PRACTICE EXAM 21: 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 4.3.2 and Table 4.3-1, the minimum design live load for a FIXED-seat assembly area (theater, concert hall, stadium with permanently fixed chairs) is:

A. 40 psf — for movable chair assembly, banquet halls, or dining rooms without fixed seats

B. 60 psf — for assembly areas with fixed seats such as theaters and concert halls per ASCE 7-16 Table 4.3-1

C. 100 psf — for open assembly floor areas without seating where standing crowds may pack tightly

D. 80 psf — for general assembly occupancies with a combination of fixed and movable chair arrangements

2. Per ASCE 7-16 Section 4.9 and Table 4.9-1, overhead traveling cranes must be designed for a minimum lateral (side thrust) force on each runway rail. This lateral force per rail is a percentage of the maximum wheel load and is applied to the top of the runway beam. For a bridge crane:

A. 20% of the maximum wheel load on that rail — the minimum lateral force per ASCE 7-16 Table 4.9-1

B. 20% of the sum of lifted load plus trolley weight — the total lateral force shared by both rails equally

C. 10% of the maximum wheel load per rail — a lighter lateral requirement used only for hand-operated crane hoists

D. 25% of lifted load plus trolley weight — an overestimate sometimes specified for extremely heavy crane service

3. Per ASCE 7-16 Section 7.5, the roof rain-on-snow surcharge load must be applied when flat-roof snow loads (pf) are less than:

A. 20 psf — the surcharge applies to all roofs with less than 20 psf flat-roof snow

B. 5 psf — only for locations with very light snowfall

C. 20 psf — the surcharge applies when pg < 20 psf and the ground snow is not negligible

D. 5 psf — the ASCE 7-16 threshold: when pf<5 psf, no rain-on-snow surcharge is needed since the design snow load already accounts for rain-on-snow

4. Per ASCE 7-16 Table 4.3-1, the minimum live load for ATTIC floors (residential) that have LIMITED STORAGE (not accessible by a stair) is:

A. 10 psf — for truly inaccessible (scissor-access only) attic storage areas

B. 40 psf — for attic floors that have been converted to habitable living space

C. 30 psf — for attic floors accessible by a permanent stair, even if not used as living space

D. 20 psf — for attic floors with limited storage, slope less than 3:12 per ASCE 7-16 Table 4.3-1

5. Per ASCE 7-16 Section 4.3.3, the minimum live load for MECHANICAL EQUIPMENT ROOMS (MER) and penthouses in a commercial building is:

A. 40 psf — the standard office floor occupancy minimum

B. 60 psf — for rooms with moderate mechanical equipment installations

C. 100 psf — for access floors over large mechanical equipment vaults

D. 150 psf — the standard for mechanical equipment rooms and penthouses per ASCE 7-16

6. Per ASCE 7-16 Section 26.12, a building parapet must be designed for wind pressure using the Component and Cladding (C&C) procedure or the MWFRS procedure. The parapet design wind pressure is computed using:

A. The MWFRS procedure for both windward and leeward faces — treating the parapet as a MWFRS element

B. The C&C procedure exclusively — parapet design uses component-level pressure coefficients

C. External pressure on windward plus leeward face combined — a net pressure per ASCE 7-16 Section 26.12

D. Internal pressure coefficients only — the parapet is exposed to interior building pressure on one face

7. Per ASCE 7-16 Section 4.3.1 and Table 4.3-1, the minimum live load for HELICOPTER LANDING AREAS (rooftop helipad) on a building structure is:

A. 40 psf — the standard roof live load for most commercial building roof areas

B. 100 psf — for heavy-lift helicopter pads rated for large aircraft over 3,000 lbs

C. 60 psf — the ASCE 7-16 Section 4.12 minimum for light helicopter landing areas on buildings

D. 60 psf plus a 3 klf strip load at the edge — per ASCE 7-16 Section 4.12, the larger effect governs

8. Per ASCE 7-16 Section 4.3.2.2, moving partition walls (non-fixed partitions) must be treated as live loads with a minimum weight of:

A. 10 psf — the minimum for light glass or aluminum partition systems

B. 15 psf — the ASCE 7-16 minimum uniform live load for movable office partition walls anywhere on the floor

C. 20 psf — for heavy reinforced concrete masonry unit partitions used in commercial buildings

D. 5 psf — for temporary demountable partition systems with no fixed connections to structure

9. Per ASCE 7-16 Section 11.4.3, the site coefficient Fa for Site Class D with Ss=1.0g is:

A. Fa = 1.0 — for Site Class D with Ss=1.0g per ASCE 7-16 Table 11.4-1

B. Fa = 1.1 — for moderate seismicity at Site Class D

C. Fa = 1.3 — for long-period site amplification

D. Fa = 1.6 — for soft soil with low spectral acceleration

10. Per ASCE 7-16 Section 26.9.2, the design velocity pressure qp for a building parapet is determined using the velocity pressure at the TOP OF THE PARAPET height. For a building with mean roof height 40 ft and parapet 3 ft tall (parapet top = 43 ft):

A. Use qh at the mean roof height 40 ft — the parapet uses the same qh as the roof MWFRS

B. Use qh at 43 ft for the windward parapet face and qh at 40 ft for the leeward face — two different pressures apply

C. Average qh between 40 ft and 43 ft — use the mean parapet height for the velocity pressure

D. Use qh at the parapet top elevation of 43 ft — the velocity pressure for parapet design uses the exposure height equal to the parapet top

11. Per ASCE 7-16 Table 4.9-1, the minimum lateral (horizontal transverse) crane force applied to the runway at each rail is 20% of the sum of the rated capacity of the crane plus the weight of the hoist and trolley. For a bridge crane with rated capacity 30 kips and trolley+hoist weight 10 kips, the total lateral force (shared by both rails) is:

A. 6 kips — 20% of the rated capacity only (not including trolley weight)

B. 8.0 kips — 20%×(30+10) = 8 kips total; distributed to each rail per wheel loads

C. 10 kips — 25% of the total including full runway beam weight

D. 4 kips — 10% of the sum for light-duty cranes

12. Per ASCE 7-16 Section 4.5.1, the minimum HYDROSTATIC PRESSURE (H load) on a buried basement wall or below-grade slab is determined by:

A. Assuming the groundwater is always at the ground surface — worst-case hydrostatic design

B. The weight of water at the high groundwater level confirmed by geotechnical investigation

C. Using 62.4 pcf × design water height — where the design height is the maximum expected water table depth confirmed by geotechnical data; ASCE 7-16 requires including H in load combinations whenever hydrostatic pressure acts

D. A minimum 30-pcf equivalent fluid pressure — the same as the minimum for basement wall soil pressure

13. Per ASCE 7-16 Table 12.3-1, a VERTICAL GEOMETRIC IRREGULARITY Type 4 (out-of-plane offsets) occurs when:

A. The lateral force resisting system has out-of-plane offsets that create discontinuities in the load path — vertical elements of the LFRS are offset out of the plane of the elements above or below

B. Adjacent story heights differ by more than 25% — a significant height irregularity

C. The mass at any floor exceeds 150% of the adjacent story mass — the mass irregularity

D. The lateral stiffness of a story is less than 70% of the story above

14. Per ASCE 7-16 Section 12.4.2, the seismic load effect E for LRFD combinations is E=Eh+Ev for the compression case and E=Eh−Ev for the tension case. The horizontal seismic load effect Eh equals:

A. Eh = QE — the seismic force from the elastic analysis without any amplification for ordinary members

B. Eh = ρ×QE — where ρ is the redundancy factor and QE is the seismic force from the linear analysis

C. Eh = Ω0×QE — the amplified seismic force used only for special elements

D. Eh = 0.7×QE — applying a 0.7 scaling factor in the LRFD seismic combination

15. Per ASCE 7-16 Section 12.3.3.4, when a structure has a PLAN HORIZONTAL IRREGULARITY (Type 1a, 1b, or 5), the design forces for connections of diaphragms to vertical elements must be increased by:

A. 10% — a minor increase applied only to SDC D buildings with minor plan irregularities

B. 25% — for all plan irregularity types in SDC B and above structures

C. 50% — reserved for structures with extreme torsional or reentrant corner irregularities only

D. 25% — the ASCE 7-16 Section 12.3.3.4 increase for diaphragm connection forces in irregular buildings

16. Per ASCE 7-16 Section 7.4.3, for unbalanced snow loads on GABLE ROOFS (slopes between 15° and 70°), the windward slope has a reduced snow load and the leeward slope has an increased snow load. The unbalanced leeward load is:

A. ps = 2.0×pf — double the balanced load on the leeward slope

B. ps = 1.5×Is×pg/Ce — based on ground snow with modified factors

C. ps = 0.5×pf on windward and 1.5×pf on leeward — the standard unbalanced gable roof distribution

D. ps = 0.3×pf on windward and 2.0×pf on leeward — only for heavily drifting regions

17. Per ASCE 7-16 Section 4.3.1 and Table 4.3-1, the minimum uniformly distributed live load for PUBLIC ROOMS AND CORRIDORS serving public rooms (e.g., hotel ballrooms, banquet rooms, corridors serving such spaces) is:

A. 40 psf — for hotel guest rooms and private dining rooms

B. 60 psf — for corridors in office buildings only

C. 80 psf — for corridors serving public assembly rooms with crowds

D. 100 psf — for public rooms (assembly) with movable seats per ASCE 7-16 Table 4.3-1

18. For a simply supported beam (span L=24 ft, EI constant), a concentrated load P=20 kips is placed at x=3L/4=18 ft from the left support. The bending moment at the load point (x=18 ft) is:

A. M = Ra×x = (P×b/L)×a = (20×6/24)×18 = 5×18 = 90 kip·ft — where Ra=P×b/L=20×6/24=5 kips

B. M = P×L/6 = 20×24/6 = 80 kip·ft — for a load at the third-span position

C. M = P×L/4 = 20×24/4 = 120 kip·ft — for a load at the midspan (not at 3L/4)

D. M = P×3L/4 = 20×18 = 360 kip·ft — incorrectly using full span distance without the b/L factor

19. In the direct stiffness method, STATIC CONDENSATION (Guyan reduction) is used to:

A. Improve the accuracy of eigenvalue solutions by adding mass terms

B. Remove internal DOFs that have no applied loads and are not needed in the final solution — expressing internal displacements in terms of boundary DOFs; reduces the system size

C. Condense multiple load cases into a single equivalent load vector

D. Apply boundary conditions by deleting rows and columns from the global stiffness matrix

20. For a symmetric two-span continuous beam under ANTI-SYMMETRIC loading (equal and opposite point loads P at the midspan of each span), taking advantage of symmetry:

A. The problem is solved using the symmetric half-structure with an anti-symmetric boundary condition (roller becomes a guided support — free to rotate, constrained against horizontal movement)

B. The interior support reaction is zero by symmetry — the two spans act as independent simply supported beams

C. The anti-symmetric loading produces maximum moment at the interior support — the symmetric condition with pinned boundary gives MB = P×L/4

D. The beam can be modeled as a fixed-fixed beam of length L using the anti-symmetry axis as a fixed boundary

21. For a propped cantilever beam (fixed at A, roller at B) under a UNIFORM LOAD w on the full span, the number of plastic hinges required to form a collapse mechanism is:

A. 3 plastic hinges — at both fixed ends and at one interior point, giving a mechanism for a fixed-fixed beam

B. 2 plastic hinges — one at the fixed end only and one at the inflection point of maximum moment

C. 1 plastic hinge — a single hinge at the fixed end where the maximum negative moment occurs

D. 2 plastic hinges — at the fixed end A and at the maximum positive moment point, since n=1 indeterminate → n+1=2 hinges for a collapse mechanism

22. For a frame structure, the NUMBER OF INDEPENDENT SWAY MECHANISMS equals:

A. The number of unbraced stories — one lateral DOF per story

B. The number of degrees of kinematic indeterminacy minus the number of plastic hinges at collapse

C. The number of side-sway DOFs — determined by the number of independent horizontal story translations in the frame

D. The number of columns — one sway mechanism per column

23. For the three-moment equation applied to a two-span beam (spans L■ and L■ with EI constant), if END A is PINNED and END C is PINNED, and only span 1 (L■=L■=L) has a UDL w, the interior moment MB is:

A. $-wL^2/16$ — the result from the three-moment equation for one span loaded, both ends pinned

B. $-wL^2/8$ — for both spans loaded with equal UDL and both ends pinned (symmetric case)

C. $-3wL^2/32$ — from a moment distribution approach with slightly different coefficients

D. $-wL^2/12$ — for a propped cantilever with one fixed end, not a pinned-pinned system

24. For a frame structure analyzed by the flexibility method (method of consistent deformations), the FLEXIBILITY COEFFICIENT fij represents:

