PE Civil Structural Exam Prep

Practice Exam 16

100-Question Simulation

NCEES PE Civil - Structural Blueprint

Exam Instructions

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

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

PRACTICE EXAM 16 -- QUESTIONS

DOMAIN 1 -- Loads (Q1-Q17)

1. Per ASCE 7-16 Section 12.11.2.1, the out-of-plane wall anchorage force Fp for concrete or masonry walls attached to flexible diaphragms in SDC C through F must be at least:

A. 0.2xSDSxIexWwall -- the standard component anchorage force from ASCE 7-16 Section 13.3.1

B. 0.4xSDSxIexWwall -- the minimum wall anchorage force for concrete/masonry walls on flexible diaphragms in SDC C-F

C. 0.3xSDSxIexWwall -- an intermediate wall anchorage value used in some pre-2016 code editions

D. 0.1xSDSxIexWwall -- the minimum for rigid diaphragm buildings only in low seismic zones

2. Per ASCE 7-16 Section 12.3.3.3, columns supporting discontinuous shear walls must be designed for the special seismic load combination using the overstrength factor Omega0. This requirement exists because:

A. All columns in SDC D must use the overstrength factor regardless of connection to shear walls

B. Discontinuous wall columns have much higher seismic drift demands than continuous wall columns

C. The column must remain elastic even when the shear wall above reaches its maximum strength -- the column cannot be designed at the same yield level as the wall system

D. Columns supporting discontinuous walls are designed for the maximum probable force the shear wall can deliver, ensuring the column does not fail before the wall yields

3. Per ASCE 7-16 Section 4.3.3, the minimum uniformly distributed live load for parking garages for passenger vehicles only (vehicles not exceeding 9,000 lb gross weight) is:

A. 40 psf -- the design live load for passenger car parking garages

B. 50 psf -- the standard office floor load

C. 100 psf -- for mixed passenger car and light truck parking

D. 60 psf -- the live load for mechanical parking garages

4. Per ASCE 7-16 Eq. 12.8-3, the upper bound seismic response coefficient for T=0.5 s (where SD1=0.6g, R=5, Ie=1.0, and T<TL) is Cs=SD1/(Tx(R/Ie)). What is Cs (upper bound)?

A. 0.12g

B. 0.18g

C. 0.24g -- Cs=0.6/(0.5x5/1.0)=0.24g

D. 0.30g

5. Per ASCE 7-16 Section 2.3.4, the load factor for soil pressure loads H (lateral soil pressure from retained soil) in the LRFD combination 1.2D+1.6L+...is:

A. 1.2H -- identical load factor to dead load, since soil pressure behaves like a gravity load

B. 1.0H -- soil pressure treated as a dead load with unity factor

C. 1.6H -- the ASCE 7-16 load factor for H loads when the soil pressure increases the effect of other loads

D. 1.4H -- a single-factor approach combining dead and soil pressure uncertainty

6. Per ASCE 7-16 Section 12.8.3, the story force distribution exponent k for a structure with fundamental period T=1.5 s (between 0.5 s and 2.5 s) is k=1+(T–0.5)/2:

A. k = 1.0 -- the linear distribution for short-period structures

B. k = 1.50 -- linear interpolation for T=1.5 s gives k=1+(1.5–0.5)/2=1.50

C. k = 1.75 -- for T=2.0 s

D. k = 2.0 -- the parabolic distribution for T>=2.5 s

7. Per ASCE 7-16 Table 4.3-1, the minimum live load for operating rooms, procedure rooms, and similar medical surgical spaces in hospitals is:

A. 40 psf -- for patient rooms and exam rooms

B. 80 psf -- for heavy medical equipment

C. 100 psf -- for corridors serving medical areas

D. 60 psf -- the live load for operating rooms and surgical areas per ASCE 7-16

8. Per ASCE 7-16 Section 26.1.4, for buildings and structures designed using simplified analytical methods, a minimum design wind pressure of _____ must be applied to the projected area of the building:

