}$ — from the von Mises yield criterion

79. Per TMS 402-16, the design of an anchor bolt embedded in masonry (in tension) uses the least of three failure modes: masonry breakout, anchor steel yielding, and anchor bolt pullout. For a 5/8-in diameter anchor bolt with 4-in embedment in concrete masonry (f'_m=1,500 psi), the masonry breakout capacity B_{ab} per TMS 402-16 Eq. 9-2 is:

A. $B_{ab} = 0.75 \times A_{pt} \times \sqrt{f'_m}$ — applying a strength reduction to the breakout area

B. $B_{ab} = 4A_{pt} \times f'_m$ — using the full masonry compressive strength

C. $B_{ab} = 4A_{pt} \times \sqrt{(f'_m/100)}$ — a normalized formula

D. $B_{ab} = 4A_{pt} \times \sqrt{f'_m} = 4 \times (\pi \times 4^2/4) \times \sqrt{1500}/1000$ — the breakout force in kips

80. For a rectangular retaining wall (stem height H=12 ft, base width B=8 ft, base thickness=1.5 ft, soil unit weight γ=125 pcf, ϕ=32°, c=0), the factor of safety against OVERTURNING per ACI 318-14 and geotechnical practice (taking moments about the toe) is FS=$\Sigma M_{resisting}/\Sigma M_{overturning}$. If $\Sigma M_{resisting}$=200 kip-ft/ft and $\Sigma M_{overturning}$=60 kip-ft/ft:

A. FS = (200–60)/60 = 2.33 — using net moment

B. FS = 60/200 = 0.30 — a ratio below 1.0 indicates failure

C. FS = 200/60 = 3.33 — above the typical minimum of 2.0 for overturning

D. FS = $\sqrt{(200/60)}$ = 1.83 — a geometric mean stability ratio

81. Per ACI 318-14 Section 22.6.5.2, the maximum punching shear strength for a two-way slab with shear reinforcement (shear studs or stirrups) is limited to $\phi V_{n,max}$. For a slab with shear stud reinforcement (ASTM A1044), the maximum ϕV_n is:

A. $\phi V_{n,max} = \phi \times 6\lambda\sqrt{f'_c} \times b_o \times d$ — for slabs with shear stud reinforcement

B. $\phi V_{n,max} = \phi \times 8\lambda\sqrt{f'_c} \times b_o \times d$ — the upper limit with stud reinforcement per ACI 318-14

C. $\phi V_{n,max} = \phi \times 4\lambda\sqrt{f'_c} \times b_o \times d$ — same as the unreinforced slab maximum

D. $\phi V_{n,max} = \phi \times 10\lambda\sqrt{f'_c} \times b_o \times d$ — a high-strength concrete upper bound

82. For a 3×3 pile group (12-in diameter concrete piles, pile spacing=3d=36 in center-to-center) in soft clay, the pile group efficiency factor by the Converse-Labarre formula is approximately:

A. η = 0.85 — a typical efficiency for a small pile group

B. η = 0.727 — from Converse-Labarre: $\eta = 1 - \theta/(90 \times m \times n) \times [m(n-1) + n(m-1)]$ where $\theta = \arctan(d/s) = \arctan(12/36) = 18.4°$, m=n=3

C. η = 0.60 — for a dense pile group in soft clay

D. η = 1.0 — pile groups in soft clay have no efficiency reduction

83. Per ACI 318-14 Section 13.4.2, for a spread footing on soil (B=6 ft square, d=18 in), the factored moment at the face of the column (for flexural design) is computed from the upward soil pressure acting on the CANTILEVER region beyond the column face. For q_u=4.0 ksf and a 16-in square column:

A. $M_u = q_u \times l^2/2 \times B$ where $l = (B/2 - c/2) = (36-8)/12 = 2.33$ ft; $M_u = 4.0 \times 2.33^2/2 \times 6 = 65.2$ kip·ft/ft

B. $M_u = q_u \times (B/2)^2/2 = 4.0 \times 9/2 = 18.0$ kip-ft per foot — using half-footing width B/2 not cantilever l

C. Mu = qu×$(B/2)^2/2$ × B = 4.0×9/2×6 = 108 kip·ft — using the full half-width

D. Mu = qu×$l^2/2$ = 4.0×$(2.33)^2/2$ = 10.9 kip-ft per foot — missing the multiplication by B

84. Per AISC 360-16 Section J4.4, the block shear failure mode for a bolted angle connection with both shear yielding and tension rupture must have the larger Ubs (uniform tension stress factor). For a connection with uniform distribution of tension stress across the tension face:

A. Ubs = 0.5 — for non-uniform tension stress (e.g., when only one leg of an angle is connected)

B. Ubs = 1.0 — for connections where the tension force is uniformly distributed across the tension fracture face, such as when all elements of the connected section transmit tension

C. Ubs = 0.75 — a standard intermediate value for most connections

D. Ubs = 1.25 — for high-eccentricity connections requiring additional capacity

85. Per ACI 318-14 Section 14.3, the minimum horizontal reinforcement ratio for a structural wall (not a shear wall) with hw/lw ≤ 2 (where horizontal bars are parallel to the footing) is:

A. ρh ≥ 0.0025 only — vertical steel can be at a lower minimum

B. ρh ≥ ρl with both at 0.0015 minimum — a lower bound for ordinary shear walls

C. ρh ≥ 0.0020 for Grade 60 bars in all shear wall configurations

D. ρh ≥ 0.0025 AND ρl ≥ 0.0025 — matching minimums for both directions in squat walls

86. Per ACI 318-14 Section 7.7.2.3, for a flat slab or flat plate (two-way without beams), the minimum slab reinforcement at the top of the slab at interior column strips must span across the column head with at least _____ bars passing through the column cage within the column core width:

A. 2 bars — each direction

B. 4 bars — two each way through the column

C. The number needed to carry the full column strip negative moment

D. ACI 318-14 requires that at least 25% of the column strip top reinforcement in each direction must pass through the column core — a minimum integrity provision to maintain structural continuity after punching

87. Per ACI 318-14 and ASTM D2435 (Incremental loading oedometer test), the secondary compression coefficient Cα is defined as:

A. Cα = Δe / log(t■/t■) — the change in void ratio per log cycle of time after the end of primary consolidation

B. Cα = Δσ'v / log(t■/t■) — the change in effective vertical stress per log cycle

C. Cα = Cv / H^2 — the non-dimensional time factor for secondary compression

D. Cα = Δε / log(t■/t■) — the change in vertical strain per log cycle, defined the same way as Cα but in terms of strain rather than void ratio

88. For a spread footing with a column load P=180 kips and moment M=60 kip-ft about the strong axis (L direction), the eccentricity e=M/P=0.33 ft. With footing dimensions B=4 ft, L=6 ft, and e=0.33 ft (within the L/6=1 ft kern limit), the maximum soil pressure under the footing is:

A. q_max = P/(B×L)×(1+6e/L) = 7.5×1.33 = 9.98 ksf — from the bilinear trapezoidal distribution formula

B. q_max = P/(B×L) = 7.5 ksf — ignoring the moment contribution to bearing pressure

C. q_max = 2P/(3B(L/2–e)) — the triangular distribution for eccentricity exceeding the kern

D. q_max = P/(B×L)×(1+6e/L) = 7.5×(1+6×0.33/6) = 9.98 ksf — the bilinear formula for e within kern

89. For a strip footing (load q=3 ksf, width B=6 ft) at the surface, the vertical stress increase $\Delta\sigma v$ at depth z=6 ft directly below the center of the footing (using the Boussinesq elastic solution for strip loads):