A. The force at point i due to a unit displacement at point j — a stiffness coefficient

B. The displacement at point i due to a unit force at point j — the fundamental definition of a flexibility coefficient

C. The moment at i due to a unit load at j — only for moment frames

D. The stiffness at point i when a unit force is applied at j — the inverse of the stiffness coefficient

25. For a three-hinge arch (span L=60 ft, rise f=12 ft, concentrated load P=40 kips at x=20 ft from the left support), the VERTICAL REACTION at the left support A is:

A. $RA = P \times (L-a)/L = 40 \times 40/60 = 26.67$ kips — same as a simply supported beam

B. $RA = P \times a/(2L)$ — for arch reaction accounting for the parabolic shape

C. $RA = P/2 = 20$ kips — arches always share load equally between supports

D. $RA = P \times (L+a)/(2L)$ — for the modified arch reaction formula

26. For Castigliano's Second Theorem applied to a SEMICIRCULAR ARCH (radius R, EI constant) under a horizontal load H at the crown with both ends pinned, the horizontal displacement at the crown is found from $\partial U/\partial H$. The key difference from a straight beam analysis is:

A. The integration must be carried out in polar coordinates — $ds = R \times d\theta$ rather than the straight-beam dx

B. The arch carries axial thrust in addition to bending — requiring both $M^2/(2EI)$ and $N^2/(2AE)$ in the strain energy

C. The curved geometry requires multiplying the moment by $\sin(\theta)$ instead of a simple distance

D. The arch does not store strain energy — all energy is transmitted as thrust to the supports

27. For a TWO-BAY portal frame (three columns, two beams) with equal bay widths and equal column heights, under a lateral load H at the top, the PORTAL METHOD assumes that:

A. Interior columns carry twice the shear of each exterior column — inflection points at mid-column and mid-beam heights

B. Exterior columns carry twice the interior column shear — based on perimeter stiffness advantage

C. All three columns carry equal shear — assuming uniform lateral stiffness regardless of position

D. Shear distributes proportionally to column EI/h^3 — the stiffness-based distribution method

28. For a beam on an elastic foundation (beam resting on Winkler springs, modulus k per unit length), the CHARACTERISTIC LENGTH λ that governs the response is:

A. $\lambda = (k/(4EI))^{(1/4)}$ — the inverse of the elastic foundation length parameter β; a shorter λ means stiffer foundation

B. $\lambda = \sqrt{(k/EI)}$ — the angular frequency of the beam vibration on the foundation

C. $\lambda = (4EI/k)$^(1/4) — the characteristic length for the beam-on-elastic-foundation problem; determines the length over which the beam deflects under a point load

D. $\lambda = EI/k$ — the dimensionless stiffness ratio of beam to foundation

29. For the MOMENT AREA METHOD applied to a simply supported beam under a UDL w (span L), the slope at the left support A equals the area of the M/EI diagram divided by L (by the first moment-area theorem for SS beams using the elastic weight method). The slope at A is:

A. $\theta A = wL^3/(24EI)$ — the standard result for a SS beam under UDL

B. $\theta A = wL^3/(48EI)$ — for a beam with one fixed end

C. $\theta A = wL^2/(8EI)$ — the moment-area slope formula, missing one power of L

D. $\theta A = wL^3/(16EI)$ — for a UDL on only half the span

30. For the FLEXIBILITY MATRIX [f] of a statically indeterminate structure with redundants chosen as a set of unknown forces, the inverse of [f] is related to the STIFFNESS MATRIX [K] of the structure by:

A. [K] = [f]■^{1} — the stiffness matrix is the inverse of the flexibility matrix (for the same DOF set)

B. [K] = –[f] — the stiffness is the negative of the flexibility for antisymmetric systems

C. [K] = [f] — they are equal for symmetric structures

D. [K] = [f]^2 — the stiffness is the square of the flexibility matrix

31. For a cantilever beam (fixed at A, free at B, span L) under a TRIANGULARLY DISTRIBUTED LOAD (0 at free end B, maximum w at fixed end A), the tip deflection at B is:

A. δB = wL■/(8EI) — for a UDL on the cantilever

B. δB = wL■/(30EI) — from the moment-area theorem for a triangular load on a cantilever with maximum at the fixed end

C. δB = wL■/(30EI) — the tip deflection for triangular load increasing from 0 at B to w at A

D. δB = wL■/(120EI) — the tip deflection for the reversed triangular load (maximum at free end)

32. BETTI'S RECIPROCAL THEOREM states that for two load systems {P■} and {P■} applied to an elastic structure, the work done by {P■} through the deflections caused by {P■} equals the work done by {P■} through the deflections caused by {P■}. One practical application of this theorem is:

A. To prove that the global stiffness matrix [K] is symmetric — Maxwell's theorem directly implies $k_{ij} = k_{ji}$ in the assembled matrix

B. To compute IL ordinates via a unit displacement approach — the IL is the elastic deflected shape from Müller-Breslau

C. To establish that structural buckling modes are mutually orthogonal in the modal decomposition of $[K-\lambda K_G]$

D. To convert impulsive dynamic loads to quasi-static equivalents using the concept of strain energy equivalence

33. For PLASTIC ANALYSIS of a fixed-fixed beam under a UDL w, three plastic hinges form at both fixed ends and at midspan. Applying the UPPER BOUND THEOREM (virtual work), if each hinge has plastic rotation θ at the ends and 2θ at midspan:

A. $wc \times L^2/4 = 2Mp \rightarrow wc = 8Mp/L^2$ — incorrectly applying only two plastic hinges

B. $wc \times L^2/8 = Mp \times \theta + Mp \times 2\theta + Mp \times \theta = 4Mp \times \theta \rightarrow wc = 32Mp/L^2$ — from the virtual work balance

C. $wc \times L^2/8 = 3Mp \times \theta \rightarrow wc = 24Mp/L^2$ — using three hinges all rotating by θ

D. $w_c = 16M_p/L^2$ — the correct result for UDL fixed-fixed collapse: external work = $w_c \times L \times (L/4)/2 \times 2 = w_c \times L^2/4$, internal = $4M_p \times \theta$; with geometry $\theta = 4\delta/L$, giving $w_c \times L^2/4 = 4M_p \times 4/L \rightarrow w_c = 16M_p/L^2$

34. For a frame with sway, the CHORD ROTATION ψ for a column of height h with a lateral sway Δ is defined as:

A. $\psi = \Delta/h$ — the ratio of relative lateral displacement of column ends to column height

B. $\psi = \Delta \times h$ — the product of displacement and height

C. $\psi = \theta_{near} - \theta_{far}$ — the difference in joint rotations at the two ends of the column

D. $\psi = \Delta/(2h)$ — using half the column height for the chord rotation

35. For a SIMPLY SUPPORTED BEAM under a MOVING CONCENTRATED LOAD P traversing from left to right, the MAXIMUM SHEAR at the LEFT SUPPORT occurs when:

A. The load is directly at the left support — producing Va=P (full reaction at A)

B. The load is at the right support — producing Va=0 at A

C. The load is at midspan — producing Va=P/2

D. The load is at distance L/4 from the left — a 75% reaction at A

36. For the stiffness method, the EQUIVALENT NODAL LOADS for a member with a distributed load are computed as the NEGATIVE of the:

A. Fixed-end reactions — the member is first restrained against all DOF, fixed-end forces are computed, and then the negatives of those are the equivalent nodal loads used in the global load vector

B. Applied distributed load intensity — the load per unit length is converted to a point load

C. Midpoint reactions — only the midpoint reaction translates to a nodal load

D. Reaction at the nodes from a simply supported beam under the same distributed load

37. For the CONJUGATE BEAM METHOD, the conjugate beam corresponding to a FIXED-FIXED real beam has what support conditions:

A. Free-Free: both ends are free in the conjugate beam — fixed real ends become free conjugate ends (no moment, no shear)

B. Fixed-Fixed: same boundary conditions as the real beam

C. Pinned-Pinned: fixed ends become pinned ends in the conjugate beam

D. Pinned-Fixed: one end changes boundary condition but the other stays fixed

38. For a multi-story MOMENT FRAME, the CANTILEVER METHOD for lateral load analysis assumes that:

A. Column axial forces are proportional to their distance from the centroid of the column group — like a vertical cantilever under overturning

B. All columns carry equal axial forces — lateral overturning is shared uniformly across all columns

C. Interior columns carry twice the axial force of exterior columns — the same as the portal method for shears

D. Column forces are proportional to stiffness — a full stiffness analysis is needed to apply the cantilever method

39. For STABILITY analysis of a rigid frame using the ENERGY METHOD (potential energy), the CRITICAL LOAD for buckling is found when:

A. The second variation of the total potential energy is zero — $d^2V=0$ at the critical load (transition from stable to unstable equilibrium)

B. The first variation of the potential energy is zero — $\partial V/\partial q=0$ gives the equilibrium condition (not the critical load)

C. The strain energy exceeds the work done by applied loads — the criterion for elastic stability

D. The column shortening under axial load equals the beam elongation under bending

40. For a BEAM-COLUMN (combined axial compression P and lateral load w), the AMPLIFICATION FACTOR for midspan deflection due to first-order analysis (δ■) compared to the second-order deflection (δ) is approximately:

A. δ/δ■ = 1/(1–P/Pe) — where $Pe=\pi^2EI/L^2$ is the Euler buckling load; the first-order deflection is amplified by this factor

B. δ/δ■ = (1+P/Pe) — the deflection increases proportionally to the axial load

C. δ/δ■ = P/Pe — the ratio of applied to critical load

D. δ/δ■ = √(Pe/P) — a geometric mean amplification factor

41. For a SPACE TRUSS (3D truss), the condition for STATIC DETERMINACY is:

A. m + r = 3j — where m=members, r=support reactions, j=joints (3 DOF per joint in 3D)

B. m + r = 2j — the 2D truss determinacy condition (incorrect for 3D)

C. m = 3j – 6 — excluding support reactions

D. m + r = 4j — overly conservative condition for 3D truss systems

42. For the STIFFNESS METHOD of analysis, the ASSEMBLY of the global stiffness matrix from element stiffness matrices is accomplished by:

A. Multiplying all element matrices together — computing the global matrix as a product of local matrices

B. Taking the inverse of the flexibility matrix — the global stiffness is assembled by inverting the global flexibility

C. Superimposing (adding) the element contributions at the appropriate DOF positions — the direct stiffness assembly adds each element's stiffness contribution to the global DOFs corresponding to that element

D. Averaging the element stiffness matrices at shared nodes — a weighted averaging scheme based on element lengths

43. Per ACI 347R-14 and ACI 305R-10 (Hot Weather Concreting), when the concrete temperature at placement exceeds 90°F (or 95°F in some references), the primary concern for formed concrete is:

A. Increased hydrostatic pressure — hotter concrete sets faster so pressure builds up more quickly

B. Reduced bleed water — hot weather lowers the water-cement ratio at the surface prematurely

C. Accelerated stiffening and reduced formwork pressure — hot concrete stiffens faster, shortening the fluid-pressure phase

D. Reduced workability — the concrete becomes difficult to vibrate but formwork pressure is not directly affected

44. Per OSHA 29 CFR 1926.703, if a formwork or shoring system failure occurs during concrete placement, the FIRST action required is:

A. Continue pouring from the unaffected portion and shore the failed zone — redistributing load is safe

B. Document the failure fully and consult the structural engineer before making any decision to stop

C. Stop all concrete placement immediately and notify the competent person — investigate before resuming

D. Remove workers from the danger zone and continue placement at a safe distance from the distress

45. Per ACI 347R-14 Section 3.1, the standard CONCRETE WEIGHT used to determine formwork lateral pressure (for normal weight concrete without special additives) is:

A. 150 pcf — the unit weight used in the ACI 347R lateral pressure formula for normal weight concrete

B. 145 pcf — a commonly used design value accounting for about 2% air entrainment in fresh concrete

C. 155 pcf — for high-density concrete mixes used in nuclear shielding and counterweight applications

D. 160 pcf — for heavily reinforced sections where steel displacement increases the unit weight

46. Per ACI 347R-14, when the concrete placement RATE EXCEEDS 7 ft/hr (for walls), the design lateral pressure should be taken as:

A. p = 150H psf — a full-hydrostatic formula based only on the pour height above the bottom of the form

B. p = CW×CC×(150+9000R/T) but capped at wc×H — ACI formula applies and limits to hydrostatic pressure

C. p = 2,000 psf — a conservative fixed maximum independent of pour rate, height, or concrete unit weight

D. p = wc×H — full hydrostatic pressure because the concrete stays fluid at high pour rates and the ACI formula no longer applies

47. Per OSHA 29 CFR 1926.601 and good engineering practice, a CONCRETE PUMP BOOM is subject to which restriction when in use on a construction site near electrical power lines:

A. No restriction — concrete pump booms are grounded and not affected by power lines

B. The boom must maintain a 10-ft minimum clearance from power lines rated up to 50 kV

C. The boom operator must have a spotter in position to observe the boom-to-powerline distance

D. An OSHA-compliant buffer zone of 20 ft around all energized power lines must be maintained for all boom equipment operations

48. Per ACI 347R-14, the minimum concrete COMPRESSIVE STRENGTH required before stripping WALL FORMS (not re-shored, for walls that do not support construction loads) is:

A. 24-hour cylinder strength regardless of specified f'c

B. 500 psi — the minimum to prevent damage during form stripping and handling

C. 70% of specified f'c — to ensure the wall can support its own weight and any minor construction loads after form removal

D. The wall must reach the design strength f'c before forms are stripped — no reduction is permitted for architectural surfaces

49. Per ACI 347R-14, the purpose of FORM TIES in wall formwork is to:

A. Connect adjacent form panels horizontally to align them vertically — ties are alignment devices

B. Resist the LATERAL CONCRETE PRESSURE that pushes the two opposing wall form faces apart — ties carry the net tensile force from the hydrostatic and fresh-concrete pressure, preventing the forms from spreading

C. Anchor the formwork to the concrete foundation to prevent uplift

D. Transfer vertical loads from the wall form to the base shoring system

50. Per ASTM A913 (Quench-and-Self-Tempered — QST structural steel), the Grade 65 minimum yield strength is 65 ksi. The primary ADVANTAGE of A913 over A572 Grade 65 for welded connections in seismic applications is:

A. Lower carbon equivalent — A913 QST process achieves high strength with reduced alloying, giving better weldability and lower preheat requirements than conventional high-strength steels

B. Higher yield strength — A913 provides Fy up to 100 ksi, far exceeding A572

C. Better corrosion resistance — A913 includes weathering additives

D. Higher fracture toughness — A913 provides Charpy impact values at –70°F

51. Per ASTM C150, TYPE III cement (high early strength) achieves its higher early compressive strength compared to Type I cement through:

A. A higher C■S (tricalcium silicate) content and finer grinding — the combined effect of more reactive phase and greater surface area accelerates early hydration

B. Addition of accelerating admixtures — Type III cement always contains calcium chloride

C. Lower water-to-cement ratio requirement — Type III needs less water

D. Different raw material composition — Type III uses limestone with lower silica content

52. Per ACI 207.1R (Mass Concrete), the primary concern in MASS CONCRETE placements (large footings, dams, thick mat slabs) is:

A. Thermal cracking — the interior heats up more than the surface, creating tensile thermal stresses that crack the concrete

B. Alkali-silica reaction — the elevated cement content in mass placements promotes early ASR deterioration

C. Insufficient hydration — the large section lacks surface moisture access for proper curing of interior concrete

D. Reduced long-term strength — elevated early temperatures permanently damage the concrete microstructure

53. Per NDS 2018 Section 3.10 and Table 3.10.3, when a WOOD MEMBER is subjected to COMBINED BENDING AND TENSION (biaxial stress), the interaction check is:

A. (ft/Ft')^2 + (fb/Fb') ≤ 1.0 — a quadratic interaction for combined tension and bending

B. ft/Ft' + fb/Fb' ≤ 1.0 — the NDS linear interaction for combined tension and bending on the same face

C. ft/Ft' + fb/Fb' ≤ 1.0 — but the bending term uses the NET section modulus reduced by tension

D. (ft/Ft')×(fb/Fb') ≤ 1.0 — a product interaction criterion

54. Per ASTM D2487, a soil is classified as CH when:

A. It has more than 50% fines, LL > 50, and plots ABOVE the A-line on the Casagrande plasticity chart (PI > 0.73(LL–20))

B. It has more than 50% fines, LL < 50, and plots above the A-line

C. It is a coarse-grained soil with more than 12% fines passing the #200 sieve

D. It has more than 50% fines and PI > 30 regardless of position relative to the A-line

55. Per ASTM C33, the maximum FINENESS MODULUS (FM) permitted for FINE AGGREGATE used in normal concrete is:

A. FM ≤ 3.1 — the ASTM C33 upper limit; coarser sand provides better workability but requires more paste

B. FM ≤ 2.3 — the standard lower limit for coarse-medium sand used in structural concrete mixes

C. FM ≤ 2.0 — for fine aggregate specified in architectural or exposed concrete surface applications

D. FM ≤ 4.0 — for specialty lightweight aggregate concrete where coarser gradation aids drainage

56. Per NDS 2018 Section 3.8 (Lag Screws), the withdrawal design value W for lag screws (lag bolts) in a main member is proportional to:

A. The specific gravity G^1.5 and the shank diameter D — withdrawal capacity W = 1,800×G^1.5×D^0.75 (per NDS Table 11G); depends on both wood density and screw size

B. The square root of the specific gravity G^1.5 and the shank diameter D

C. Only the penetration depth — diameter has no effect on withdrawal per NDS

D. The ratio of penetration to diameter — a normalized withdrawal resistance

57. Per ACI 318-14 Section 4.2.2 and ASTM C330, the equilibrium (dry) unit weight of LIGHTWEIGHT AGGREGATE CONCRETE used in structural elements must not exceed:

A. 100 pcf — for very lightweight insulating concretes

B. 115 pcf — for all-lightweight concrete (lightweight fine and coarse aggregates)

C. 130 pcf — for light-weight aggregate meeting ACI structural requirements

D. 135 pcf — the ASTM C330 upper limit for structural lightweight concrete

58. Per TMS 402-16 Section 9.1, for REINFORCED MASONRY designed by the STRENGTH design method, the nominal moment capacity of a rectangular section (b × d, single reinforcement As) is:

A. Mn = 0.9×As×fy×d — applying a reduction factor to the yield moment without the stress block

B. Mn = As×fy×d — using the full effective depth without the compression block reduction

C. Mn = f'm×b×d² / 6 — treating the masonry as an elastic section

D. Mn = As×fy×(d – a/2) — where a = As×fy/(0.85×f'm×b) is the equivalent stress block depth, analogous to ACI 318-14

59. Per NDS 2018 Section 12 and the NDS Supplement Table 12N, the REFERENCE LATERAL DESIGN VALUE Z for a laterally loaded NDS nail (common wire nail) depends on:

A. Nail shank diameter alone — the NDS value is standardized per diameter class without species adjustment

B. Specific gravity of the wood species only — the EYM design value is normalized to specific gravity

C. Nail length and embedment depth only — the NDS yield value depends on penetration, not diameter

D. Nail diameter, embedment depth, and wood species specific gravity — all three govern via the EYM yield mode equations

60. Per ASTM A572 vs ASTM A992 (for W-shapes), ASTM A992 imposes additional requirements that A572 does not, specifically:

A. A maximum yield strength of 65 ksi — A992 limits Fy to a range rather than a minimum only

B. A minimum alloy content of chromium plus molybdenum

C. A required yield-to-tensile ratio Fy/Fu ≤ 0.85 AND a maximum Fy = 65 ksi — both controls ensure adequate strain hardening capacity for plastic hinging in seismic frames

D. A minimum Charpy toughness at low temperature

61. Per ACI 318-14 Section 26.12.3 and general structural concrete practice, the MAXIMUM CONCRETE TEMPERATURE AT PLACEMENT (temperature of fresh concrete as it is deposited in forms) per ACI 305R recommendations for hot weather concreting is:

A. 70°F — the ideal concrete placement temperature

B. 90°F — the ACI 305R maximum recommended placement temperature for most structural concrete

C. 95°F — the absolute maximum in ACI 318-14 for all concrete classes

D. 100°F — permitted for mass concrete only with special cooling measures

62. Per ASTM A572 and general practice, GRADE 50 structural steel is preferred over A36 for COMPRESSION MEMBERS (columns) primarily because:

A. A572 Grade 50 has higher compressive strength — compression members benefit from the increased Fy for yielding-governed design

B. A572 Grade 50 has better buckling resistance — the higher modulus E prevents elastic buckling

C. A572 columns are lighter for the same load capacity — the higher Fy allows smaller cross-sections, saving weight in strength-governed cases

D. A572 is exclusively used for columns — A36 is restricted to tension members only by AISC

63. Per TMS 402-16 Section 7.3.3, the MINIMUM CLEAR DISTANCE between parallel reinforcing bars in a grouted masonry wall is:

A. db — one bar diameter minimum between bars

B. db or 1 in — the larger of one bar diameter or 1 inch

C. 25 mm (approximately 1 in) — following the international masonry code convention

D. Not less than db, 1 in, or 1.33 × the maximum aggregate size — whichever is greatest, per TMS 402-16

64. Per ACI 318-14 Section 22.4.2 (Biaxial Bending in Columns), the Bresler Load Contour or Reciprocal Load Method approximates the nominal biaxial axial capacity Pni using: 1/Pni = 1/Pnx + 1/Pny − 1/P0. For a tied column with Pnx=600 kips, Pny=450 kips, and P0=1,000 kips:

A. Pni = min(Pnx, Pny) = 450 kips

B. Pni = 1/(1/600+1/450–1/1000) = 1/(0.00167+0.00222–0.001) = 1/(0.00289) ≈ 346 kips

C. Pni = P0×Pnx×Pny/(Pnx×Pny + P0×(Pnx+Pny)) — the alternative form

D. Pni = (Pnx×Pny)/(Pnx+Pny) = 257 kips — harmonic mean

65. Per ACI 318-14 Section 22.4.2.1, columns must be designed for a minimum eccentricity of the axial load. The minimum design eccentricity (expressed as a fraction of column dimension h) for a TIED COLUMN is:

A. e_min = 0.0 — ACI 318-14 no longer uses minimum eccentricity; it uses ϕPn,max = 0.80ϕPo to limit concentric load capacity

B. e_min = 0.10h — the column must be designed for at least 10% of the column dimension in eccentricity

C. e_min = 0.05h or 1.0 in — a fixed minimum eccentricity for tied columns regardless of applied load

D. e_min = 0.10h or 1.5 in — for seismic applications where the column may see unexpected eccentricity

66. Per AISC 360-16 Section J8 and AISC Design Guide 2, for ANCHOR BOLTS (post-installed anchors in concrete), when shear and tension interact, the design must satisfy:

A. (Nu/ϕNn)^(5/3) + (Vu/ϕVn)^(5/3) ≤ 1.0 — the ACI 318-14 5/3-power interaction for anchors in tension and shear

B. Nu/ϕNn + Vu/ϕVn ≤ 1.0 — the linear interaction for tension-shear

C. $(Nu/\phi Nn)^2 + (Vu/\phi Vn)^2 \le 1.0$ — a circular interaction criterion for anchors

D. Nu/ϕNn + Vu/ϕVn ≤ 1.2 — with the 1.2 allowance for combined loading

67. Per AISC 360-16 Section J4.1, for a GUSSET PLATE loaded in compression by a diagonal brace, the COMPRESSION BUCKLING limit state requires checking:

A. The plate as a column using the Whitmore section width as b and the free length as L — KL/r applies to the effective length of the plate between the Whitmore section and the nearest brace connection

B. The plate using the net section in compression — buckling does not govern gusset plates

C. The plate as a beam-column under combined axial and bending — the diagonal brace induces both

D. Only the connection welds — gusset plates in compression are always limited by weld capacity

68. Per ACI 318-14 Section 9.3.1, the MINIMUM DEPTH of a non-prestressed one-way slab (simply supported, L=18 ft) to satisfy the serviceability deflection limit WITHOUT computing deflection is:

A. h_min = L/20 = 18×12/20 = 10.8 in — the ACI 318-14 Table 9.3.1.1 minimum for simply supported slabs

B. h_min = L/16 = 18×12/16 = 13.5 in — for one-end-continuous (spandrel) slabs

C. h_min = L/28 = 7.7 in — for two-end-continuous slabs per ACI Table 9.3.1.1

D. h_min = L/30 = 7.2 in — for cantilever slabs with fixed end at the wall

69. Per ACI 318-14 Section 18.6.3.1, the maximum longitudinal tensile reinforcement ratio in SPECIAL MOMENT FRAME BEAMS must not exceed:

A. ρmax = 0.025 — the ACI 318-14 Section 18.6.3.1 absolute maximum for SMF beam tension reinforcement

B. ρmax = 0.033 — the general maximum reinforcement ratio for non-seismic beams

C. ρmax = 0.75ρb — the older ACI 318-99 maximum for over-reinforced sections

D. ρmax = 0.04 — a relaxed limit for beam-columns where axial load reduces plastic rotation demand

70. Per AISC 360-16 Section F3, for an I-shaped member with a NON-COMPACT FLANGE (λ_f between λpf and λrf) and lateral bracing sufficient to prevent LTB (Lb < Lp), the nominal flexural strength Mn is:

A. Mn = Mp — the plastic moment, since LTB does not govern

B. Mn = Fcr×Sx — using the critical stress for elastic local buckling

C. Mn = 0.7FySx — the elastic limit for all non-compact sections

D. Mn = Mp – (Mp – 0.7FySx)×(λf–λpf)/(λrf–λpf) — the linear interpolation between Mp and 0.7FySx as flange slenderness increases

71. Per ACI 318-14 Section 14.3.2, the design of BASEMENT WALLS (cast-in-place reinforced concrete walls spanning vertically between the footing and the first floor) under LATERAL SOIL PRESSURE is treated as:

A. A deep beam spanning horizontally between pilasters — the wall bends in its own plane

B. A two-way slab spanning in both the horizontal and vertical directions between supports

C. A shear wall resisting the lateral soil pressure by in-plane shear action alone

D. A one-way slab spanning vertically between the footing and first floor, carrying lateral soil pressure in bending

72. Per AISC 360-16 and AISC Design Guide 9, the design of an ECCENTRICALLY LOADED WELD GROUP (horizontal weld under a shear + moment) uses the INSTANTANEOUS CENTER (IC) method or the ELASTIC method. The IC method is PREFERRED because:

A. It is simpler and faster than the elastic method — fewer steps are needed to calculate eccentric weld forces

B. It always gives more conservative results — the IC method overestimates demand on the critical weld segment

C. It accurately models non-linear weld force-deformation and gives a more economical result than the elastic method

D. AISC requires the IC method — the elastic method is explicitly prohibited for eccentric weld groups per AISC J2

73. Per AISC 360-16 Section G3, when TENSION FIELD ACTION (post-buckling shear resistance) is permitted in a plate girder web, the nominal shear strength is increased. Tension field action is NOT permitted in:

A. Interior panels with a/h > 3 — aspect ratio too wide for effective diagonal tension field development

B. End panels adjacent to the simple beam supports — no adjacent panel to anchor the tension field boundary

C. Panels with web cutouts or access holes — the opening disrupts the diagonal stress path

D. Composite girders — the deck slab prevents formation of the diagonal tension field in the compression direction

74. Per ACI 318-14 Section 13.2.2.1, for a STRIP FOOTING under a masonry wall (wall width = 12 in, footing width B=5 ft, effective depth d=14 in), the critical section for FLEXURE in a footing under a wall is at:

A. At the centerline of the wall — the footing is designed as a full cantilever from one edge to the other

B. At the face of the wall — the footing cantilevers from the wall face to the free edge on each side

C. At d/2 from the wall face — the two-way punching shear critical section (not applicable for strip footings)

D. At L/4 from the footing edge — an intermediate section used for fatigue checks on cyclically loaded footings

75. Per AISC 360-16 Section J8.2 and Table J3.1, when a bolt is installed in an OVERSIZED HOLE in a bearing-type connection, the bearing strength reduction factor compared to a standard hole is:

A. ϕ×(2.4FuDt) — same as for standard holes, no reduction for oversized holes

B. ϕ×(2.0FuDt) — reduced from 2.4 to 2.0 for oversized holes in bearing

C. ϕ×(2.4FuDt) for single bolts but ϕ×2.0 for bolt groups — an averaging reduction

D. ϕ×(1.8FuDt) — a significant reduction for oversized holes relative to the standard 2.4

76. For a sand deposit (Gs=2.70, void ratio e=0.65), the critical hydraulic gradient ic at which QUICK SAND (upward seepage-induced floating) occurs is:

A. ic = Gs–1 = 1.70 — an oversimplified formula that omits the void ratio correction

B. ic = (Gs–1)×(1+e) = 2.81 — incorrectly multiplying rather than dividing by (1+e)

C. ic = (Gs–1)/(1+e) = 1.70/1.65 = 1.030 — the correct quick-sand critical gradient formula

D. ic = Gs/(1+e) = 2.70/1.65 = 1.636 — using full Gs rather than the submerged specific gravity (Gs–1)

77. Per ACI 318-14 Section 13.3.1, for a COMBINED FOOTING under two columns (Column 1: P■=150 kips at x=0; Column 2: P■=200 kips at x=12 ft), the footing LENGTH L for UNIFORM SOIL PRESSURE (centroid of footing coincides with resultant) must satisfy the condition that the resultant location x_R = P■×12/(P■+P■) from Column 1:

A. x_R = 200×12/(150+200) = 6.86 ft from Column 1 — the footing centroid is at x_R; total length L = 2×x_R + overhang past Col 1

B. x_R = 150×12/(350) = 5.14 ft from Column 1 — the resultant is closer to the lighter column

C. x_R = (P■×0+P■×12)/(P■+P■) = 6.86 ft — L is determined so that L/2 = x_R + edge offset

D. x_R = 6 ft — the midpoint regardless of load magnitudes

78. For a normally consolidated clay layer (Hdr=4 ft for double drainage), the DEGREE OF CONSOLIDATION U at time factor Tv=0.50 is approximately:

A. U = 55% — from the early-time approximation Tv=π/4×U^2 giving U=√(4×0.50/π)=0.798=80% — this approximation overestimates U for Tv>0.20

B. U = 65% — an intermediate value from linear interpolation of consolidation tables

C. U = 70.7% — from the square root formula U=√(Tv×4/π) — exact for early times only

D. U ≈ 76.4% — from the Terzaghi consolidation solution; at Tv=0.50, the standard consolidation chart gives U≈76%

79. Per AISC 360-16 Section G2.2, the WEB CRIPPLING limit state for a concentrated compressive load on the top flange of a W-shape occurs at a distance from a beam end of more than d/2. The nominal web crippling strength (bearing AWAY from beam end) is:

A. Rn = 0.80tw^2[1+3(N/d)(tw/tf)^1.5]√(EFytf/tw) — the AISC G2.2 crippling formula for interior loads

B. Rn = 0.40tw^2[1+3(N/d)(tw/tf)^1.5]√(EFytf/tw) — for loads at the beam end

C. Rn = Fyw×tw×N — a simple web bearing formula without crippling effects

D. Rn = 2.5kdes×Fyw×tw — the web local yielding formula, not crippling

80. Per ACI 318-14 Section 18.12 (Diaphragms in Special Structural Wall Buildings), the diaphragm design force Fpx for each diaphragm level x must satisfy the equation Fpx = (ΣFi/ΣWi)×Wpx and must not be less than:

A. 0.2×SDS×Ie×Wpx — the ASCE 7-16 Section 12.10.1.1 minimum diaphragm design force

B. 0.15×SDS×Ie×Wpx — for Risk Category II buildings only

C. 0.1×SDS×Ie×Wpx — an absolute minimum regardless of building configuration

D. 0.25×SDS×Ie×Wpx — for buildings with nonparallel systems irregularity

81. Per ACI 318-14 Section 18.10.3 (Special Structural Walls — Shear), the maximum nominal shear strength Vn for a special structural wall web shall not exceed:

A. Vn,max = 6λ√f'c×Acv — for each individual wall pier with hw/lw < 1.5

B. Vn,max = 8λ√f'c×Acv — for individual wall segments in coupled wall systems

C. Vn,max = 10λ√f'c×Acv — the upper limit for the entire wall section

D. Vn,max = 8λ√f'c×Acv for the overall wall AND 10λ√f'c×Acv for individual segments of the same wall system

82. For a stratified soil deposit (Layer 1: k■=1×10■■ cm/s, thickness h■=2 m; Layer 2: k■=1×10■■ cm/s, thickness h■=3 m), the HORIZONTAL permeability kH of the entire deposit is:

A. kH = (k■h■+k■h■)/(h■+h■) = (2×10■■+3×10■■)/5 = 4.06×10■■ cm/s — the weighted arithmetic average

B. kH = (h■+h■)/(h■/k■+h■/k■) = 5/(20,000+300,000) = 1.56×10■■ — the harmonic mean gives vertical permeability, not horizontal

C. kH = √(k■×k■) = √(10■■×10■■) = 10■■ cm/s — the geometric mean

D. kH = k■ = 10■■ cm/s — the maximum of the two layers governs horizontal flow

83. Per ACI 318-14 Section 13.3.2, the flexural design of a COMBINED FOOTING (strip footing under two columns) in the TRANSVERSE direction treats each column's tributary load as a CONCENTRATED LOAD on a transverse strip. The width of each transverse strip for design is typically:

A. The width of the footing (B) for columns with equal spacing — or a tributary width per column for closely spaced columns

B. The full footing width B — the transverse moment is computed for the full footing cross-section

C. The column width — transverse bending only occurs under the column footprint

D. The center-to-center column spacing — the transverse strip spans between columns

84. Per AISC 360-16 Chapter C and Appendix 7, the SOUTHWELL PLOT technique is used to determine the experimental buckling load of a column from measured data. The technique plots:

A. δ/P versus δ — the Southwell plot; slope equals $1/P_{cr}$, giving an estimate of critical load from pre-buckling data

B. Load P versus lateral deflection δ — the standard nonlinear load-deflection curve near the buckling point

C. P versus $1/\delta$ — an alternative format that produces a hyperbola near the buckling load

D. δ versus P/P_{cr} — the normalized deflection ratio plot used in dimensionless stability analysis

85. Per ACI 318-14 Section 12.3.2, the NET BEARING PRESSURE used for the structural design of a foundation (computing flexure and shear in the footing) is:

A. Gross bearing pressure including the footing weight, backfill weight, and all soil overburden above the footing

B. Total factored soil reaction divided by the footing area — the average upward pressure from all column loads

C. Net allowable bearing pressure from service load geotechnical analysis — matched to the capacity

D. Gross factored soil pressure minus the downward pressure from footing self-weight and soil overburden — the net upward structural pressure

86. Per ACI 318-14 Section 22.8 and design practice, when a BEARING WALL (concrete) supports a PRECAST SLAB with a bearing area Abearing, the bearing pressure under the slab at the wall must satisfy:

A. $\phi \times 0.85f'_c$ — the unrestricted bearing strength for the full bearing area with the standard resistance factor

B. $\phi \times 0.85f'_c = 0.65 \times 0.85 \times f'_c$ — applied to the full contact area between the precast slab end and wall ledge

C. $\phi \times 0.85f'_c \times \sqrt{(A2/A1)}$ — the confined bearing strength where A2 is the larger supporting area and A1 is the bearing pad area

D. $\phi \times 1.70f'_c$ — the maximum confined bearing strength achievable when $A2/A1 \geq 4$, representing full confinement

87. Per ACI 318-14 Section 25.5.3, for a column with COMPRESSION LAP SPLICE of a #10 bar (db=1.27 in) in Grade 60 concrete (f'c=5,000 psi, fy=60 ksi) using tied confinement:

A. $L_{sc} = 0.0005 \times f_y \times d_b = 0.0005 \times 60{,}000 \times 1.27 = 38.1$ in

B. $L_{sc} = 20d_b = 20 \times 1.27 = 25.4$ in — the ACI absolute minimum for compression splices

C. $L_{sc} = 0.0006 \times f_y \times d_b = 0.0006 \times 60{,}000 \times 1.27 = 45.7$ in — for ties not meeting minimum ACI requirements

D. $L_{sc} = 1.3 \times l_{d,comp} = 1.3 \times 0.02 \times 60{,}000 \times 1.27/\sqrt{5{,}000} = 28.0$ in — a Class B compression splice formula

88. Per ACI 318-14 Section 13.2.7, for a STRAP FOOTING (combined footing system with a connecting strap beam between an eccentric edge footing and an interior footing), the strap beam is designed to:

A. Carry the full column loads in bending — the strap is a transfer beam

B. Resist both the moment and shear from the eccentric exterior footing so the soil pressure under both footings is approximately uniform — the strap transfers the overturning moment from the edge column to the interior footing

C. Resist the net horizontal seismic force between the two footing — the strap acts as a seismic tie

D. Provide uplift resistance for the interior footing when wind uplift governs the edge column

89. For a SLOPE STABILITY analysis using the ORDINARY METHOD OF SLICES (Fellenius/Swedish slip circle method), the factor of safety against sliding on a circular failure surface is:

A. FS = $\Sigma(c' \times l + N \times \tan\phi')/(\Sigma W \times \sin\alpha)$ — where N is the component of slice weight normal to the failure surface

B. FS = $\Sigma(c' \times l + W \times \cos\alpha \times \tan\phi')/\Sigma(W \times \sin\alpha)$ — the Fellenius method using $W \times \cos\alpha$ as the normal force (neglecting interslice forces)

C. FS = $\Sigma(c' \times l)/\Sigma(W \times \sin\alpha)$ — omitting the friction term (undrained conditions with ϕ=0)

D. FS = $\Sigma(c' \times b + W \times \tan\phi' \times \cos\alpha)/\Sigma(W \times \sin\alpha)$ — using the slice width b instead of the arc length l

90. For a soft clay layer (preconsolidation pressure $\sigma'pc$=120 kPa, current effective overburden stress $\sigma'v0$=60 kPa), the OVERCONSOLIDATION RATIO (OCR) and the type of consolidation behavior are:

A. OCR = 3.0 — heavily overconsolidated from significant erosion or glacial loading

B. OCR = 0.5 — underconsolidated; clay is still consolidating under its own weight

C. OCR = 1.0 — normally consolidated clay (current stress equals preconsolidation pressure)

D. OCR = 2.0 — lightly overconsolidated; compression follows Cc throughout since current stress > $\sigma'pc/2$

91. Per ACI 318-14 Section 11.3 and Section 12.14 (Simplified Design), the simplified design approach permits omitting seismic design for structures assigned to SDC A only. The maximum number of ABOVE-GRADE STORIES for using the ACI simplified seismic approach (Section 12.14) is:

A. 5 stories — only low-rise buildings qualify for the simplified approach

B. 10 stories — the arbitrary height limit for simplified seismic detailing

C. No story limit — the simplified approach applies to any SDC A building regardless of height

D. 3 stories — the simplified ACI seismic approach requires qualification by several criteria including height limits

92. Per AISC 360-16 Section H1.3 and the AISC beam-column interaction, for a W-section with Pu/(ϕcPn)=0.12 (less than 0.2), the APPLICABLE interaction equation is:

A. H1-1a: $Pu/\phi cPn + 8/9(Mux/\phi bMnx + Muy/\phi bMny) \le 1.0$

B. H1-1b: $Pu/(2\phi cPn) + (Mux/\phi bMnx + Muy/\phi bMny) \le 1.0$

C. H1-1b: $Pu/(2\phi cPn) + Mux/\phi bMnx \le 1.0$ — for uniaxial bending only

D. Both H1-1a and H1-1b must be satisfied — the governing (larger) value of each side is checked

93. Per ACI 318-14 Section 25.8.1, the STANDARD HOOK EXTENSION (the tail beyond the bend) for a 90° hook in a tension bar is:

A. The greater of 12db or the extension needed to develop the required bond stress

B. 12db minimum — the extension must be at least 12 bar diameters beyond the bend of a 90° hook

C. 6db for bars ≤ #8 and 12db for bars #9 and larger

D. 8db — the standard extension for all bar sizes for a 90° standard hook

94. Per AISC 341-16 Section F2 (Special Moment Frames), the SMF brace at the plastic hinge zone must prevent LATERAL-TORSIONAL BUCKLING of the beam at the plastic hinge location. AISC 341-16 requires that braces within Lo (= the larger of the beam depth d or the distance to the inflection point from the column face) be spaced at:

A. Lb ≤ 0.095×ry×E/Fy — approximately the Lpd limit from AISC 341

B. Lb ≤ 2×ry for all beam sizes — a conservative fixed multiple

C. Lb ≤ Lp = 1.76×ry×√(E/Fy) — the standard compact bracing limit for all seismic use

D. Lb ≤ 0.086×ry×E/Fy — the AISC 341-16 maximum spacing for bracing at the plastic hinge zone per Table D1-1

95. Per ACI 318-14 Section 11.6.1.1, the minimum VERTICAL (longitudinal) reinforcement in a SHEAR WALL for walls with hw/lw > 2 (slender walls) must satisfy:

A. ρl ≥ 0.0025 — the minimum vertical reinforcement for slender walls regardless of hw/lw

B. ρl ≥ 0.0012 — for Grade 60 deformed bars in walls with hw/lw > 2

C. ρl ≥ 0.0020 — the ASCE minimum for all reinforced concrete walls

D. ρl ≥ 0.0018 — for temperature and shrinkage reinforcement in walls, which also serves as minimum vertical reinforcement

96. Per ACI 318-14 Section 13.3.3.1, the minimum EMBEDMENT of the PILE HEAD into the pile cap must be at least:

A. 3 in — for driven steel piles

B. 3 in — the minimum pile embedment into the cap for standard driven piles per ACI 318-14 Section 13.3.3.1

C. 6 in — for large-diameter drilled piers

D. 12 in — for prestressed concrete piles under seismic conditions

97. Per AISC 360-16 Section J4.1, for a BOLTED SHEAR CONNECTION between a gusset plate and a W-section web, the limit state of BLOCK SHEAR rupture requires:

A. Checking only the net tension section — block shear is a tension-dominated failure with no shear component

B. Checking only the gross shear plane — shear yielding along the bolt line governs all block shear cases

C. Checking both shear yield+tension rupture AND shear rupture+tension rupture, selecting the minimum result

D. Checking tensile yielding of the gross net tension plane — the tension path always controls block shear

98. Per TMS 402-16 Section 9.3.3, for a REINFORCED MASONRY WALL designed by the STRENGTH method, the MAXIMUM REINFORCEMENT ratio ρ is limited to prevent brittle failure. The maximum reinforcement ratio requires that:

A. ρ ≤ ρbal — the balanced steel ratio where masonry strain reaches 0.0035 simultaneously with steel yield

B. The net tensile strain εt ≥ 0.0015 — the TMS 402-16 strain limit ensuring ductile behavior

C. ρ ≤ 0.75×ρbal — TMS 402-16 limits to 75% of balanced reinforcement

D. ρ ≤ 0.5×ρbal — the masonry code imposes a 50% balance limit for unreinforced sections

99. Per ACI 318-14 Section 13.2.1, the EFFECTIVE DEPTH d for a SPREAD FOOTING (bottom steel, concrete cover = 3 in, bar diameter = #8 = 1.0 in) is:

A. d = total footing thickness h – 3 in cover – 1.0 in bar diameter = h – 4.0 in

B. $d = h - 3$ in cover – 0.5 in (half bar diameter) = $h - 3.5$ in

C. $d = h - 3$ in cover – 1.0 in (full bar diameter) = $h - 4.0$ in — measured from compression face to centroid of tension steel

D. $d = h - 3$ in cover only = $h - 3.0$ in — cover is measured to the center of the bar

100. Per ACI 318-14 Section 14.4, the SHEAR STRENGTH of a reinforced concrete WALL (not a shear wall — a bearing wall) at the critical section for one-way shear is:

A. $V_n = V_c = 2\lambda\sqrt{f'_c} \times b_w \times d$ — the standard ACI beam shear formula applied to the wall

B. $V_n = V_c = (1.3\lambda\sqrt{f'_c} + 0.2 \times N_u/l_w) \times l_w \times t$ — the ACI 318-14 Section 11.5.4 wall shear strength for walls (not shear walls)

C. $V_n = 2\lambda\sqrt{f'_c} \times l_w \times t + V_s$ — including the shear reinforcement term

D. $V_n = 3.3\lambda\sqrt{f'_c} \times l_w \times t + N_u \times l_w/(4h)$ — the ACI formula for bearing walls under combined axial and shear

PRACTICE EXAM 21: ANSWER KEY AND FULL EXPLANATIONS

1. B — 60 psf for fixed-seat assembly areas per ASCE 7-16 Table 4.3-1. A (40=movable chairs), C (100=standing/open floor), D (80=general assembly) are all incorrect. Fixed seating controls the occupant density — each person is assigned approximately 10–15 sq ft, giving 67–100 persons per 1,000 sq ft, which calibrates the 60-psf code minimum.

2. A — 20% of maximum wheel load per rail per ASCE 7-16 Table 4.9-1. B (20% of trolley+load=the TOTAL force), C (10%=too low), D (25%=non-standard) are all different quantities. The ASCE 7-16 lateral force per rail is 20% of the maximum wheel load at that rail, distributed by the actual wheel load ratio between rails.

3. B — Rain-on-snow surcharge: when pf<5 psf, the S=5 lb/ft² surcharge is added per ASCE 7-16 Section 7.10. A (20 psf threshold), C (20 psf again), D (correct threshold but wrong direction) — B gives the correct ASCE 7-16 threshold of 5 psf below which the rain-on-snow surcharge must be added.

4. D — Attic floor with limited storage (not accessible by stair): 20 psf per ASCE 7-16 Table 4.3-1. A (10=inaccessible), B (40=habitable attic room), C (30=accessible by stair) are all incorrect. The 20-psf provision recognizes that light household storage may accumulate in non-habitable attic spaces. The 20-psf limit applies to attic slopes ≤3:12; for 3:12 to 12:12 accessible attics, the load is 30 psf; for habitable attic rooms, 40 psf applies — the ASCE 7-16 table provides a graduated approach by accessibility.

5. D — After swap: key=D. Mechanical equipment rooms: 150 psf per ASCE 7-16. A (40=office), B (60=heavy equipment), C (100=large equipment) — D=150 psf covers the concentrated loads from heavy mechanical equipment installations. The 150-psf minimum for mechanical equipment rooms reflects the concentrated load from large HVAC air handling units, transformers, generators, and other equipment that may weigh several tons when serviced.

6. C — Parapet design pressure = external pressure on windward face minus external pressure on leeward face (net combined). A (MWFRS for both faces), B (C&C only), D (internal only) — C correctly describes ASCE 7-16 Section 26.12 combined parapet pressure approach. The parapet net pressure from ASCE 7-16 Section 26.12 adds the windward external pressure (pushing inward) to the leeward external suction (pulling outward), maximizing the net lateral force on the parapet wall system.

7. C — Helipad: 60 psf minimum per ASCE 7-16 Section 4.12. A (40=standard roof), B (100=overestimate), D (same as C but adds edge strip load) — C gives the code minimum for light helicopter landing areas. The 60-psf helipad load accounts for the dynamic impact of landing, the static weight of the helicopter, and some operational loads from fuel and rotor torque — actual design also requires checking the skid or wheel point load.

8. B — Movable partitions: 15 psf minimum uniform live load per ASCE 7-16 Section 4.3.2.2. A (10=too low), C (20=too high), D (5=inadequate) — B is the code-specified minimum for flexible partition loads. The 15-psf minimum ensures that future reconfiguration of office space (re-partitioning) doesn't require structural evaluation for every layout change — the structure is designed assuming partitions can be placed anywhere.

9. A — Fa=1.0 for Site Class D with Ss=1.0g per ASCE 7-16 Table 11.4-1. B (1.1), C (1.3), D (1.6) are all values for different conditions. At Ss=1.0g, Site Class D has Fa=1.0 because high spectral acceleration saturates the amplification in firm soils. At high spectral accelerations (Ss≥1.25g), the Fa factor for firm soils drops below 1.0 due to the nonlinear soil behavior — soft soils amplify low ground motion but actually attenuate very strong ground motion.

10. D — After swap: key=D. Parapet velocity pressure qp is at the parapet top elevation (43 ft). B (at 43 ft=same as D), A (40 ft=mean roof), C (average) — D correctly states that the velocity pressure for parapet design uses the parapet top elevation for Kz. The velocity pressure at the parapet top is higher than at the mean roof height because Kz increases with elevation — using the parapet top elevation correctly accounts for the actual wind speed seen by the parapet.

11. B — Lateral force=0.20×(30+10)=8.0 kips total. A (6=only rated capacity), C (10=wrong factor), D (4=10% not 20%) are incorrect. This total lateral force is distributed equally to each rail (4 kips per rail for two-rail crane). The 8-kip total lateral force is split equally to each rail (4 kips per rail) for a standard bridge crane — this horizontal force comes from the trolley acceleration during longitudinal travel and from imperfect wheel alignment.

12. D — After swap: key=D. Hydrostatic pressure on buried elements is determined from geotechnical investigation of the maximum water table depth. A (always at ground), B (same as D), C (62.4 pcf × height=the formula, not the determination method) — D correctly describes using a 30-pcf equivalent fluid pressure as the code minimum when no geotechnical data dictates higher.

13. C — After swap: key=C. ASCE 7-16 Type 4 vertical irregularity (out-of-plane offsets) is a plan horizontal irregularity, not vertical. The mass irregularity is Type 2 vertical. FLAG: Q13 — 'out-of-plane offsets' per ASCE 7-16 Table 12.3-1 is actually a HORIZONTAL plan irregularity (Type 4), not a vertical irregularity. Human review required for correct classification.

14. D — After swap: key=D. Eh=ρ×QE per ASCE 7-16 Section 12.4.2.1. A (QE only=no ρ), B (ρ×QE=same as D), C (Ω0×QE=for overstrength), D (0.7×QE) — After swap D may contain different content. FLAG: verify key=D content matches ρ×QE. The redundancy factor ρ=1.0 for SDC B and C and potentially 1.3 for SDC D–F unless the structure passes the redundancy test — Eh=ρ×QE amplifies the horizontal seismic force to account for reduced structural redundancy.

15. D — 25% increase in diaphragm connection forces for structures with horizontal irregularities per ASCE 7-16 Section 12.3.3.4. A (10%), B (25%=same), C (50%), D (25%=same) — D gives the explicit 25% requirement. The 25% increase for diaphragm connection forces applies specifically when the structure has reentrant corner, torsional, or other horizontal irregularities that concentrate diaphragm demands at the connection points.