A. 16 psf net horizontal -- the ASCE 7-16 minimum net horizontal design wind pressure for MWFRS

B. 10 psf -- a simplified floor minimum

C. 12 psf -- for residential structures in hurricane-prone regions

D. 8 psf -- the minimum for low-rise structures under Exposure B

9. Per ASCE 7-16 Section 11.4.2, the mapped spectral response acceleration parameter Ss for a site is obtained from:

A. ASCE 7-16 Table 11.4-1 -- a simplified table giving Ss by state

B. The USGS seismic hazard maps (or ASCE 7 online hazard tool) using the site's latitude and longitude -- Ss is the 0.2-second period spectral acceleration on reference rock for the 2%-in-50-years (MCE) hazard level

C. The mapped Ss from the ASCE 7-16 figures -- a contour map giving Ss rounded to the nearest 0.1g

D. The site-specific probabilistic seismic hazard analysis (PSHA) at the 0.2-second period -- required for all sites in SDC D or higher

10. Per ASCE 7-16 Section 7.10, a rain-on-snow surcharge of 5 psf must be added to the ground snow load when:

A. When the site is in a coastal hurricane zone where rain is common during winter storm events

B. When the site elevation exceeds 5,000 ft where rain-on-snow events are most frequent

C. When pg > 20 psf -- heavier snow accumulation zones already include rain effects in their ground load

D. When pg is between 1 and 20 psf and roof slope is less than 1/2 on 12 -- rain-on-snow applies to light snow areas with nearly flat roofs

11. Per ASCE 7-16 Section 13.3.1, the seismic design force Fp for a mechanical component (such as an air handler unit) supported above grade in a building is Fp=0.4apxSDSx(Wp/Rp)x(1+2z/h)xIe. The amplification factor (1+2z/h) reflects:

A. The factor (1+2z/h) reflects amplified floor acceleration at height z -- components at the roof (z=h) receive 3x the ground level acceleration from the building's dynamic response

B. The factor reduces seismic force at higher elevations to account for lower overburden

C. This amplification formula applies only to rigid components; flexible components use a different equation

D. The factor captures the story shear at height z, making seismic equipment force proportional to story shear

12. Per ASCE 7-16 Table 12.3-1, a torsional irregularity (Type 1a) exists when the maximum story drift at one end of the building exceeds _____ of the average story drift computed including accidental torsion:

A. 110% -- the first threshold at which stiffness irregularity of any type is considered

B. 120% -- the maximum-to-average story drift ratio trigger for torsional irregularity Type 1a

C. 150% -- the trigger for extreme torsional irregularity (Type 1b) with more severe consequences

D. 200% -- the trigger for accidental torsion amplification factor Ax>1.0 per ASCE 7-16

13. Per ASCE 7-16 Table 4.3-1, the minimum live load for hospital patient sleeping rooms is:

A. 40 psf -- for dormitories and lightly loaded residential rooms

B. 50 psf -- for general office and light commercial spaces

C. 40 psf -- the ASCE 7-16 live load for hospital patient rooms, nurse stations, and patient care rooms

D. 80 psf -- for medical procedure and recovery rooms

14. Per ASCE 7-16 Section 2.3.2 (LRFD Combination 6), when wind load W acts as the primary load in combination with dead load, the combination is:

A. 0.9D + 1.0W -- for wind uplift, taking minimum dead load (0.9D) against maximum wind uplift
B. 1.2D + 1.6W -- typically the wind-dominated combination for gravity-plus-wind effects
C. 1.0D + 1.6W -- dead load always uses 1.0 combined with maximum wind load factor
D. 1.2D + 1.0W -- the combination for serviceability-level wind checks and other conditions

15. Per ASCE 7-16 Section 4.7.5, the live load reduction for two-way slabs (KLL=2 for supporting two adjacent bays) is based on the reduced AT=KLLxAT. The minimum reduced live load for office floors after reduction cannot be less than:

A. 40 psf -- the unreduced value for office floors
B. 50% of the unreduced LL for members supporting two or more floors -- 0.50x50=25 psf for office floors
C. 60% of the unreduced LL for two-way slabs
D. No reduction permitted -- two-way slabs must always use full unreduced LL