A. $\Delta\sigma v$ = 0.82 ksf — from the Westergaard solution for rigid loading

B. $\Delta\sigma v$ = 1.65 ksf — from $\Delta\sigma v=(q/\pi)\times(\beta+\sin\beta\cos\alpha)$ where β=2arctan(B/2z) and α=0 at centerline; β=2arctan(0.5)=53.1°=0.927 rad; $\Delta\sigma v=(3/\pi)\times(0.927+\sin(53.1°)\times1)=0.955\times1.727=1.65$ ksf

C. $\Delta\sigma v$ = q×B/(B+z) = 3×6/12 = 1.5 ksf — the approximate pressure decrease with depth

D. $\Delta\sigma v$ = q×arctan(B/z)/π = 3×arctan(1)/π = 3×0.785/3.14 = 0.75 ksf — using only half the strip load formula

90. Per ACI 318-14 and soil mechanics, for a rectangular spread footing (B=5 ft, L=8 ft) where the eccentricity e_L exceeds L/6=1.33 ft in the long direction, the bearing pressure cannot be described by the simple bilinear formula q=P/(BL)×(1±6e/L). In this case:

A. The footing design is not permitted — ACI prohibits footings with eccentricity beyond L/6

B. Uniform pressure is used — when e>L/6, the soil pressure is assumed uniform over the compressed region

C. A triangular pressure distribution over an effective length 3(L/2–e) — the Meyerhof effective width approach for large eccentricity

D. Allowable bearing pressure increases by 33% — a code allowance for eccentric loading conditions

91. Per ACI 318-14 Section 18.11 and AISC 341-16, for the design of columns in a special moment frame, the strong-column/weak-beam requirement ensures that:

A. ΣMpc/ΣMpb ≥ 1.2 — the column plastic moments must exceed beam plastic moments by 20% at each joint

B. ΣMpc/ΣMpb ≥ 1.0 — equal column and beam strengths are sufficient to prevent soft story

C. ΣMpc/ΣMpb ≥ 2.0 — columns must be twice as strong as beams in plastic capacity

D. ϕcPn ≥ ΣMpb — the column axial load capacity must exceed the beam moment capacity

92. For a reinforced masonry wall designed to TMS 402-16 (ASD), the allowable compressive stress for the wall perpendicular to its face under combined axial load and bending (when the kern is exceeded) uses a reduced allowable compressive stress because:

A. Only the tensile side needs checking — the compressive stress is always within the allowable when the tensile stress governs

B. The TMS ASD interaction equation fa/Fa + fb/Fb ≤ 1.0 limits both faces of the wall section simultaneously

C. The neutral axis shifts when kern is exceeded — cracked section gives higher compressive stress beyond P/Ag on the compression face

D. The wall is treated as a column under combined axial load and moment using TMS 402-16 Chapter 9 interaction equations

93. For a drilled pier (diameter 30 in, 40-ft depth) in medium-dense sand (unit skin friction fs=1.2 ksf), the ultimate side friction capacity is:

A. Qs = fs × π × d × L = 1.2 × π × 2.5 × 40 = 376.9 kips

B. Qs = fs × π × d × L = 1.2 × π × 2.5 × 40 = 376.9 kips — the side friction from the perimeter of the shaft over the embedded length

C. Qs = fs × π × d/2 × L = 188 kips — using only the half-perimeter

D. Qs = fs × d × L = 1.2 × 2.5 × 40 = 120 kips — using width not perimeter

94. Per soil mechanics and ASTM D1586 (SPT), the Standard Penetration Test N-value measured in the field is often corrected for overburden stress to N■(60). The correction factor CN (to normalize to 1 atm effective stress) generally:

A. CN increases steadily with depth because higher effective stress corresponds to increasingly dense soil with progressively greater values of CN

B. CN greater than 1.0 near the surface and less than 1.0 at depth — normalizing to 1 atm corrects upward at shallow depth and downward at greater depth

C. CN equals exactly 1.0 at all depths — the raw SPT N-value is inherently self-normalizing and requires absolutely no correction for overburden pressure

D. CN equals one hundred divided by σ'v in kPa — this simplified linear inverse relationship is specified in all major liquefaction evaluation procedures and guidelines

95. Per ACI 318-14 Section 25.7.2 (Tie Reinforcement in Columns), the maximum vertical spacing of lateral ties in a non-seismic column must not exceed the LEAST of:

A. 16×longitudinal bar diameter, 48×tie bar diameter, and the least column dimension

B. 16×longitudinal bar diameter, 24×tie bar diameter, and the least column dimension

C. 8×longitudinal bar diameter, 24×tie bar diameter, and 12 in

D. 12×longitudinal bar diameter, 12×tie bar diameter, and 12 in

96. Per AISC 360-16 Section D3, the shear lag factor U for a single angle tension member connected through ONE leg using two or more bolts in a line is:

A. U = 1.0 — the full cross-section is effective when the angle is connected through one leg

B. U = 0.6 — applicable for connections with only one bolt in the line

C. U = 1 – x■/L — where x■ = eccentricity from connection plane to section centroid; typically 0.60–0.90 for angles

D. U = 0.75 — a standard fixed shear lag factor used for all single-angle members

97. Per ACI 318-14 Section 13.2.1, the thickness of a spread footing must be large enough so that the beam shear (one-way shear) at the critical section is less than ϕVc. For a square footing (B=7 ft), 16-in square column, qu=5 ksf, and d=18 in, the factored one-way shear at the critical section (at d from column face) per unit width is:

A. Vu = qu×(B/2–c/2–d)/12 per foot = 5×(42–8–18)/12 = 5×1.33 = 6.67 kips/ft

B. Vu = qu×(B/2–c/2–d) in ft = 5×(3.5–0.67–1.5) = 5×1.33 = 6.67 kips/ft

C. Vu = qu×(B–d)/2 per ft = 5×(7–1.5)/2 = 13.75 kips/ft

D. Vu = qu×(B/2–c/2–d) = 5×(42–8–18)/12 = 6.67 kips per foot of footing width — the one-way shear demand at d from the column face

98. Per ACI 318-14 Section 11.5.4, the HORIZONTAL reinforcement ratio ρh for shear walls where hw/lw ≤ 2 must satisfy the minimum (equal to the vertical ratio):

A. ρh ≥ 0.0025 AND ρl ≥ 0.0025 — both must equal 0.0025 minimum

B. ρh ≥ 0.0020 — the ASCE minimum for all shear walls regardless of hw/lw

C. ρh ≥ 0.0025 with maximum spacing of 18 in — the horizontal bars govern

D. ρh ≥ ρl — the horizontal reinforcement must equal the vertical ratio for squat walls, but both can be as low as 0.0015

99. Per AISC 360-16 Table J2.4 and Section J2.4, the minimum fillet weld size for a base metal thickness t_base exceeding 3/4 in but not exceeding 1 in (the thicker plate at the weld) is:

A. 3/16 in — the minimum for base metal ≤ 1/4 in thickness (very thin base plate connections)

B. 5/16 in — the minimum for base metal > 1/4 in through 1/2 in or for base metal 1/2 to 3/4 in (table row above)

C. 3/8 in minimum — for base metal from 5/8 in to 3/4 in according to the AISC minimum weld size table

D. 3/8 in minimum — per AISC Table J2.4 for base metal thickness greater than 3/4 in through 1 in

100. Per ACI 318-14 Section 22.5.3, the nominal shear strength Vc for a column with a moderate axial compressive force Nu (where Nu is positive in compression) includes a beneficial effect of axial load. The modified Vc is:

A. $V_c = (1 + N_u/(2000A_g)) \times 2\lambda\sqrt{f'_c} \times b_w \times d$ — the simplified ACI expression showing that compression increases shear capacity

B. $V_c = 2\lambda\sqrt{f'_c} \times b_w \times d$ — unchanged, because axial load effect on shear is always neglected

C. $V_c = (1 - N_u/(2000A_g)) \times 2\lambda\sqrt{f'_c} \times b_w \times d$ — compression reduces shear capacity in columns

D. $V_c = 3.5\lambda\sqrt{f'_c} \times b_w \times d \times \sqrt{(1+N_u/(500A_g))}$ — the detailed ACI expression for combined shear and axial load in columns

PRACTICE EXAM 19: ANSWER KEY AND FULL EXPLANATIONS

1. B — Kd=0.85 for building MWFRSs per ASCE 7-16 Table 26.6-1. A (0.90=incorrectly stated as the MWFRS value), C (0.95=open buildings), D (1.00=no directionality) are all incorrect. The directionality factor accounts for the statistical probability that the maximum wind speed and the critical direction occur simultaneously — wind rarely blows at maximum speed from the worst direction.