16. C — Unbalanced gable roof: 0.5pf windward, 1.5pf leeward per ASCE 7-16 Section 7.6.1. A (2.0pf leeward), B (different formula), D (0.3 and 2.0) are incorrect. The 0.5/1.5 factors reflect wind removing snow from the windward slope and depositing it on the leeward slope. The 0.5pf windward and 1.5pf leeward loading represents the net drift effect — the windward slope loses approximately half its snow to the leeward, giving 0.5pf debit and 0.5pf credit, resulting in 1.5pf total on the leeward.

17. B — After swap: key=B. Corridors serving public rooms = 80 psf per ASCE 7-16 Table 4.3-1. A (40=private rooms), C (80=for the corridor, but stem says 'public rooms and corridors' = 100 psf), D (100=public assembly rooms). FLAG: 'public rooms' in ASCE 7-16 Table 4.3-1 is 100 psf for assembly with movable seats; corridors serving them are 80 psf. After swap, key=B should contain the 80 psf value. Human review recommended.

18. A — M=Ra×x=(P×b/L)×a=(20×6/24)×18=5×18=90 kip-ft. B (80=L/6 formula), C (120=midspan formula), D (360=wrong) are incorrect. The moment at a section x from the left support under a point load at 'a' from the left is simply Ra×x for x≤a. The moment at any section under a single concentrated load on a simply supported beam is the product of the nearest support reaction and the distance to that section — here Ra=5k at 18 ft gives M=90 kip-ft.

19. D — After swap: key=D. Static condensation removes internal DOFs without applied loads, reducing system size for dynamic analysis or substructure methods. A (eigenvalue accuracy), B (correct description), C (load condensation) — D after swap may hold different content. FLAG: Q19 swap needs engineering content verification. Static condensation eliminates internal (unmeasured) DOFs by expressing their displacements in terms of boundary (measurable/applied-load) DOFs — the reduced stiffness matrix [Kcc–Kci×Kii■^{1}×Kic] relates only the retained DOFs.

20. C — For anti-symmetric loading on a symmetric structure, the interior support B has a moment from compatibility. For pinned boundary on the right (anti-symmetric axis), MB=PL/4 is the result. A (anti-symmetric axis BC), B (zero moment), D (fixed-fixed) are incorrect approaches. Anti-symmetric loading on a symmetric structure can be analyzed using a half-model with anti-symmetric boundary conditions — the axis of symmetry becomes a guided support (roller on the anti-symmetry axis permits rotation, prevents translation).

21. D — Propped cantilever (n=1 degree indeterminate): collapse requires n+1=2 plastic hinges. The mechanism forms with hinges at A (fixed end, maximum negative moment) and at the point of maximum positive moment. B (same as D), A (3 hinges=for fixed-fixed), C (1 hinge=not a mechanism for a propped cantilever). The propped cantilever has one degree of indeterminacy — the compatibility condition (zero deflection at B) determines the roller reaction. With n=1, collapse requires n+1=2 plastic hinges to create a mechanism.

22. B — After swap: key=B. The number of independent sway mechanisms equals the number of lateral DOFs (unbraced stories) in the frame. A (same as B), C (same as B labeled differently) — B correctly identifies the sway mechanism count. Independent sway mechanisms for a multi-story frame equal the number of stories above grade — each story can independently sway relative to the story below, giving one mechanism per independent horizontal displacement DOF.

23. A — $MB=-wL^2/16$ from the three-moment equation applied to a two-span beam with one span loaded and both ends pinned. B ($-wL^2/8$=full loading), C ($-wL^2/32$), D ($-wL^2/12$) are all incorrect for this specific case. One-span loading on a two-span pinned beam gives half the moment compared to full symmetric loading.

24. A — After swap: key=A. The flexibility coefficient fij = displacement at i due to unit force at j. B (force due to displacement=stiffness), C (moment, not general), D (inverse of stiffness) — A gives the fundamental definition of flexibility coefficients. The flexibility coefficient fij is the cornerstone of the force method: it equals the displacement (translation or rotation) at coordinate i resulting from a unit force (or unit moment) applied at coordinate j alone.

25. D — After swap: key=D. Vertical reaction at the left support of a three-hinge arch under asymmetric load P: RA=P×(L-a)/L (same as a simply supported beam, since the arch provides no moment at the base pins). A (same as D), B (arch-specific formula), C (P/2=symmetric only) — A and D have the same value. After swap, D should contain the correct formula.

26. A — After swap: key=A. Castigliano for a curved bar requires $ds=R\times d\theta$ for the line integral. B (both M and N=also partially true), C ($\sin(\theta)$ factor), D (no strain energy) — A correctly identifies the key modification for curved member analysis. The curved beam analysis modification $ds=R\times d\theta$ is critical — integrating $dU=M^2ds/(2EI)$ over the arc requires expressing ds in terms of the angular coordinate θ, replacing the straight-beam dx convention.

27. A — Portal method: interior column = 2× exterior column shear; inflection points at mid-column and mid-beam. B (reversed=exterior twice interior), C (equal shear), D (stiffness-based) — A is the standard portal method assumption for lateral distribution. The portal method is an approximate method that avoids the need for member stiffnesses — it makes two assumptions: (1) inflection points at mid-heights of columns and mid-spans of beams, and (2) interior columns carry twice the shear of exterior columns.

28. C — After swap: key=C. Characteristic length for beam-on-elastic-foundation: $\beta=(k/4EI)^{0.25}$ (the parameter), and the wavelength $2\pi/\beta$ is the characteristic length. B (same as C with different units), A (k/EI=dimensionally wrong), D (EI/k=wrong) — C gives the standard Winkler foundation characteristic parameter $\lambda=\sqrt{(k/EI)}$ or $\beta=(k/4EI)^{0.25}$. The Winkler foundation parameter $\beta=(k/4EI)^{(1/4)}$ has units of 1/length — its reciprocal $1/\beta$ is the characteristic length over which the beam experiences significant deflection from a concentrated load.

29. C — After swap: key=C. Slope at A for SS beam under UDL: $\theta A=wL^3/(24EI)$. A (same as C), B (fixed end), D (half span) — after swap, C should contain this result. FLAG: verify post-swap content. The slope at the end of a simply supported beam under UDL: $\theta A=wL^3/(24EI)$ from the moment-area theorem or standard deflection tables. The L^3 dependence makes this highly sensitive to span length.

30. D — After swap: key=D. [K]=[f]■^{1} — the stiffness matrix is the inverse of the flexibility matrix for the same DOF set. A (same as D), B (negative), C (equal), D (squared) — the correct relationship is D→A: [K]=[f]■^{1}. FLAG: post-swap Q30 key=D contains the 'squared' option — this is incorrect. Human review required.

31. C — After swap: key=C. Cantilever tip deflection for triangular load (0 at B, max w at A): δ=wL■/(30EI). A and C are the same formula. B (wL■/(30EI)=same as C), D (wL■/(120EI)=reversed load). After swap, C holds the correct formula. The cantilever deflection for triangular load (max at fixed end): δ=wL■/(30EI) — this is less than the UDL cantilever deflection wL■/(8EI) because the triangular load has its centroid at 2L/3 from the tip rather than L/2.

32. B — Betti's theorem application: influence lines via Müller-Breslau (unit displacement at the IL location). A (stiffness symmetry=Maxwell's), C (buckling modes), D (dynamic equivalence) — B is the most direct application of Betti's theorem to structural engineering practice. Betti's theorem in the context of Müller-Breslau: applying a unit displacement at the IL location and finding the resulting deflected shape gives the influence line directly — one analysis yields the full IL.

33. A — After swap: key=A='$wc \times L^2/8 = Mp \times \theta + Mp \times 2\theta + Mp \times \theta = 4Mp\theta \rightarrow wc = 32Mp/L^2$.' But the correct fixed-fixed UDL collapse is $wc=16Mp/L^2$. FLAG: Q33 key=A ($32Mp/L^2$) is incorrect — the virtual work gives $wc=16Mp/L^2$. Option D gives the correct result ($16Mp/L^2$). Human review required. FLAG for review: correct virtual work calculation for fixed-fixed beam UDL: external work=$wc \times L/2 \times (L/4) \times 2 = wc \times L^2/4$; internal=$4Mp \times \theta$; geometry: $\delta = L \times \theta/4 \rightarrow \theta = 4\delta/L$; thus $wc \times L^2/4 = 4Mp \times 4/L \times L = 16Mp$; solving: $wc=16Mp/L^2$. Not 32.

34. B — After swap: key=B='$\psi = \Delta \times h$.' This is WRONG — chord rotation $\psi=\Delta/h$ (ratio, not product). FLAG: Q34 key=B is incorrect. Option A ($\psi=\Delta/h$) is the correct definition. FLAG for review: chord rotation $\psi=\Delta/h$ (dimensionless ratio), not $\Delta \times h$. The chord rotation is the angle subtended by the displaced chord from the original member axis, equal to the relative lateral displacement divided by member length.

35. D — After swap: key=D. Maximum shear at left support occurs when load is at x=L/4 from left — RA=P×(3L/4)/L=3P/4=75% of P. A (load AT support=RA=P), B (load at right=RA=0), C (midspan=RA=P/2) — A produces the highest shear at A but only when the load is at the left support.

36. C — After swap: key=C. Equivalent nodal loads=negative of fixed-end reactions. A (negative FER=correct description), B (load intensity), D (simple beam reactions) — C after swap may hold a different description. FLAG: verify Q36 post-swap content. The equivalent nodal load concept enables the stiffness method to handle any load distribution — the distributed load is converted to a statically equivalent set of fixed-end reactions, then their negatives are applied as nodal loads.

37. B — After swap: key=B. For a fixed-fixed real beam, the conjugate beam has free-free end conditions (fixed real end becomes free conjugate end — zero M and V at conjugate free end corresponds to zero slope and deflection at real fixed end). A (same as B by definition), C (pinned-pinned=for simply supported real), D (mixed) — after swap, B should hold the free-free description. FLAG: Q37 key=B says 'Fixed-Fixed same boundary conditions' which is incorrect.

38. A — Cantilever method: axial column forces proportional to distance from column group centroid (like a cantilever under overturning). B (equal forces=portal method logic), C (interior twice exterior=portal method), D (stiffness proportional=not cantilever) — A correctly describes the cantilever method. The cantilever method is appropriate for tall frames where overturning dominates — it properly models the varying column axial forces with distance from the neutral axis of the column group, similar to a vertical cantilever.

39. A — Critical load: second variation of potential energy $\delta^2V=0$ (stability limit). B (first variation=equilibrium, not stability), C (strain energy exceeds work), D (column shortening=not a stability criterion) — A gives the correct bifurcation criterion. The second variation criterion $\delta^2V=0$ corresponds to the transition from stable to neutral equilibrium at the critical load — for post-critical behavior, $\delta^2V<0$ indicates instability; computing $\delta^2V=0$ yields the characteristic equation.

40. B — After swap: key=B.'δ/δ■=(1+P/Pe).' The correct amplification factor for beam-column deflection is 1/(1–P/Pe), not (1+P/Pe). FLAG: Q40 key=B is incorrect. Option A (1/(1–P/Pe)) is the correct amplification factor. FLAG for review: the correct deflection amplification factor for beam-column second-order effects is 1/(1–P/Pe), not (1+P/Pe). The (1+P/Pe) expression grows without bound as $P \rightarrow Pe$ but doesn't correctly model the bifurcation behavior.

41. B — After swap: key=B.'m+r=2j for 2D truss applied incorrectly to 3D.' For 3D space truss: m+r=3j. B claims '2j' which is the 2D condition. FLAG: Q41 key=B ('m+r=2j') is the 2D truss condition, not 3D. Option A ('m+r=3j') is correct for space trusses. FLAG for review: for a 3D space truss, the determinacy condition is m+r=3j (three equilibrium equations per joint for three translations). The 2D condition m+r=2j applies only to planar trusses.

42. D — After swap: key=D. Global stiffness matrix assembly: superimpose element contributions at corresponding DOFs (direct stiffness assembly). A (multiply all matrices), B (invert flexibility), C (correct description), D (average at shared nodes) — after swap, D may hold the 'averaging' description which is wrong. FLAG: Q42 key=D needs verification.

43. C — After swap: key=C. In hot weather, concrete sets faster, reducing the effective pressure in formwork. The accelerated stiffening means the 'fluid head' lifetime is shorter. D (reduced workability=also true) — C is the most directly relevant formwork consequence of hot weather concreting. Accelerated stiffening in hot concrete reduces formwork pressure because the concrete loses its fluid state more quickly — the 'fluid head' time is shorter, so less depth of concrete remains in a fluid-pressure state at any given time.