16. Per ASCE 7-16 Eq. 12.8-4, for structures with T>TL (the long-period transition period), the seismic response coefficient Cs is limited to Cs=SD1xTL/(T2x(R/Ie)). For SD1=0.6g, TL=4 s, T=6 s, R=8, Ie=1.0:

A. 0.015g
B. 0.020g
C. 0.025g
D. 0.0083g -- Cs=0.6x4/(36x8)=2.4/288=0.0083g

17. Per ASCE 7-16 Table 4.3-1, the minimum live load for storage areas above garage (residential) where used for items that can be moved or removed and the area is only used for storage is:

A. 40 psf
B. 50 psf
C. 100 psf -- the code live load for storage areas above ceilings
D. 125 psf -- heavy storage in residential construction

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

18. A cantilever beam AB (A=fixed, B=free, L=20 ft) has a point load P=24 kips applied at C, which is 3 ft beyond the free end B (overhang). Wait -- restatement: A simply supported beam (A=pin, B=roller, span L=20 ft) has a point load P=24 kips at C, 3 ft beyond B (overhang). Taking moments about A: RBx20=Px23. What is RB?

A. 27.6 kips upward -- RB=24x23/20=27.6k; RA=24–27.6=–3.6k (downward)
B. 12.0 kips upward
C. 20.0 kips upward
D. 24.0 kips upward

19. For a T-section (bf=24 in, tf=3 in, bw=8 in, hw=12 in, total depth=15 in) with a neutral axis located at ■=6.5 in from the BOTTOM, the moment of inertia about the centroidal axis is:

A. 1,200 in4 -- using simplified formulas without parallel-axis corrections
B. 1,850 in4 -- accounting for the web but not the full flange-centroid offset
C. 2,310 in4 -- from the parallel axis theorem applied to both the flange and web areas
D. 980 in4 -- using the web-only contribution without flange area or offset

20. For a propped cantilever (fixed at A, roller at B, span L=24 ft, EI=constant) with a concentrated load P=30 kips at 8 ft from A, the deflection at midspan can be found by superposition: combining the cantilever deflection from the load and the roller reaction. The midspan deflection is approximately:

A. Upward 0.22 in -- roller reaction more than compensates for load, producing net upward deflection

B. Downward 0.35 in -- net midspan deflection after combining load deflection and roller relief

C. Zero -- midspan deflection is always zero for propped cantilevers by symmetry

D. Downward 0.65 in -- ignoring the upward relief from the propped roller at B

21. In the moment distribution method, the carry-over factor (COF) for a prismatic beam member with the FAR end FIXED is:

A. COF = 0.5 -- one-half of the distributed moment at the near end is carried over to the far end when it is fixed

B. COF = 1.0 -- full moment is carried over when the far end is rigidly fixed

C. COF = 0.25 -- a reduced carry-over for non-prismatic or weakened-section members

D. COF = 0 -- carry-over is zero only when the far end is pinned, not fixed

22. For a section subjected to biaxial bending with Mx and My acting simultaneously, the neutral axis (where normal stress is zero) is NOT parallel to either principal axis unless Mx/My=Ix/Iy. When this ratio is not satisfied, the neutral axis is inclined at angle alpha where:

A. tan(alpha) = MyxIx/(MxxIy) -- the neutral axis is the locus of zero stress points; bending about any axis other than a principal axis with this ratio will make the NA perpendicular to that axis

B. alpha = 45° always -- unsymmetric bending always produces a 45° neutral axis

C. tan(alpha) = 0 -- the neutral axis is always horizontal for symmetric sections

D. tan(alpha) = MxxIy/(MyxIx) -- the neutral axis angle depends on the inverse ratio of moments and section properties

23. For a two-span continuous beam (spans L1=16 ft and L2=20 ft), the influence line for the reaction at the interior support B has positive ordinates when the unit load is:

A. Between A and B only -- the unit load in the left span only produces positive reaction at B

B. Between A and C (both spans) -- positive reaction at B for any load position within the beam

C. At the midspan of either span -- only the midspan positions produce peak IL ordinates at B

D. Only at support B itself -- the IL for any support reaction is only nonzero at that support

24. The deflection at the free end of a cantilever beam (L, EI=constant) under a uniform load w per unit length is δ=wL4/(8EI), computed using virtual work with a unit load at the free end. The integrand M(x)xm(x)/(EI) at a distance x from the fixed end is:

A. No integration needed -- use the tabulated formula directly

B. The unit virtual moment m(x)=x and the real moment M(x)=wL2/2–wx2/2 (from fixed end) are multiplied and integrated from 0 to L

C. M(x)=wx2/2 (from free end as origin) and m(x)=(L–x) -- the product integrated from 0 to L gives the free-end deflection

D. The integration is: ∫0■[w(L–x)2/2 x (L–x)/(EI)]dx = wL4/(8EI) using x measured from the free end

25. For a plane stress state with sigmax=10 ksi, sigmay=–4 ksi, and tauxy=6 ksi, the major principal stress sigma1 is:

A. 11.5 ksi

B. 7.0 ksi

C. 12.2 ksi -- sigma1=(sigmax+sigmay)/2+sqrt(((sigmax–sigmay)/2)2+tauxy2)=3+9.22=12.22 ksi

D. 9.0 ksi

26. A two-span continuous steel beam (W21x50, I=984 in4) of total length 2L=36 ft is subjected to a uniform temperature decrease DeltaT=50°F (bar contracts, causing tension). With fixed ends (both A and C) and a roller at B (interior support), the reaction at B resulting from the thermal effect is:

- **A.** RB = 0 -- temperature causes no reaction at B because the roller is free to move, eliminating constrained thermal stress in the beam
- **B.** RB = EAalphaDeltaT -- the fully constrained thermal force acts at the roller support as an equivalent reaction
- **C.** RB depends on the relative stiffness of the two spans and the specific boundary conditions of the frame
- **D.** RB = 0 only for symmetric beams -- asymmetric configurations develop significant thermal reactions at interior rollers

27. For a two-degree indeterminate propped cantilever (fixed at both A and B, span L) under a concentrated load P at L/3 from A, the force method requires two compatibility equations. The primary structure is obtained by releasing:

- **A.** The pin at B only -- a released pin gives one compatibility equation
- **B.** The fixed end moment at A and the pin at B -- releasing two redundants gives the statically determinate primary structure
- **C.** Only the moment at A -- the rotation compatibility at A alone gives the second equation
- **D.** The moment at B and the vertical reaction at A -- any combination of two releases that creates a stable determinate structure is acceptable as primary structure

28. For a fixed-pinned beam (fixed at A, pinned at B, span L=20 ft) under uniform load w, the plastic collapse mechanism forms with two plastic hinges: one at A (fixed end) and one at the location of maximum positive moment. The collapse load wc (from the virtual work equation) for Mp=200 kip.ft is:

- **A.** wc = 4.24 kip/ft -- the collapse load for a fixed-free beam under UDL with one hinge only
- **B.** wc = 5.83 kip/ft -- plastic collapse of fixed-pinned beam under UDL with hinges at A and at x=0.414L
- **C.** wc = 8.00 kip/ft -- the collapse load for a fixed-fixed beam with central point load
- **D.** wc = 3.00 kip/ft -- the collapse load for a simply supported beam with hinges at midspan only

29. For a three-hinged arch (span L=60 ft, rise f=12 ft, pin at crown) under a uniform load w=2 kip/ft on the full span, the horizontal thrust H is:

- **A.** 75 kips -- H=wL2/(8f)=2x602/(8x12)=7200/96=75 kips
- **B.** 60 kips
- **C.** 90 kips
- **D.** 45 kips

30. For a propped retaining wall (flexible sheet pile) with a single row of anchors, the moment distribution along the pile depends on the location and magnitude of the anchor force. The maximum bending moment in the wall is:

- **A.** At the anchor elevation -- the anchor tie-rod connection is always the maximum moment location
- **B.** Between the anchor elevation and the dredge line -- the maximum moment occurs in the span between the anchor and the point of zero shear in the net pressure diagram
- **C.** At the dredge line -- the dredge line separates the active and passive zones
- **D.** At mid-height of the retained soil -- for uniform soil, the moment peaks at half the retained depth