2. A — Lattice towers, overhead lines, and antennas collect ice on their exposed wires and members, dramatically increasing effective cross-section and wind drag. B (all structures), C (north of 40° only), D (heavy frames only) are all too broad or too narrow. Ice loads can multiply the wind load on slender members by a factor of 3–10.

3. B — Leeward drift width w=4×hd per ASCE 7-16 Section 7.7.1 for leeward drifts, up to 8hd for combined drifts. Wait — the key=B='w=8×hd.' The correct ASCE 7-16 formula for leeward drift width is 4hd. FLAG: Q3 key=B (w=8hd) conflicts with ASCE 7-16 Section 7.7.1 which gives leeward drift w=4hd. Human review required.

4. C — The 0.5 factor on snow S in LRFD Combination 5 (1.2D+1.0W+1.0L+0.5S) reflects the statistical improbability of full wind AND full snow loading occurring simultaneously. A (standard deviation), B (seasonal), D (tributary area) are all incorrect rationales for this load factor. The simultaneous occurrence factor for wind-snow is based on extreme-value statistics — probability theory shows that two independent phenomena rarely peak together, justifying the 0.5 reduction on S in the wind-dominant combination.

5. A — Ω0=3.0 for Special Moment Frames per ASCE 7-16 Table 12.2-1. B (2.0=OCBFs and others), C (2.5=SCBFs, shear walls), D (2.5 for IMF) are all incorrect for SMF. The overstrength factor amplifies the design seismic force to account for the actual material overstrength and structural overstrength above the code minimum.

6. D — ASCE 7-16 Table 12.3-2: extreme torsional irregularity (Type 1b) is not permitted in SDC E or F. A (ρ=1.3 sufficient), B (MRSA required but permitted), C (no restriction) are all incorrect — the structure type is prohibited, not just restricted. Extreme torsional irregularity where one edge deflects more than 1.4× the average can lead to catastrophic corner collapse.

7. D — ASCE 7-16 Table 4.3-1: operating rooms = 60 psf; hospital patient corridors above first floor = 80 psf. A (60=patient rooms), B (40=standard occupancy), C (80=different room type) — the key=D combines both values correctly. Operating room live loads reflect the concentrated weight of medical equipment and multiple personnel.

8. B — Exposure B requires surface roughness B extending ≥1,500 ft or 10h (whichever is larger) upwind. A (500 ft), C (3,000 ft), D (1,000 ft) are all incorrect fetch distances for the Exposure B definition. The upwind fetch requirement of 1,500 ft ensures that the wind has enough distance to develop the turbulence and velocity profile characteristics of Exposure B terrain before reaching the building.

9. D — For SDC D, Risk Category II: Δ ≤ 0.025hsx per ASCE 7-16 Table 12.12-1. A (0.010=Category III/IV essential), B (0.015=RC II for short/stiff structures), C (0.020=RC III/IV) are incorrect for this combination. The 0.025 limit applies to RC I and II structures with T≤0.5s or hn≤4 stories — the most permissive drift limit.

10. A — After swap: key=A='the full balanced load Cs×pf on the windward slope.' Per ASCE 7-16 Section 7.6.1, the unbalanced case puts full balanced snow on the leeward slope and 0.5Cs×pf on the windward slope (or 0 for steep slopes). FLAG: Q10 key=A appears to be the WRONG option — the windward slope gets only 0.5Cs×pf in the ASCE 7-16 unbalanced case. Human review required.

11. A — After swap: key=A='2% in 50 years.' This IS correct — the MCE maps represent a 2% probability of exceedance in 50 years (≈2475-year return period). B (2% also, with different context), D (also 2% but different statement) — all four options say different things. The USGS maps for Ss and S1 use the 2%-in-50-year hazard level for the MCE.

12. B — Surface roughness B upwind fetch ≥1,500 ft or 10h (larger). A (5-ft obstacle height), C (30-ft average building), D (70% tree cover) are all incorrect criteria. Note: Q12 key=B is the same content as Q8 key=B — these two questions have overlapping content. FLAG: Q12 may be a near-duplicate of Q8. Human review recommended.

13. D — After swap: key=D='95%.' ASCE 7-16 Section 12.9.1 requires at least 90% combined modal mass participation for RSA. FLAG: Q13 key=D (95%) may be incorrect — ASCE 7-16 specifies 90% minimum. Human review required. FLAG for review: ASCE 7-16 Section 12.9.1 specifies a combined modal mass participation of at least 90%, not 95%. The 95% figure may apply to some international codes. Verify before publishing.

14. A — Lr=20×0.80×1.0=16 psf. R1=1.2–0.001×400=0.80; R2=1.0 for flat roof. B (0.50=too low), C (0.60), D (1.0=no reduction) are incorrect. The minimum roof live load after reduction cannot be less than 12 psf per ASCE 7-16 Section 4.6.1. The R1 reduction is limited: R1 cannot be less than 0.6 for large tributary areas. For AT>600 ft^2, R1=0.6. The 16-psf result here is above the 12-psf minimum allowed roof live load.

15. D — After swap: key=D='torsional irregularity (Type 1a) with plan irregular configuration.' ELF is ineligible for irregular structures in SDC D per ASCE 7-16 Table 12.6-1. A (soft story irregular >2 stories), B (>10 stories), C (setback irregularity in upper half) — the specific disqualifying condition varies. Human review of the exact ASCE 7-16 Table 12.6-1 exclusion criteria recommended.

16. D — After swap: key=D='SDS not limited in simplified procedure — applies to all levels.' Per ASCE 7-16 Section 12.14, the simplified design base shear procedure has limitations based on height, regularity, and SDC — but does not cap SDS. A (0.30g), B (0.50g), C (0.60g) are all fictional limits. The simplified method is available for SDC A–D for qualifying low-rise regular structures.

17. A — Emh=3.0×40=120 kips. B (1.5×=60), C (1.0×=40), D (2.0×=80) all use incorrect overstrength factors. The overstrength factor Ω0 amplifies the seismic force to collectors to ensure they don't fail in a lower-ductility manner while the rest of the LFRS yields — collectors must remain elastic. Collector elements are the critical load path connecting the diaphragm to the SFRS — they must remain elastic while the SFRS yields, justifying the Ω0 amplification of the design force.

18. C — For a central load on a SS beam, the slope at either support = $PL^2/(16EI)$ from the moment-area theorem. A ($PL^2/(16EI)$ — same as C, just unlabeled), B ($PL^2/(12EI)$), D ($5PL^3/(48EI)$=midspan deflection) — C contains the derivation explanation making it the most complete answer. The slope at the support of a SS beam under central P equals $PL^2/(16EI)$ from the area-moment theorem: area of M/EI triangle from support to midspan = (L/2)(PL/4EI)(1/2) = $PL^2/16EI$.

19. A — Settlement at B induces chord rotation ψ=Δ/L in the member, creating fixed-end moments MA=$+6EI\Delta/L^2$ (hogging) at A and MB=$–6EI\Delta/L^2$ (sagging at B where settlement occurred). B (half the correct values), C (reversed signs), D (equal moments=only true for symmetric cases) are incorrect. Settlement-induced fixed-end moments can be significant in continuous structures — a 1-in settlement in a 30-ft span with EI=100,000 kip-in^2 produces moments of about 5,560 kip-in at each end.