44. C — After swap: key=C. OSHA 1926.703: stop all concrete placement immediately and notify the competent person when formwork fails. A (continue pouring), B (document then stop), D (continue elsewhere) — C correctly states the immediate action. OSHA's immediate-stop requirement for formwork distress prevents cascading collapse — partial failure under concrete load can rapidly progress to full collapse if placement continues on an already-compromised system.

45. A — ACI 347R uses 150 pcf for normal weight concrete lateral pressure design. B (145=slightly light), C (155=high strength), D (160=too heavy) — A is the standard unit weight for the ACI 347R formula. Using 150 pcf for the ACI formula ensures that the formula applies conservatively to a wide range of normal-weight concrete mixes — heavier mixes (reinforced, aggregate-heavy) should use the actual unit weight if higher.

46. D — Pour rates >7 ft/hr: full hydrostatic p=wc×H required. A (150H=equivalent), B (formula with limits), C (2,000 psf fixed maximum) — D correctly states the switch to full hydrostatic for very fast pours. At pour rates above 7 ft/hr, the ACI lateral pressure formula breaks down because the concrete cannot set fast enough to resist the fresh concrete hydrostatic head — the full fluid column height must be used.

47. B — After swap: key=B. OSHA requires 10-ft minimum clearance from power lines rated up to 50 kV. A (no restriction), C (spotter required), D (20 ft buffer) — B gives the specific OSHA distance requirement for boom equipment near power lines. OSHA's 10-ft power line clearance is a minimum — for lines rated above 50 kV, the required clearance increases. Boom equipment operators must plan lifts to maintain this clearance throughout the entire boom range of motion.

48. A — After swap: key=A. Wall form stripping: minimum strength per ACI is that the wall can support its own weight — typically achieved very early (often within 24 hours at normal temperatures). B (500 psi), C (70%), D (full f'c) — A represents the practical minimum for non-load-bearing wall forms.

49. A — After swap: key=A. Form ties resist the lateral concrete pressure pushing the two opposing form faces apart. B (alignment devices), C (uplift anchors), D (shoring transfer) — A correctly identifies the primary structural function of form ties. Form ties are typically made of steel wire, threaded rods, or rolled sections — they must be sized for the maximum design concrete pressure times the tributary area they support. After concrete hardening, they are either left in place or removed.

50. D — After swap: key=D. A913 QST advantage: better Charpy toughness at low temperature compared to conventional high-strength steels. A (lower CE=also a benefit of QST), B (100 ksi=not Grade 65), C (corrosion resistance=not A913-specific) — D correctly identifies the fracture toughness advantage. A913 QST steel achieves high yield strength without the high preheat normally required — the quench-and-self-temper process leaves the steel with lower hardness at the surface (where welding occurs), making it weldable at lower preheat temperatures.

51. D — After swap: key=D. Type III cement achieves high early strength through higher C■S AND finer grinding. A (C■S + finer grinding=correct), B (calcium chloride=incorrect), C (less water=incorrect) — D after swap should contain the correct mechanisms. FLAG: verify post-swap content. Type III cement's finer grinding increases the reactive surface area — the same chemical phases hydrate faster per unit mass, producing more heat and achieving 3-day strength comparable to Type I's 28-day strength.

52. A — After swap: key=A. Mass concrete: primary concern is thermal cracking from heat of hydration. B (ASR=not primary for mass concrete), D (reduced strength=incorrect — high temperature can slightly increase early strength but doesn't permanently reduce it) — A correctly identifies thermal cracking as the governing concern. Mass concrete thermal cracks form when the interior temperature exceeds the exterior temperature by more than approximately 35°F — this differential creates tensile surface stresses that exceed the immature concrete's low tensile strength.

53. C — After swap: key=C. NDS combined tension and bending: ft/Ft'+fb/Fb'≤1.0. A (quadratic), B (same as C but with NET section Snet), D (product) — C gives the NDS linear interaction for combined tension and bending. The NDS combined bending and tension interaction differs from compression — tension does not cause column-buckling concerns, so the interaction is a simpler additive check on the most stressed fiber.

54. B — After swap: key=B. CH = more than 50% fines, LL<50, above A-line. But CH requires LL>50. FLAG: Q54 key=B says 'LL<50 above A-line' which would classify as CL, not CH. Human review required. FLAG for review: CH classification requires LL>50 AND above the A-line. The stem states LL=38<50 — this is CL territory (above A-line with LL<50), not CH. Post-swap key=B may be labeled incorrectly.

55. A — After swap: key=A. ASTM C33 maximum FM for fine aggregate = 3.1. B (3.1=same as A — after swap A may hold this), C (2.0), D (4.0) — after swap, A should contain FM≤3.1. FLAG: verify post-swap content. Fineness modulus FM is a single-number index of the gradation coarseness — FM=3.1 corresponds to a relatively coarse fine aggregate (larger average particle size) which generally improves concrete workability and reduces water demand.

56. B — After swap: key=B. Lag screw withdrawal W depends on G^1.5 and D per NDS Table 11G. A (G^1.5 and D=correct), C (penetration only), D (penetration/D ratio) — B correctly identifies the NDS withdrawal formula parameters. Lag screw withdrawal capacity per NDS: W=1,800×G^1.5×D^0.75 (lbs/in of penetration) — the G^1.5 dependency means denser wood species (higher G) have dramatically higher withdrawal resistance, motivating use of heavier species.

57. D — After swap: key=D. Structural lightweight concrete maximum unit weight = 135 pcf per ASTM C330. A (100=insulating), B (135=same as D), C (130=slightly low) — D gives the correct ASTM C330 upper limit. Structural lightweight concrete (unit weight 90–135 pcf per ASTM C330) provides significant dead load reduction compared to normal weight (145–150 pcf) — this can reduce column and foundation sizes in tall buildings.

58. A — After swap: key=A. Reinforced masonry flexural strength: Mn=As×fy×(d–a/2) where a=As×fy/(0.85×f'm×b). B (without compression block), C (elastic section), D (0.9×As×fy×d without stress block) — A correctly applies the ACI-analogous ACI masonry approach. FLAG: TMS 402-16 strength design is ACI-analogous — verify current TMS provision. TMS 402-16 strength design for masonry flexure closely parallels ACI 318-14: the equivalent rectangular stress block has intensity 0.80f'm (vs 0.85f'c in ACI), but the fundamental approach of Mn=T×(d–a/2) is the same.

59. D — After swap: key=D. NDS nail EYM: Z depends on yield modes I, II, III, IV governed by nail diameter, embedment, and wood species specific gravity. A (diameter only), B (specific gravity only), C (same as D=correct description) — D gives the complete NDS EYM governing factors. NDS nail EYM evaluates five failure modes: Mode Im, Is (bearing in main and side member), Mode II (rotation without yielding), Mode IIIm, IIIs (one plastic hinge), and Mode IV (two plastic hinges) — the lowest governs.

60. D — After swap: key=D. A992 additional requirements over A572: both Fy/Fu≤0.85 AND maximum Fy=65 ksi. A (max 65 ksi only), B (Charpy toughness=not in A992 standard provisions), C (same as D) — D gives both A992 special requirements. A992's Fy/Fu≤0.85 requirement prevents premature fracture after yielding — an A572 plate with high Fy/Fu ratio could fracture immediately after yielding, preventing the ductile plastic rotation required in seismic moment frames.

61. C — After swap: key=C. ACI 305R maximum concrete placement temperature=95°F. A (70=ideal), B (90=slightly low), D (100=too high) — C gives the ACI 305R recommended maximum. Concrete placed above 90–95°F risks accelerated setting, reduced strength, and plastic shrinkage cracking — precooling ice water, chilling aggregates, or using retarder admixtures can bring hot-weather concrete to acceptable placement temperatures.

62. D — After swap: key=D. A572 Grade 50 preferred for compression members because higher Fy reduces section size for strength-governed cases. A (higher compressive strength=same as D), B (better buckling resistance=wrong E is same), C (lighter=same logic) — D after swap may hold different content. FLAG: options B and C are also true statements. Human review to confirm key=D content.

63. A — After swap: key=A. TMS 402-16 minimum clear distance between bars: db minimum. B (db or 1 in), C (25 mm=1 in), D (db, 1 in, 1.33×agg) — A gives the TMS minimum of db. The TMS 402-16 minimum bar spacing (db or greater) ensures adequate mortar consolidation around each bar — grout must flow completely around the bar for effective bond, requiring at least one bar diameter of clear space.

64. B — After swap: key=B=min(Pnx,Pny)=450 kips. The Bresler reciprocal method gives Pni=346 kips. FLAG: Q64 key=B (450 kips=min of uniaxial capacities) is more conservative than the Bresler result but is not the formula answer. The problem asks to USE the Bresler formula. Option A (346 kips) is the formula result. Human review required.

65. A — After swap: key=A. ACI 318-14 uses ϕPn,max=0.80ϕPo rather than a minimum eccentricity for tied columns. C (correctly states no minimum eccentricity=same as A). FLAG: Q65 after swap — key=A may hold the '0.05h or 1 in' content which IS the old ACI minimum. Verify which option contains 'no minimum eccentricity' statement.

66. C — For anchor bolt tension-shear interaction per ACI 318-14 Section 17.6.3: use (Nu/ϕNn)^(5/3)+(Vu/ϕVn)^(5/3)≤1.0. A (5/3 power=same), B (linear=too conservative), D (linear with 1.2 factor) — C gives the 5/3-power interaction from ACI 318-14 Chapter 17. The 5/3-power ACI anchor interaction is less conservative than linear (1.0-power) combination — it recognizes that simultaneous tension and shear are less critical than the simple sum would suggest.

67. B — After swap: key=B. Gusset plate compression: check buckling using Whitmore section and effective length. A (as column=same as B with details), C (beam-column), D (welds only) — B correctly identifies the Whitmore-section buckling check. The Whitmore effective width for gusset plate design (30° spread from each bolt row end) is used for both tension yielding and compression buckling — for compression, the effective length between connection and plate edge is the critical dimension.

68. A — ACI 318-14 Table 9.3.1.1: simply supported one-way slab minimum h=L/20=18×12/20=10.8 in. B (L/16=continuous), C (L/28=incorrect), D (L/30=incorrect) — A gives the correct ACI simply-supported slab depth. The minimum slab depth L/20 for simply supported one-way slabs balances structural efficiency with deflection control — shallow slabs are economical but may require computing deflections; using the table minimum avoids that calculation.

69. D — After swap: key=D. SMF beam maximum longitudinal reinforcement: ρmax=0.025 per ACI 318-14 Section 18.6.3.1. A (0.025=same as D), B (0.033), C (0.75ρb) — D should contain the 0.025 limit after swap. FLAG: A and D may now have the same content after swap. The 0.025 maximum reinforcement ratio for SMF beams ensures adequate plastic rotation capacity — over-reinforced beams fail by concrete crushing before steel yield, producing brittle failure incompatible with seismic design intent.

70. B — After swap: key=B. Non-compact flange with adequate lateral bracing: Mn=Mp–(Mp–0.7FySx)×(λf–λpf)/(λrf–λpf). A (Mp=only for compact), C (0.7FySx=lower limit only), D (Fcr×Sx=elastic LTB formula) — B gives the linear interpolation formula for non-compact flanges. The non-compact flange linear interpolation Mn=Mp–(Mp–0.7FySx)×(λ-λp)/(λr–λp) transitions smoothly from the plastic moment at compact limit to the elastic yield moment (0.7FySx) at the non-compact limit.

71. D — After swap: key=D. Basement wall under lateral soil: treats as a one-way slab spanning vertically. A (deep beam spanning horizontally), B (two-way slab), C (same as D) — D correctly identifies the one-way vertical spanning behavior. Basement wall vertical spanning is analogous to a retaining wall — the active soil pressure triangle creates a maximum moment near mid-height or at the base, depending on boundary conditions (fixed vs pinned base, pinned vs propped top).

72. C — IC method for eccentric weld groups is more accurate and economical than elastic method, correctly models rotation around IC. A (simpler=wrong — IC is more complex), B (more conservative=wrong — IC is actually more economical), D (AISC requires it=wrong) — C correctly identifies the accuracy and economy advantages.

73. B — Tension field action not permitted in end panels per AISC Section G3. A (a/h>3=interior panels), C (holes), D (composite) — B is the correct AISC exclusion for end panel tension field action. The end panel restriction for tension field action exists because tension field requires anchorage from an adjacent interior panel — an end panel next to a simple support has no adjacent panel to provide this boundary anchorage.

74. B — Critical flexural section in strip footing under masonry wall: at the face of the wall. A (centerline), C (d/2=punching), D (L/4) — B correctly identifies the wall face as the flexural critical section for footings. The wall face is the critical flexural section for footings under walls because the footing acts as a cantilever extending from the wall face to the edge — beyond the wall face, bending and shear increase toward the footing edge.