31. For a cantilever beam (L, fixed at A, free at B) under a uniform load w, the deflection at the free end B using the conjugate beam method is:

- **A.** δB = wL4/(24EI) -- cantilever with point load formula often confused with UDL case
- **B.** δB = wL4/(12EI) -- double the correct UDL cantilever result
- **C.** δB = wL4/(8EI) -- the standard cantilever under uniform load tip deflection result
- **D.** δB = 5wL4/(384EI) -- the simply supported beam midspan UDL deflection formula

32. In moment distribution for a two-span beam (EI constant, spans L1=18 ft and L2=24 ft, all ends simply supported at A and C, fixed at the interior support B), the distribution factor for member BA at joint B is:

A. DF_BA = 0.75 -- for the shorter span with pinned far end

B. DF_BA = K_BA/(K_BA+K_BC) = (3EI/L1)/(3EI/L1+3EI/L2) = (1/18)/(1/18+1/24) = 0.57

C. DF_BA = 0.50 -- equal distribution for any two members

D. DF_BA = 0 -- no moment distribution at simply supported ends

33. For a rectangular cross-section (b=6 in, h=12 in), the 'kern' is the central region within which an axial load P must fall to ensure no tensile stress anywhere in the section. The kern is a rhombus (diamond shape) with dimensions:

A. b/6=1 in and h/6=2 in half-dimensions -- the kern half-dimensions for a rectangle are b/6 and h/6, giving a total kern width of b/3 and height of h/3

B. b/4 and h/4 -- the quarter-dimension kern for rectangular sections

C. b/3 and h/3 -- the full-dimension kern dimensions

D. b/6 and h/6 -- a very small kern for thick sections

34. Per AISC 360-16 Section G3, the nominal shear strength from tension field action in a plate girder web is Vn=AwxFcr where Fcr includes both the direct buckling resistance and the post-buckling tension field. The tension field mechanism is permitted when:

A. The web is non-compact -- tension field action applies to all non-compact webs

B. Intermediate web stiffeners are provided, and the end panels adjacent to supports are excluded from tension field -- the end panel is designed without TFA for stability reasons

C. The aspect ratio a/h <= 3.0 and h/tw <= 260 -- both limits must be satisfied

D. The applied shear is less than 60% of Vcr -- the slotted web criterion

35. For a stepped column (two-segment, upper segment L1, lower segment L2, with I2>I1), the effective buckling length can be found from the transcendental eigenvalue equation. For the simplified case where the upper segment is very flexible (I1→0), the effective length K approaches:

A. K = 0.5 -- both ends fully fixed, providing maximum restraint and minimum effective length

B. K = 0.7 -- one end fixed, one end pinned, for a braced frame column

C. K = 1.0 -- both ends pinned, the reference effective length for Euler column buckling

D. K = 2.0 -- fixed base with free top (cantilever column); the very flexible upper segment provides no rotational restraint

36. For a multi-story braced frame (regular, all bays braced), the story shear at any level i for lateral seismic loading is:

A. Vi = V1 -- the base shear applies equally to every story in a simplified analysis

B. Vi = Wixhi/ΣWjxhj x V -- the story force at level i, not the story shear

C. Vi = ΣFj (j from i to n) -- the story shear is the sum of all seismic forces at level i and above

D. Vi = Fi x n -- the story force at level i times the total number of stories

37. For a composite steel-concrete beam, the effective moment of resistance depends on the degree of composite action (partial vs. full). For PARTIAL composite action (50% composite), the nominal moment strength Mn is:

A. Between Mp_steel and Mp_full_composite -- partial composite strength increases monotonically with the number of shear studs up to the full composite limit

B. Equal to Mp_steel -- partial composite connection provides no strength benefit over bare steel

C. Equal to 50% of the full composite moment for 50% composite action

D. Always equal to the full composite moment regardless of the partial stud ratio

38. In a second-order (P-delta) analysis of a building frame, the geometric stiffness matrix [KG] represents:

A. The elastic stiffness matrix for frame members under gravity load only -- no P-delta effects

B. The additional bending demand from lateral loads at each story level

C. The reduction in effective lateral stiffness from axial compression -- [Keff]=[Kelastic]–[KG], where [KG] accounts for the P-Delta stiffness loss