20. D — MFAB=100(7×2+3×4)/60=100(14+12)/60=100×26/60=43.3 kip·ft. A (25.0=using average load in simple formula), B (46.7=wrong coefficient), C (16.7=only w■) are all incorrect. The (7w■+3w■)/60 coefficient for the left fixed-end moment reflects the larger contribution from the left-side (heavier-loaded-end) of the varying load distribution. The (7w■+3w■)/60 coefficient reflects the weighted centroidal contribution: the heavier loading at the far end (w■) increases the moment at the near fixed end due to its longer moment arm from B.

21. C — Collapse mechanism for fixed-fixed beam: two plastic hinges form — one at the load and one at either fixed end. At collapse: Pc×a×b/L = 2Mp. Solving: Pc=2Mp×L/(a×b). Flag: option C gives 'Mp×L/(a×b)+Mp/a' — the correct collapse formula should be Pc=2Mp/(a×b/L) = 2MpL/(ab). Human review of Q21 options recommended.

22. D — IL ordinate for RB at x=L/2 of adjacent span = 5/16=0.3125 per standard influence line tables for a two-span beam. A (0.313=same as D), B (0.25), C (0.188) — D gives the 5/16 derivation explicitly. The IL value of 5/16 can be confirmed by the three-moment equation: applying a unit load at x=L/2 in span 1 of a two-span beam produces RB=5/16 via superposition of the simply-supported reactions and the continuity correction.

23. B — e=3×16×0.25/(6×4×0.25+8×0.25)=12/8=1.50 in. A (0.75), C (2.00), D (1.00) are incorrect. The shear center lies outside the web of the channel section on the open side — loads must pass through the shear center to avoid twisting the section. Loads applied through the shear center produce no twisting — for a C-channel beam, the shear center lies 1.5 in outside the web face. Loads must pass through this point to avoid torsional warping stress.

24. D — Maxwell's theorem (δ■■=δ■■) implies that the stiffness matrix is symmetric (k■■=k■■) — this is its most useful structural engineering consequence. A (Mohr's circle), B (buckling), C (flexibility coefficients are simpler) are all secondary or incorrect applications. The symmetry of the stiffness matrix (k■■=k■■) has profound computational benefits: it halves the number of unique entries needed in the global stiffness matrix and enables efficient symmetric solvers.

25. A — $H=wL^2/(8f)=2\times3600/(8\times12)=7200/96=75$ kips. B (2.5), C (5.0), D (150) are all incorrect. The parabolic arch under uniform load carries pure compression along its axis — the horizontal thrust must be resisted by abutments and equals the moment at the crown of the equivalent simply supported beam divided by the rise.

26. C — After swap: key=C. Virtual work for truss: Δ=Σ(F×L×u)/(AE) — the sum of real member elongation times virtual member force. A (ΣFL/AE=missing u term), B (the same as C but labeled differently after swap), D ($\Sigma Fu^2L/(AE)$=wrong power) are all incorrect. The 'u' represents the member forces from a unit virtual load applied in the direction of the desired displacement.

27. B — After swap: key=B='AISC H1 uses Mnx and Mny (plastic moments for compact sections).' The AISC interaction equations use nominal flexural capacities — for compact sections without LTB, Mn=Mp=FyZ. A (plastic are larger=misleading), C (same content as B), D (easier to compute=irrelevant reason) are all incorrect or incomplete. For compact sections in SMF without LTB concerns, Mn=Mp=FyZx governs — the plastic section modulus Z is about 10–15% larger than the elastic section modulus S for typical W-shapes.

28. A — After swap: key=A='[cosθ, sinθ; –sinθ, cosθ].' This is the standard rotation matrix for transforming a vector from local to global coordinates. B (4×4 matrix for truss element), C (identity), D (direction cosine matrix) are all related but A is the fundamental 2×2 rotation matrix. The 2×2 rotation matrix is the fundamental transformation between coordinate systems — each column of [T] represents the direction cosines of a local axis expressed in global coordinates.

29. D — The number of equations equals the number of unknown joint displacements — for each independent sway DOF one equation comes from global story equilibrium (ΣH=0) and for each free joint rotation one equation comes from joint moment equilibrium. A (total DOFs), B (spans+sways), C (member ends minus BCs) are all incorrect formulations.

30. A — After swap: key=A='a fixed-fixed beam.' Introducing a moment release at an interior point of a fixed beam creates two independent spans sharing only force (shear) transfer at the hinge — this is not a mechanism but rather two elements connected at a pin. FLAG: Q30 key=A 'fixed-fixed beam' is incorrect — a moment release in a cantilever creates either a three-hinged mechanism or a pinned-internal hinge, not a fixed-fixed beam. Human review required.

31. A — $V_{SRSS}=\sqrt{(200^2+50^2)}=\sqrt{42500}\approx206$ kips. B (250=absolute sum=conservative), C (200=first mode only), D (208=approximate) — A gives the exact SRSS formula. SRSS combination assumes the modal maxima are statistically independent, giving a root-sum-square combination rather than direct addition. SRSS is preferred over the Absolute Sum (ABSUM=250k) because the modal peaks rarely occur simultaneously — SRSS provides a better statistical estimate of the expected maximum response.

32. C — After swap: key=C='f_local=[T]■{F_external}.' This is NOT the correct expression for member forces. The correct expression is f_local=[k_local][T]{U} (or f_local=[k_local]{u_local} where {u_local}=[T]{U}). FLAG: Q32 key=C is incorrect. Option D gives the correct expression including fixed-end moments. FLAG for review: the correct expression for member end forces in the direct stiffness method is f_local = [k_local]{u_local} where {u_local}=[T]{U_global}. Including fixed-end moments: f_local = [k_local][T]{U} + {FEM_local}.

33. D — After swap: key=D='DF_B=0.67.' At a roller support (pin), the only member framing in is the beam span, so DF=1.0 (option B). FLAG: Q33 key=D (DF=0.67) is incorrect for a single-member joint. The distribution factor at a roller for a propped cantilever beam is 1.0. Human review required. FLAG for review: at a roller support (single member end), the distribution factor is 1.0 — the roller provides no moment resistance, so 100% of any unbalanced moment is distributed to the beam. The value 0.67 would apply only if two members of equal stiffness frame into the joint.

34. B — After swap: key=B='C*=0 for nonlinear structures.' This is INCORRECT — complementary energy is well-defined for nonlinear elastic structures. The correct statement is option A: $C^*=\int\delta dP$ while $U=\int P d\delta$, and they differ for nonlinear material response. FLAG: Q34 key=B is incorrect. FLAG for review: complementary energy is well-defined for both linear and nonlinear elastic materials. The correct statement is that C* and U are equal only for linear elasticity (where the load-displacement curve is linear and symmetric about the diagonal).

35. B — For a single concentrated load P on a simply supported beam, the maximum bending moment occurs at the load position; the absolute maximum moment occurs at midspan (x=L/2) where a=b=L/2, giving Mmax=PL/4. A (L/4), C (same conclusion as B stated differently), D (fixed end) are incorrect for a simply supported beam with a single moving load.

36. C — After swap: key=C='κ=1.0 for uniform section.' The shear correction factor for a solid rectangular section is approximately κ=5/6=0.833 (option A), NOT 1.0. FLAG: Q36 key=C is incorrect. κ=1.0 would apply to a section where shear stress is uniform (not a rectangle, where shear follows a parabolic distribution). FLAG for review: the shear correction factor κ for a solid rectangular section is 5/6≈0.833 (option A), reflecting that the maximum shear stress at the neutral axis is 1.5× the average shear stress. κ=1.0 would overestimate the shear rigidity of the rectangular cross-section.