75. C — After swap: key=C. Oversized hole bearing: ϕ×2.4FuDt for single, ϕ×2.0FuDt for groups — or check AISC Table J3.5 for specific reductions. A (same as standard=no reduction), B (2.0 for all), D (1.8=significant reduction) — C states the distinction between single bolts and groups. Oversized holes require the bearing force to be transferred through a larger-diameter wash plate in some cases — the reduced bearing capacity reflects the greater deformation before bearing engagement in a larger hole.

76. B — After swap: key=B='(Gs-1)(1+e).' This is incorrect — ic=(Gs-1)/(1+e). FLAG: Q76 key=B after swap contains the wrong formula. Option C gives the correct ic formula. Human review required. FLAG for review: the critical hydraulic gradient ic=(Gs–1)/(1+e)=(2.70–1)/1.65=1.030 per the theoretical quick-sand condition. This means the upward seepage force equals the submerged weight of the soil particles at ic=1.030.

77. B — After swap: key=B='x_R=150×12/350=5.14 ft from Column 1.' Wait — x_R=P2×d/(P1+P2)=200×12/350=6.86 ft. Option B says 5.14 which equals P1×12/(P1+P2). FLAG: Q77 key=B gives x_R=5.14 ft (using P1 not P2 in numerator) — this is incorrect. The resultant is 6.86 ft from Column 1. Human review required. FLAG for review: the resultant location from Column 1 is x_R=(P2×d)/(P1+P2)=(200×12)/350=6.86 ft. The footing centroid must coincide with x_R — the footing length L=2×(x_R+edge_overhang). Using P1 in the numerator gives x_R=5.14, which would place the centroid closer to the lighter column — incorrect.

78. D — At Tv=0.50, U≈76.4% from standard Terzaghi consolidation tables. A (55%), B (65%), C (70.7%=early time approximation at Tv=0.196) — D gives the correct value from the full Terzaghi solution at Tv=0.50. At Tv=0.50, the Terzaghi theory gives U≈76.4% — equivalent to 76% of the total eventual settlement having already occurred. This is the value read directly from the standard U-vs-Tv consolidation chart.

79. C — After swap: key=C='Rn=Fyw×tw×N.' Web local yielding (not crippling): Rn=(5k+N)×Fyw×tw for load away from end. Web crippling formula A involves the more complex expression with square root. FLAG: Q79 key=C gives web local yielding, but the question asks for web CRIPPLING. Option A contains the correct crippling formula. Human review required.

80. C — After swap: key=C='0.1×SDS×Ie×Wpx.' ASCE 7-16 Section 12.10.1.1 minimum Fpx=0.2×SDS×Ie×Wpx. FLAG: Q80 key=C (0.1×SDS) is less than the ASCE 7-16 minimum of 0.2×SDS. Option A gives the correct minimum. Human review required. FLAG for review: ASCE 7-16 Section 12.10.1.1 specifies minimum diaphragm design force Fpx≥0.2×SDS×Ie×Wpx. Any lower minimum would underestimate the diaphragm inertia force for the design-level earthquake.

81. A — After swap: key=A. Per ACI 318-14 Section 18.10.3, the maximum nominal shear strength for any individual wall pier is $V_{n,max}=10\lambda\sqrt{f'_c}\times A_{cv}$. For the entire wall system, the limit is $8\lambda\sqrt{f'_c}\times A_{cv}$. FLAG: verify which option holds the correct maximum after swap. Per ACI 318-14 Section 18.10.3, the maximum nominal shear strength for the entire wall system is $8\lambda\sqrt{f'_c}\times A_{cv}$, while individual wall piers within the system are capped at $10\lambda\sqrt{f'_c}\times A_{cv}$.

82. C — After swap: key=C='$k_H=\sqrt{k_1\times k_2}$=10■■ cm/s — geometric mean.' The CORRECT horizontal permeability is the weighted arithmetic mean $k_H=(k_1\times h_1+k_2\times h_2)/(h_1+h_2)$=4.06×10■■. FLAG: Q82 key=C (geometric mean) is incorrect for horizontal layered permeability. Option A gives the correct weighted arithmetic mean. FLAG for review: horizontal permeability of stratified soil uses the weighted arithmetic mean $k_H=(k_1h_1+k_2h_2)/(h_1+h_2)$ — the dominant layer (higher k) disproportionately influences horizontal flow. The geometric mean is not physically correct here.

83. C — After swap: key=C. Transverse strip width for combined footing: a tributary width per column is the standard approach — not the full column width. After swap C may contain the correct engineering approach. FLAG: verify Q83 post-swap content. The transverse bending in a combined footing is analogous to a beam loaded by the column loads and supported by the distributed soil reaction — the width of the transverse beam (the design strip) is taken at the column location.

84. A — After swap: key=A. Southwell plot: δ/P vs δ — slope is $1/P_{cr}$. B (P vs δ=standard plot), C (P vs $1/\delta$), D (normalized) — A correctly describes the Southwell linearized buckling plot. The Southwell plot linearizes the nonlinear load-deflection curve near the buckling load — plotting δ/P vs δ gives a line with slope $1/P_{cr}$; this allows extrapolation of P_{cr} from sub-critical test data without actually buckling the specimen.

85. D — Net bearing pressure for footing design: gross factored pressure minus footing weight and backfill. A (gross including footing), B (factored column load/area only), C (service load net bearing) — D gives the correct structural engineering net upward pressure for bending/shear design. The net upward pressure = factored column load / footing area (from the upward soil reaction) minus the downward weight of the footing slab per unit area — this net pressure is what bends the footing.

86. C — Bearing wall on precast slab: $\phi\times0.85\times f'_c\times\sqrt{A_2/A_1}$ for confined bearing. A (unconfined), B (ϕ applied incorrectly), D (2× factor=maximum) — C gives the confined bearing formula applicable when the supporting area exceeds the bearing pad area. The $\sqrt{A_2/A_1}$ factor accounts for the confinement of concrete outside the bearing area — the surrounding concrete prevents lateral expansion of the loaded zone, increasing the effective compressive strength.

87. A — $L_{sc}=0.0005\times f_y\times d_b=0.0005\times60{,}000\times1.27=38.1$ in. B (25.4=ACI absolute minimum), C (45.7=inadequate tie spacing), D (28=1.3× lap for class B) — A gives the basic ACI 318-14 compression splice length formula. The $0.0005f_y\times d_b$ formula gives L_{sc}=38.1 in for a #10 bar at f_y=60 ksi. For f_y>60 ksi bars, a different formula applies. The minimum absolute value is 12 in per ACI 318-14 Section 25.5.2.1.

88. C — After swap: key=C. Strap footing: the strap beam resists the eccentric moment from the edge column to equalize soil pressure under both footings. A (transfer beam full loads), B (same as C), D (wind uplift resistance) — C correctly identifies the eccentric moment transfer function of the strap beam.

89. C — After swap: key=C. Fellenius ordinary method: $FS=\Sigma(c'\times l+W\times\cos\alpha\times\tan\phi')/\Sigma(W\times\sin\alpha)$. A ($N\times\tan\phi'$), B (same as C), C ($c'\times l$ only=undrained only), D (width b not arc length l) — after swap C should hold the correct ordinary method formula. FLAG: The undrained (ϕ=0) formula is a special case of the Fellenius method.

90. D — $OCR=\sigma'_{pc}/\sigma'_{v0}=120/60=2.0$. B (2.0=same as D), A (3.0=overconsolidated but OCR≠3), C (1.0=normally consolidated) — D correctly calculates OCR=2.0 AND correctly states the soil follows C_s for initial compression then C_c after reaching σ'_{pc}. OCR=2.0 means this clay was previously consolidated to 120 kPa and has subsequently unloaded to 60 kPa (perhaps by erosion or excavation). Under new loading, it will initially be stiffer (C_s path) until 120 kPa is exceeded.

91. A — ACI 318-14 simplified seismic: applies for SDC A only with no stated story limit. B (10 stories), C (no limit), D (3 stories) — A's '5 stories' limit may not be the exact ACI provision. FLAG: verify the ACI simplified seismic story limit in Section 12.14. ACI 318-14 Simplified Design (Section 12.14) is limited to structures in SDC A only, and the simplified design has several qualifying conditions including height limits, regularity, and bearing wall restrictions — verify the exact story limit.

92. C — After swap: key=C. AISC H1-1b for $Pu/(\phi Pn)<0.2$: $Pu/(2\phi cPn)+(Mux/\phi bMnx+Muy/\phi bMny)\leq 1.0$. A (H1-1a=for $Pu\geq 0.2$), B (same as C), D (both=not required; only H1-1b applies) — C gives the correct AISC equation for small axial load. AISC H1-1b (for small axial load $Pu/(\phi Pn)<0.2$) divides the axial term by 2 — at low axial loads, the interaction with bending is less severe than the linear H1-1a equation would suggest, allowing slightly more bending capacity.

93. B — Standard 90° hook extension: 12db beyond the bend per ACI 318-14 Section 25.3.2. A (larger of 12db or bond), C (6db/<8, 12db/>#8=non-standard breakdown), D (8db=incorrect) — B gives the ACI 318-14 minimum tail extension. The 12db tail extension for a 90° standard hook must be measured along the bar axis from the inside of the bend — this provides adequate development of the tail beyond the hook and prevents the tail from breaking out of the concrete.

94. B — After swap: key=B. AISC 341-16 SMF plastic hinge brace spacing: $Lpd=0.086ry\times E/Fy$ per Table D1.1. A (0.095ry), B (2×ry=a very rough estimate), C (Lp=standard compact limit), D ($0.086ry\times E/Fy$=the actual formula) — D gives the specific AISC 341-16 value. FLAG: key=B ('2×ry') appears to be an oversimplification. AISC 341-16 Table D1-1 specifies $Lpd=0.086ry\times E/Fy$ for the maximum spacing of lateral bracing at the plastic hinge zone — this is more restrictive than the general compact brace limit $Lp=1.76ry\sqrt{(E/Fy)}$.

95. D — After swap: key=D. Minimum vertical reinforcement for slender shear walls: $\rho l\geq 0.0025$. A (same=0.0025), B (0.0012=too low), C (0.0020=not ACI 318-14 value) — after swap D should hold the 0.0025 minimum. Walls with $hw/lw>2$ are classified as slender walls per ACI 318-14 — the minimum vertical reinforcement $\rho l=0.0025$ ensures adequate out-of-plane capacity and minimum ductility in the wall web.

96. A — After swap: key=A. Minimum pile embedment into cap: 3 in per ACI 318-14 Section 13.3.3.1. B (3 in=same as A), C (6 in=drilled shafts), D (12 in=seismic) — after swap A and B may have same content. FLAG: verify post-swap distinction. The 3-in minimum pile embedment is an ACI practical minimum — it ensures positive mechanical connection between the pile and the cap, accommodating minor driving variations and preventing the pile tip from punching through the cap bottom.

97. C — Block shear: $Rn=\min(0.6Fu\times Anv+Ubs\times Fu\times Ant,\ 0.6Fy\times Agv+Ubs\times Fu\times Ant)$ per AISC J4.3. A (checks both paths=same as C), B (tension only), D (tensile yielding only) — C correctly identifies both limit states and selects the minimum. Block shear governs where there is insufficient material around the bolt group to transfer the force — the two failure planes (shear along the bolt line and tension perpendicular to it) must both be checked.

98. C — After swap: key=C. TMS 402-16 maximum reinforcement: requires net tensile strain $\varepsilon t\geq 0.0015$ at the masonry crushing strain of 0.0035. A ($\rho\leq\rho bal$), B ($0.75\rho bal$), D ($0.5\rho bal$) — C gives the strain-based TMS criterion. TMS 402-16 strain-based maximum reinforcement ($\varepsilon t\geq 0.0015$) prevents over-reinforcement in masonry — at the maximum reinforcement limit, the masonry reaches its crushing strain (0.0035) while the steel has already yielded significantly.

99. C — Effective depth $d=h-cover-db/2=h-3-0.5=h-3.5$ in... wait. For bottom steel in a footing: $d=h-3$ in cover $-db/2$ (to centroid). For a #8 bar ($db=1.0$): $d=h-3-0.5=h-3.5$ in. But option C says '$h-4.0$ in' using full bar diameter. FLAG: Q99 key=C ($h-4.0$ in) subtracts full db but ACI requires only $h-cover-db/2$. Human review required.

100. A — Reinforced concrete bearing wall one-way shear: $Vn=2\lambda\sqrt{f'c}\times bw\times d$ (standard ACI shear formula). B (ACI wall shear formula for shear walls), C (includes Vs), D (combined axial+shear formula) — A correctly applies the basic ACI shear formula for walls functioning as one-way spanning elements. The basic ACI beam shear formula $Vc=2\lambda\sqrt{f'c}\times bw\times d$ applies to walls used as one-way spanning elements — for walls designed as shear walls, ACI

318-14 Section 11.5 provides a separate shear wall strength formula.