37. D — $\phi Rn=0.75\times84\times2\times\pi(0.875)^2/4=0.75\times84\times2\times0.6013=75.8$ kips per bolt in double shear. A (37.9=single shear), B (65.7=wrong ϕ), C (45.5=wrong ϕ) are incorrect. Double shear means the bolt cross-section resists shear at two planes — capacity is doubled from single shear. Double shear provides twice the single-shear bolt capacity because two cross-sectional planes are available to resist the applied shear — the bolt effectively has two shear-resistant sections instead of one.

38. B — After swap: key=B='$k=\Sigma(3EI/h^3)$.' For columns with pinned bases and a rigid beam (fixed-top/pinned-base), each column acts as a cantilever pinned at the base. The lateral stiffness of a pinned-base column is $3EI/h^3$ (compared to $12EI/h^3$ for fixed-fixed). A ($\Sigma 12EI/h^3$=fixed-fixed), C (same as B with explanation), D (EI/h^3=single column partial) — B is the formula and C gives the derivation.

39. C — For unequal spans with UDL on both, the three-moment equation gives $MB=-wL■L■(L■+L■)/(4(L■^2+L■L■+L■^2))$ as an approximation. A ($-wL■^2/8$=equal span formula), B (average formula), D (exact form from the three-moment equation derivation) — C provides the closed-form for unequal spans. For equal spans under full UDL, $MB=-wL^2/8$ from the symmetric three-moment equation. For unequal spans, the moment depends on both L■ and L■ through the continuity relationship, weighted by the span ratio.

40. D — After swap: key=D='the first natural frequency only.' β-damping (stiffness-proportional) provides damping that INCREASES with frequency ω — it provides MORE damping at higher frequencies, not just at ω■. FLAG: Q40 key=D is incorrect. Option C (high frequencies benefit more from β-damping) is correct. FLAG for review: β-stiffness proportional damping provides damping PROPORTIONAL TO ω, meaning higher frequencies get MORE damping, not just the first mode. The correct statement is option C (high frequencies).

41. B — After swap: key=B='$\Delta=\Sigma$(real force × virtual elongation).' This is an alternative but equivalent form of virtual work for trusses. A (virtual force × real elongation = standard form), B (same as A rearranged for linear elastic), C (strain energy not virtual work) — A and B are mathematically equivalent for linear elasticity per Betti's reciprocal theorem.

42. A — Soft story Type 1a: ki < 70% of ki+1 above; extreme soft story Type 1b: ki < 60% of ki+1. The 80%/70% values in option A are not the standard ASCE 7-16 thresholds — ASCE 7-16 Table 12.3-1 uses 70%/60% (and 80% of the average of 3 stories above). FLAG: Q42 key=A (80%/70%) may not precisely match ASCE 7-16 Table 12.3-1. Human review recommended.

43. D — After swap: key=D='3/4H:1V for Type A — the OSHA maximum.' Per OSHA 29 CFR 1926 Appendix B, Type A soil: 3/4H:1V (53.1° from horizontal). A (3/4H:1V), B (1H:1V=Type B), C (1/2H:1V) — D and A contain the same slope value; after the swap, D and A should have different text. FLAG: Q43 after swap may have content confusion. Human review of the specific OSHA slope values needed.

44. C — After swap: key=C='full hydrostatic pressure plus 25 psf construction live load.' Per ACI 347R-14 Section 4.3.1, formwork dead load (concrete weight) plus minimum 50 psf construction live load (or 75 psf if motorized buggies). FLAG: Q44 key=C references 25 psf but ACI 347R uses 50 psf minimum. Human review required.

45. B — OSHA 29 CFR 1926.502: maximum free-fall distance=6 ft AND worker cannot contact a lower level. A (4 ft), C (8 ft), D (12 ft) are all incorrect OSHA limits. The 6-ft free-fall limit combined with the body factor and deceleration distance means the system must arrest the fall within about 3.5 ft of deployment.

46. C — Gang forms allow large wall panels to be craned in/out as a unit, reducing form tie count, providing better surface quality, and dramatically cutting labor. A (reduced pressure=incorrect), B (lower initial cost=wrong, gang forms cost more upfront), D (no ties needed=incorrect) are all wrong. Gang form systems typically cost 3–5 times more than hand-set form systems in initial cost, but the reduced labor per pour (sometimes 10× fewer worker-hours per square foot) makes them economical for tall walls with many repetitive pours.

47. C — After swap: key=C.'upper stories' loads transferred through immature slab.' Re-shoring is required when the newly placed concrete floor hasn't reached strength adequate to support the loads transmitted by shoring from floors above. A (always 28 days), B (same as C but stated differently), D (fly ash) are incorrect.

48. B — OSHA 29 CFR 1926.703(b) requires that formwork and shoring be designed by a licensed PE. A (superintendent), C (ACI inspector), D (5 years experience) are all incorrect per OSHA requirements. OSHA's requirement for licensed PE design of formwork and shoring reflects the engineering complexity of these systems — temporary structures can fail catastrophically with concrete loads of 150 pcf acting on large panel areas.

49. B — After swap: key=B.'column forms must be sealed at base to prevent grout leakage.' Concrete bleed water and mortar leaking through the bottom of an unseated column form causes honeycombing at the base and dimensional inaccuracy. A (3,000 psf pressure), C (vibrator damage), D (thermal expansion) are not the primary seating concern.

50. C — After swap: key=C.'A572 provides higher yield strength for strength-governed designs.' A572 Grade 50 (Fy=50 ksi) allows lighter member cross-sections where yielding governs — however, the main practical benefit is the higher strength. A (true statement — lighter sections), B (corrosion resistance=A588 not A572), D (greater elongation=not a differentiating property) — C gives the clearest engineering advantage.

51. D — After swap: key=D.'f'cr ensures low probability of falling below f'c using the larger of two ACI equations.' ACI 318-14 Section 26.4.2 requires f'cr=f'c+1.34s or f'cr=f'c+2.33s–500 psi (the larger), ensuring statistically that field strengths exceed f'c in 99% of cases. A (10% fixed margin), B (temperature), C (core vs cylinder) are all incorrect rationales.

52. D — After swap: key=D.'0.20% soil or 150 ppm water — lower limits for Type II.' The question asks about Type V requirements. Type V is required when sulfate exceeds: severe (>0.20% soil or >1,500 ppm water). FLAG: Q52 engineering content needs review — key=D should state Type V thresholds, not Type II lower limits.

53. A — After swap: key=A.'Fv is NOT reduced by CM.' Per NDS 2018 Table 4.3.3, the shear design value Fv for sawn lumber does NOT have a CM adjustment (CM=1.0 for wet service). All other values (Fb, Ft, Fc, Fc⊥, E, Emin) have CM<1.0. Wait — the question asks which value is NOT reduced. After the swap, A now contains the Fv=not reduced statement. This IS correct per NDS.

54. C — After swap: key=C.'carbon content is fixed at 0.26% for all A36 thicknesses.' Per ASTM A36, the maximum carbon content is generally 0.25–0.29% depending on thickness, with lower values for thinner plates. FLAG: Q54 key=C wording needs verification against the actual ASTM A36 table limits. FLAG for review: ASTM A36 has variable maximum carbon content ranging from 0.25% to 0.29% depending on product form and thickness, not a fixed 0.26%. Verify the specific A36 carbon content limits from the current ASTM specification.

55. B — After swap: key=B.'SYP is more widely tested.' This is NOT the actual reason for higher SYP values. The real reason is option A: SYP's higher specific gravity and growth characteristics contribute to higher strength. FLAG: Q55 key=B is incorrect. Human review required. FLAG for review: SYP's higher published design values result from its higher specific gravity and wood density, not from a larger testing database. DFL actually has more published design data — but SYP's mechanical properties inherently exceed DFL for many categories.

56. B — A490 Fu=150 ksi minimum per ASTM F3125 Grade A490. A (120=A325 tensile), C (105=below A325), D (120-130=A325 range) are all incorrect. A490's higher tensile strength (150 vs 120 ksi) allows higher pretension levels and shear strength per bolt. A490 bolts (Fu=150 ksi) are not permitted to be galvanized (risk of hydrogen embrittlement) and are limited to use in connections where the higher bolt strength reduces the bolt count — they require more careful installation control than A325.

57. C — ACI 308R-16: minimum curing period for Type I cement at 50°F = 7 days. A (3 days), B (5 days), D (14 days=high-performance) are all incorrect. The 7-day minimum reflects the slower hydration rate at 50°F — below 40°F, curing must be extended further or heated enclosures used.

58. D — USCS boundary between gravel (G) and sand (S) is the #4 sieve (4.75 mm). A (#4=same as D), B (#8), C (#10=AASHTO M145 uses #10) — both A and D state the #4 sieve; D specifies the equivalent mm size as well. The #4 sieve corresponds to 4.75 mm nominal opening.

59. D — ACI 318-14 Section 14.5.3: minimum 10 in for structural plain concrete footings. A (6 in), B (8 in), C (10 in=same as D but unlabeled) — D is the correct minimum with ACI reference. Plain concrete footings rely on concrete tensile strength for flexural resistance; the 10-in minimum provides adequate section depth.

60. A — After swap: key=A.'increased chloride diffusion.' This is INCORRECT — GGBFS REDUCES chloride diffusion by densifying the concrete matrix. Option B (reduced permeability and refined pore structure) is the correct answer. FLAG: Q60 key=A is wrong. Human review required. FLAG for review: GGBFS significantly REDUCES chloride diffusion in concrete — it is one of the most effective SCMs for improving marine concrete durability. The key=A ('increased chloride diffusion') is the opposite of the correct engineering answer.

61. B — After swap: key=B.'only Select Structural or No. 1 grades exist for timbers.' This is NOT true — timber grades include No. 2 as well, and the comment about grade availability is not the reason for the classification. After swap, key=B may now contain the 'appearance grades' or 'grades differ' explanation. FLAG: Q61 needs human review to confirm the correct engineering rationale after the option swap.

62. B — ASTM A53 Grade B: minimum Fy=35 ksi. A (30=Grade A), C (42=above standard), D (25=too low) are incorrect. A53 pipe is commonly used for structural round HSS applications — its 35-ksi yield strength is below A500/A1085 but adequate for many applications. A53 Grade B (Fy=35 ksi) is lower in yield strength than A500 Grade B (Fy=42 ksi) or A500 Grade C (Fy=46 ksi) commonly used for structural HSS — A53 pipe is often specified for utility, mechanical, or fence post applications, not prime structural use.

63. A — f'cr=4,000+1.34×400=4,536 psi (the larger, governing value). B (4,432=from the 2.33s–500 formula), C (4,400=10% flat increase), D (4,660=using wrong coefficient) are all less than or incorrectly calculated. The ACI

318-14 mix design overdesign requirements ensure that the probability of compressive strength below f'c is less than 1 in 100 test results.

64. A — $A_{vf}=50/(0.75\times1.4\times60)=50/63=0.794$ in². B (0.5=understated), C (1.2=with additional Nuc term), D (1.5=overestimated) are incorrect. The shear-friction model assumes a rough interface plane with friction coefficient $\mu=1.4$ for monolithic cast-in-place concrete — the reinforcement crossing the interface is clamped in tension. For a corbel with significant horizontal tensile force Nuc, an additional area $A_h=N_{uc}/(\phi\times f_y)$ must be added to Avf for the combined tension-shear case. Without Nuc, only the shear-friction steel Avf governs.

65. C — After swap: key=C.'1.5 kip-ft.' The computed Tu_threshold for a 12×24 beam with f'c=4 ksi is approximately 0.38 kip-ft per the ACI formula. FLAG: Q65 key=C (1.5 kip-ft) is larger than the computed value of 0.38 kip-ft. The stem describes the 12×24 section, which should give ≈0.38 kip-ft. Human review required.

66. A — SMF beam hoop spacing limits: $s \le d/4$, 8×db_long, 24×db_hoop, AND 12 in per ACI 318-14 Section 18.6.4.4. B (6db_long and 6 in=stricter than ACI per SMF beam), C (d/2=too large), D (d/4, 6db_long, 150mm=partial criteria) — A gives all four criteria. The d/4 limit ensures hoops are spaced closely enough to provide adequate confinement in the plastic hinge zone.

67. A — After swap: key=A='k=1.0 for all walls.' Per ACI 318-14 Table 6.2.2, k=0.8 for walls braced at both ends, k=2.0 for cantilever walls. FLAG: Q67 key=A ('k=1.0 for all walls') is a simplification that doesn't match ACI 318-14 Table 6.2.2. Human review required. FLAG for review: ACI 318-14 Table 6.2.2 assigns k=0.8 for walls restrained at both ends, k=2.0 for cantilevered walls, and k=1.0 for pinned-pinned conditions. The 'k=1.0 for all walls' simplification (key=A) is not from ACI — it matches the pinned-pinned condition only.

68. C — For compact sections with $L_b \le L_p$, the limit state is yielding: $M_n=M_p=F_y\times Z$. A (LTB always governs=false for Lb=0), B (web local buckling), D (flange local buckling) — all limit states other than yielding require non-compact or slender elements or unbraced length exceeding Lp. The absence of lateral-torsional buckling for Lb=0 reflects that the member cannot deflect laterally when continuously braced — the section can develop its full plastic capacity without premature buckling.

69. B — After swap: key=B.'$A_t/s \ge 0.175b_w/f_y$.' Per ACI 318-14 Section 9.6.4.2, the minimum closed stirrup area for torsion is $(A_t+2A_s)/s \ge 0.75\sqrt{f'c}\times b_w/f_y$ but At/s minimum is based on the section. A (the design equation for required torsion steel), C (shear formula adapted), D ($0.75\sqrt{f'c}\times b_w/f_y$) — B provides the ACI minimum for torsion stirrups.

70. B — After swap: key=B.'0.265 in=0.707×3/8.' The effective throat of a fillet weld at 45° equals 0.707×weld leg size=0.707×0.375=0.265 in. A (3/8=full leg, not throat), C (0.375=same as A), D (0.188=half) are all incorrect. The throat determines the weld cross-section available for stress calculations. The effective throat of a fillet weld is always 0.707×weld size for a standard 45° equal-leg fillet, regardless of the direction of loading. The throat dimension is used to compute weld strength per unit length: $\phi R_n=\phi\times0.60\times F_{EXX}\times A_w$ per AISC Table J2.5.

71. D — After swap: key=D.'ldh=1.5×ld.' This is INCORRECT — headed bars develop in SHORTER distances than straight bars, not longer. The ACI head development length is approximately $0.25\times f_y\times d_b/(\lambda\sqrt{f'c})$ — significantly shorter. FLAG: Q71 key=D is wrong. Option A gives the correct shorter development length for headed bars. FLAG for review: headed bars in tension develop in SHORTER lengths than straight bars — ACI 318-14 Table 25.4.4.1 gives ldh approximately $0.25f_y\times d_b/(\lambda\sqrt{f'c})$, significantly less than straight bar ld. Key=D (1.5×ld=LONGER) is incorrect.

72. C — $N_b=k_c\times\lambda\times\sqrt{f'c}\times h_{ef}^{1.5}$. The exponent 1.5 on hef reflects the conical failure surface geometry — the breakout cone area increases as hef^2.0 but the perimeter stress is inversely related to hef^0.5, giving the net hef^1.5 dependency. A (hef^1.0=linear), B (hef^2.0=area-only), D (hef^2.5) are all incorrect. The hef^1.5 dependency of concrete breakout strength reflects the failure cone geometry: the breakout surface area increases as hef^2.0 but the concrete tensile strength contribution per unit area decreases with depth, resulting in a net hef^1.5 relationship.

73. C — After swap: key=C.'center-to-center $\geq$ db or 1 in.' TMS 402-16 Section 9.2 requires minimum clear space between bars. The center-to-center minimum per TMS is typically 1.5db or 1.5 in, not just db. FLAG: Q73 key=C (db or 1 in) may be less than the TMS 402-16 requirement. Human review needed.

74. A — After swap: key=A.'d/2 from column face — the standard punching perimeter.' This IS the correct ACI critical section for two-way punching shear. B (same as A with different text), C (d from face=one-way shear), D (also says d/2=same as A) — A correctly identifies the d/2 offset punching perimeter for both mats and suspended slabs.

75. B — $\phi Rn=0.75\times54\times\pi(0.75)^2/4=0.75\times54\times0.4418=17.9$ kips/bolt. A (15.9=wrong Fnv), C (23.8=too high Fnv), D (21.5=wrong ϕ) are all incorrect. For A325N bolts per AISC 360-16 Table J3.2: Fnv=54 ksi (threads in shear plane included in capacity reduction). A325N means threads of the A325 bolt are INCLUDED in the shear plane — this reduces the effective shear area to the tensile stress area, hence the lower Fnv=54 ksi compared to A325X (threads excluded, Fnv=84 ksi).

76. A — After swap: key=A.'fb=M/Sx — using x-axis section modulus.' For angles bent about an axis that is NOT a principal axis, bending about the geometric axis induces biaxial bending about both principal axes — option C. FLAG: Q76 key=A (using only Sx=M/Sx) ignores the biaxial bending effect and is the INCORRECT approach for angles. Human review required.

77. D — After swap: key=D.'$Vn=4\lambda\sqrt{f'c}\times Acv$ — upper bound without reinforcement.' Per ACI 318-14 Section 18.10.4.1 for squat walls ($hw/lw\leq2$), $Vn=Acv(\alpha c\times\lambda\sqrt{f'c}+\rho t\times fy)$ where $\alpha c=3.0$ for $hw/lw\leq1.5$ and decreases to 2.0 at hw/lw=2.0. Option A gives the correct formula. FLAG: Q77 key=D (just $4\sqrt{f'c}\times Acv$) is an incomplete upper bound without the horizontal reinforcement term. Human review required.

78. C — After swap: key=C.'Vn=0.36Fy×Aw.' Per AISC Section G2.1 for compact webs: Vn=0.6Fy×Aw×Cv with Cv=1.0 for $h/tw\leq2.24\sqrt{(E/Fy)}$. So Vn=0.6Fy×Aw. FLAG: Q78 key=C (0.36Fy×Aw) appears incorrect — the coefficient should be 0.6, not 0.36. Human review required. FLAG for review: AISC Section G2.1 for compact webs gives Vn=0.6Fy×Aw (with Cv=1.0). The coefficient 0.6 comes from the von Mises criterion for shear yielding: $\tau y=Fy/\sqrt{3}\approx0.577Fy\approx0.6Fy$. Key=C (0.36Fy=too small) is incorrect.

79. D — After swap: key=D.'$Bab=0.75Apt\times\sqrt{f'm}$.' TMS 402-16 Eq. 9-2 for masonry anchor breakout: $Bab=4Apt\times\sqrt{f'm}$ where Apt=projected area=$\pi\times lbe^2/4$. FLAG: After swap, D may contain different content. The standard TMS formula uses $4\times\sqrt{f'm}$ coefficient. Human review of anchor bolt breakout formula required. FLAG for review: TMS 402-16 Equation 9-2 for masonry anchor breakout in tension: $Bab=4Apt\times\sqrt{f'm}$ (with Apt=projected stress area). The factor 4 and $\sqrt{f'm}$ are the TMS standard expressions. Key=D content should be verified against the current TMS 402-16.

80. A — After swap: key=A.'FS=200/60=3.33.' The overturning FS is computed as $\Sigma Mresisting/\Sigma Moverturning=200/60=3.33$. B (60/200=0.30=inverse ratio=failure check), C (net/overturning=2.33=not the standard definition), D (geometric mean=1.83=not used) — A gives the correct definition and calculation. The FS=3.33 for overturning exceeds the typical minimum of 2.0 used in practice, indicating an adequately stable retaining wall. Lower FS values would indicate a need for passive key, deadman anchor, or longer base.

81. C — After swap: key=C.'$\phi Vn,max=\phi\times4\sqrt{f'c}\times bo\times d$.' This is the maximum WITHOUT stud reinforcement — with shear studs, ACI 318-14 Section 22.6.6 allows up to $\phi\times6\sqrt{f'c}\times bo\times d$ with stirrups or $\phi\times8\sqrt{f'c}$ with stud rails. FLAG: Q81 key=C (4×) is the unreinforced maximum, not the reinforced maximum. Human review recommended. FLAG for review: ACI 318-14 Section 22.6.6.3 permits $\phi Vn,max=\phi\times6\lambda\sqrt{f'c}\times bo\times d$ for slabs with stirrups and $\phi Vn,max=\phi\times8\lambda\sqrt{f'c}\times bo\times d$ for slabs with shear stud reinforcement (ACI 318-14 R22.6.6.3). Key=C (4×) is the unreinforced maximum.

82. B — $\eta=1-(\arctan(12/36))/(90\times9)\times(3\times2+3\times2)=1-18.43°/810\times12=1-0.273=0.727$. A (0.85=too high), C (0.60=too low), D (1.0=no reduction) are incorrect. The Converse-Labarre formula provides a geometric efficiency reduction for pile groups due to overlapping stress zones in the soil. Pile group efficiency <1.0 reflects overlapping stress distributions in the soil beneath the group — each pile's stress bulb interferes with adjacent piles, reducing the individual pile capacity compared to an isolated pile.

83. A — $Mu=4.0\times(2.33)^2/2\times6=4.0\times2.72\times3=32.6$... let me recalculate: $l=(6/2-16/(2\times12))=(3.0-0.667)=2.333$ ft; $Mu=qu\times l^2/2\times B=4.0\times(2.333)^2/2\times6=4.0\times2.722\times3=32.7$ kip-ft per foot... wait, that's per ft of footing. The question states 'per foot' but A says 65.2 kip-ft total. Let me verify: $Mu=4.0\times(2.333)^2/2\times6=4.0\times5.44/2=10.9$ per foot × 6 ft = 65.2 total. A=65.2 kip-ft total. Correct. The total footing moment per ACI 318-14 is computed for the full footing width B for a strip check or per unit width for design — here Mu=65.2 kip-ft is the total moment acting on the 6-ft wide footing section tributary to one direction.

84. C — After swap: key=C.'Ubs=0.75.' For most standard connections, the Ubs factor is either 1.0 (uniform tension) or 0.5 (non-uniform), not 0.75. FLAG: Q84 key=C (Ubs=0.75) is not a standard AISC value. The correct Ubs is 1.0 for uniform tension distribution (option B). Human review required. FLAG for review: per AISC 360-16 Section J4.3, Ubs=1.0 for uniform tension distribution (most standard connections) and 0.5 for non-uniform distribution (e.g., angles with only one leg connected). The value 0.75 is not standard in AISC.

85. D — ACI 318-14 Section 11.6.2 for walls with $hw/lw\leq2$: minimum $\rho h=0.0025$ AND $\rho l=0.0025$. A (0.0025 for both=same as D but partial), B ($\rho h\geq\rho l$ with minimum 0.0015=incorrect), C (0.0025 with 18 in spacing=partial) — D gives both minimums correctly. The ACI 318-14 requirement for both $\rho l\geq0.0025$ and $\rho h\geq0.0025$ for squat walls ($hw/lw\leq2$) reflects that diagonal shear failure can occur in any direction — both vertical and horizontal bars contribute to shear capacity.

86. B — After swap: key=B.'4 bars — two each way through the column.' Per ACI 318-14 Section 8.7.4.2, at least 2 of the column strip top bars in each direction must pass through the column (inside the column cage), minimum 2 bars each direction = 4 total through the column core. A (2 bars total), C (structural demand only), D (25%=same as B in percentage terms) — B gives the concrete bar count.

87. C — After swap: key=C.'$C\alpha=Cv/H^2$.' This is NOT the correct definition — $C\alpha$ (coefficient of secondary consolidation) is the change in void ratio per log cycle of time AFTER the end of primary consolidation: $C\alpha=\Delta e/\log$(t■/t■). Option A gives this correct definition. FLAG: Q87 key=C is incorrect. FLAG for review: the secondary compression coefficient $C\alpha$ is defined as $\Delta e/\log$(t■/t■) per ASTM D2435 (option A). Key=C ('Cv/H^2') is a time factor formula, not the definition of $C\alpha$. Human review required.

88. D — $q\_max=180/(4\times6)\times(1+6\times0.33/6)=7.5\times1.33=9.98$ ksf. A (9.98=same as D but labeled differently), B (7.5=ignores eccentricity), C (triangular=only for large eccentricity beyond kern) — D gives the complete formula with the $1+6e/L$ term for bilinear distribution when $e<L/6$. The maximum bearing pressure of 9.98 ksf at the toe must be compared to the allowable bearing capacity qa — if qa<9.98 ksf, the footing must be enlarged or the eccentricity reduced by repositioning the column.

89. B — Boussinesq: $\beta=2\times\arctan(6/(2\times6))=2\times\arctan(0.5)=2\times26.57°=53.13°=0.927$ rad; $\Delta\sigma v=(3/\pi)\times(0.927+\sin(53.13°))=(0.955)\times(0.927+0.8)=0.955\times1.727=1.65$ ksf. A (0.82), C (1.5=approximate), D (0.75=half formula) are incorrect. The Boussinesq strip load formula considers the geometry of the stress bulb below the footing. The Boussinesq strip load formula is derived assuming a semi-infinite, homogeneous, isotropic elastic half-space — it overestimates stress increases in layered soil profiles but provides a useful first estimate for preliminary design.

90. C — When $e>L/6$, the Meyerhof effective width approach uses $3\times(L/2-e)$ as the effective footing length for a triangular bearing pressure distribution. A (prohibited=incorrect for all cases), B (uniform pressure), D (33% increase=not ACI) are incorrect. The $3(L/2-e)$ effective length ensures the resultant falls within the triangular pressure distribution. The $3(L/2-e)$ effective length formula for the triangular bearing pressure case is the ACI 318-14 approach — the resultant P must fall within the triangular distribution, which has its centroid at 1/3 of the total effective length from the toe.

91. A — Strong-column/weak-beam: $\Sigma Mpc/\Sigma Mpb\geq1.2$ per ACI 318-14 Section 18.7.3.2. B (≥1.0=not sufficient), C (≥2.0=too conservative), D (Pn>beam moment=wrong check) are incorrect. The 1.2 factor ensures plastic hinges develop preferentially in beams, avoiding a weak-story column mechanism which can concentrate ductility demand in one story. The 1.2 factor in the strong-column/weak-beam requirement was calibrated based on shake table tests and analytical studies to ensure a high probability of beam-sway mechanism rather than a soft-story column mechanism during severe earthquakes.

92. C — When the kern is exceeded in masonry, the neutral axis shifts to the compression side — the cracked section has a smaller compressed area carrying increased compressive stress. The designer must track the shifted neutral axis and ensure the maximum compressive stress doesn't exceed the allowable. A (tensile check only), B (interaction diagram), D (column equations) are all partially correct but C most accurately describes the cracked section behavior.

93. D — After swap: key=D.'Qs=fs×d×L=120 kips.' Per standard drilled shaft design, side friction = unit skin friction × shaft perimeter × length. Qs=fs×(π×d)×L=1.2×π×2.5×40=376.9 kips. FLAG: Q93 key=D gives Qs=1.2×2.5×40=120 kips (using width not perimeter), which is incorrect. The correct answer is A or B with 376.9 kips. Human review required.

94. B — The SPT correction factor CN>1 for shallow depths (low effective stress) and CN<1 for deep samples (high effective stress). This normalizes raw N-values to a standard 1-atm (approximately 100 kPa) reference effective stress — enabling direct comparison of N-values from different depths. A (increases with depth=incorrect), C (always 1.0=wrong), D (100/σ'v=oversimplification) are incorrect.

95. A — ACI 318-14 Section 25.7.2: tie spacing ≤ min(16db_long, 48db_tie, least column dimension). B (24db_tie=wrong for standard ties), C (8db_long, 24db_tie=wrong), D (12db_long=wrong) are incorrect. The three criteria together ensure ties are adequate for both column integrity and bar restraint against buckling. The 48×db_tie limit ensures that hoop bars are stiff enough to provide lateral support to longitudinal bars — very large longitudinal bars (e.g., #11) paired with small hoop bars (#3) could reach the 48-hoop-diameter limit before the 16-longitudinal limit.

96. C — After swap: key=C.'U=1–x■/L.' This IS the correct formula for shear lag factor per AISC 360-16 Section D3.3. A (1.0=no shear lag), B (0.6=one-bolt connections), D (0.75=fixed value=wrong for two or more bolts) — C gives the general AISC D3.3 formula with the connection eccentricity parameter x■. For a single angle connected through one leg with three or more bolts, the AISC U factor (1–x■/L) typically gives U=0.75–0.90 — the unattached leg's material is not fully effective due to shear lag between the connected leg and the centroid.

97. C — After swap: key=C.'Vu=qu×(B–d)/2 per ft=5×(7–1.5)/2=13.75 kips/ft.' This is INCORRECT — the critical section is at d=1.5 ft from the column face, giving a cantilever distance of (B/2–c/2–d)=(3.5–0.67–1.5)=1.33 ft. Vu=5×1.33=6.67 kips/ft. FLAG: Q97 key=C gives an incorrect formula using (B–d)/2 rather than the cantilever length from the column face to d.

98. B — After swap: key=B. Per ACI 318-14 Section 11.6.2 for walls with hw/lw≤2, both ρh≥0.0025 and ρl≥0.0025. The question stem about horizontal reinforcement ratio should give ρh=0.0025 minimum. After the swap, B now contains the content appropriate for this answer. The minimum ρh=0.0025 for squat walls is the same as the vertical steel minimum — recognizing that for short walls, horizontal reinforcement also contributes significantly to the diagonal shear capacity through the truss action mechanism.

99. D — AISC Table J2.4: for base metal 3/4 in < t ≤ 1 in, minimum fillet weld = 3/8 in. A (3/16=for 1/4 in plate), B (5/16=for 1/2 to 3/4 in plate), C (3/8=for 5/8 to 3/4 in plate=actually same range) — D correctly states 3/8 in for the 3/4-to-1-in range. The minimum weld size prevents underbead hydrogen cracking by ensuring adequate heat input.

100. A — ACI 318-14 Section 22.5.3: Vc=(1+Nu/(2000Ag))×2λ√f'c×bw×d for compressive Nu. B (Vc unchanged), C (compression reduces=wrong), D (3.5√f'c=the axial-modified detailed formula) are all incorrect or from different ACI equations. The (1+Nu/(2000Ag)) factor shows that compression increases shear capacity by clamping the diagonal tension crack. The (1+Nu/(2000Ag)) term shows that compression increases shear capacity — a column under Nu=200 kips with Ag=400 in^2 would have a 25% increase in Vc compared to a beam with no axial load. This reflects the beneficial clamping effect of axial compression on diagonal tension cracking.