D. The contribution of diagonal tension-only cable elements to overall lateral resistance

39. For a two-degree-of-freedom (2-DOF) shear building model (m1=m2=100 kips/g, k1=k2=600 kip/in), the natural frequencies are found from the eigenvalue equation. The fundamental (lower) frequency ω1 is approximately:

A. ω1 = sqrt(k/m) = sqrt(600x386/100) = 48.1 rad/s
B. ω1 = 1.93 sqrt(k/m) x correction -- from the 2-DOF characteristic equation
C. ω1 ~ 0.618 x sqrt(k/m) -- the lower eigenvalue of the 2-DOF system using the golden ratio approximation
D. ω1 = sqrt(2k/m) -- both springs act in parallel for the first mode

40. For a short beam (span-to-depth ratio L/d=2) where shear deformation is significant, the total midspan deflection from a central concentrated load P includes both flexural and shear components. The shear deflection is:

A. Negligible -- shear deflection is always less than 5% of the flexural deflection
B. δ_shear = 1.2xPxL/(4xGxAw) -- the shear deflection for a rectangular section
C. δ_shear = PxL3/(48EI) -- the same as the flexural deflection formula
D. δ_shear = 1.2xPL/(AxGx4) -- significantly more than 5% for very short beams where shear governs deflection

41. For a two-span continuous beam (equal spans L), the influence line for the bending moment at the INTERIOR SUPPORT B has maximum negative ordinate when:

A. The unit load is at midspan of either span -- load at the midspan of either span creates maximum negative interior support moment
B. The unit load is at B itself -- maximum moment at the support occurs when the load is at the support
C. The unit load is at the midspan of the left span (or alternately the right span) -- the IL ordinate is maximum negative when the load is in the adjacent span, not over the interior support
D. The unit load is at A or C -- end support loads create maximum negative B moment

42. A parabolic cable (horizontal span L=120 ft, sag f=15 ft) carries a uniform load w=1.5 kip/ft. The horizontal thrust H=wL2/(8f) and maximum cable tension T=sqrt(H2+(wL/2)2). What is T?

A. 185 kips
B. 192 kips
C. 196 kips
D. 201 kips -- H=1.5x1202/(8x15)=180k; T=sqrt(1802+902)=sqrt(32400+8100)=sqrt40500=201 kips

DOMAIN 3 -- Temporary Structures (Q43-Q49)

43. Per ACI 308R-16, the minimum concrete curing temperature for Type I Portland cement concrete is 50°F (10°C), and the minimum curing duration at this temperature is:

A. 7 days -- the minimum curing period for Type I cement concrete at or above 50°F per ACI 308R
B. 3 days -- adequate for high early strength Type I concrete
C. 14 days -- double the standard minimum for exposed concrete
D. 28 days -- the age at which design strength is specified

44. Per OSHA 29 CFR 1926.702(e), concrete chutes used for placing concrete must be sloped at not more than:

A. 30° from horizontal -- a shallow slope to prevent concrete segregation
B. 45° from horizontal -- the maximum angle for concrete chutes to prevent excessive segregation from high-velocity flow
C. 60° from horizontal -- steep chutes cause segregation
D. Any angle if vibration is applied at the bottom

45. Per ACI 347R-14 and ACI 301, the minimum concrete compressive strength required before removing the side forms (not the shoring) from walls and columns is typically:

A. 500 psi -- forms can be stripped early while the element is still curing

B. 2,000 psi -- the ACI minimum for form removal from walls and columns when the concrete can support itself without cracking or excessive deflection

C. 3,000 psi -- the design compressive strength

D. The full design strength f'c -- forms must remain in place until f'c is achieved

46. Per ACI 207.1R-05 (mass concrete), the maximum allowable temperature in the interior of a mass concrete placement should not exceed:

A. 100°F (38°C) -- a conservative limit for all mass concrete regardless of cement content

B. 140°F (60°C) -- appropriate for moderate cement content and good quality aggregate

C. 160°F (71°C) -- the threshold beyond which hydration kinetics become dangerous

D. 158°F (70°C) -- the ACI 207.1R recommended maximum to prevent delayed ettringite formation and thermal gradient cracking

47. Per ACI 347R-14 Section 3.4, concrete formwork design for construction loads should include the weight of:

A. Fresh concrete plus a minimum 25 psf construction live load -- per ACI 347R minimum construction live load

B. Fresh concrete only -- live loads are not a formwork concern

C. Fresh concrete plus a minimum 50 psf when motorized concrete carts are used on the forms

D. The dead load of the form plus the weight of concrete plus reinforcement plus a minimum 50 psf for motorized equipment -- or 75 psf when concrete buggies are used

48. Per ACI 347R-14 and the concrete construction industry practice, the sequence for removing shores in multi-story construction (re-shoring sequence) requires:

A. Removing all shores from the lowest floor first, then progressively working upward floor by floor

B. Removing shores only after the concrete above has reached its full specified 28-day compressive strength

C. Installing reshores on the floor below before removing shores from the floor above -- the rolling shore sequence prevents overloading any partially cured floor

D. Simultaneously removing shores from all floors at 7 days on a fast-cycle construction schedule

49. Per ACI 304R-00 and standard practice, the maximum slump for pump-placed concrete (excluding self-consolidating concrete) to ensure adequate pumpability without excessive segregation is typically:

A. 3 in -- the standard slump for conventional non-pumped structural concrete

B. 6 in -- the upper practical limit for pump-placed concrete, above which segregation and pump blockage risk increases

C. 8 in -- the maximum slump permitted by ACI for any structural concrete using conventional admixtures

D. 2 in -- pump-placed concrete requires very low slump to maintain adequate pump pressure

DOMAIN 4 -- Materials (Q50-Q63)

50. Per AWS D1.1 and standard structural practice, preheat requirements for welding ASTM A36 structural steel are generally:

A. No preheat required for A36 steel regardless of thickness -- A36 has low carbon equivalent

B. Preheat required above 1 in thickness -- the carbon equivalent of A36 exceeds the threshold for thicker plates

C. Preheat always required -- A36 requires mandatory preheat for all welding

D. Preheat required only when ambient temperature is below 0°F -- a temperature-only limitation for A36

51. Per ACI 232.2R-03, fly ash added to concrete as a partial cement replacement (typically 15-35% by mass) improves concrete by:

A. Reducing the initial set time -- fly ash accelerates the early hydration reaction

B. Increasing heat of hydration -- fly ash contributes additional exothermic reaction

C. Reducing heat of hydration, improving long-term strength through pozzolanic reaction, reducing water demand, and enhancing durability against sulfate and alkali-silica reactions

D. Eliminating the need for air entrainment in freeze-thaw exposures

52. Per NDS 2018 Supplement Table 4A, the size factor CF for bending design value Fb applied to a 2x10 sawn lumber member (width=9.25 in, depth=1.5 in -- a flat-wise application) is:

A. CF = 1.1 -- the size factor for a 2x10 in edgewise bending

B. CF = 1.0 -- no adjustment for standard depths

C. CF = 0.9 -- a reduction for wide members

D. CF = 1.2 -- for the shallowest dimensions

53. The fiber saturation point (FSP) of wood is the moisture content above which free water exists in the cell cavities but below which no further shrinkage or swelling occurs. For structural lumber, the FSP is approximately:

A. 15% MC -- the equilibrium moisture content typical of interior heated building environments

B. 19% MC -- the NDS reference threshold above which the wet service factor CM applies

C. 25% MC -- the approximate equilibrium moisture content for very humid tropical or marine environments

D. 28% MC (range 25-30%) -- the fiber saturation point above which wood dimensions do not change with moisture content

54. Per ASTM C33, the fineness modulus (FM) of fine aggregate (sand) for structural concrete should fall within:

A. FM = 1.0 to 2.0 -- extremely fine sand producing unworkable, paste-hungry concrete

B. FM = 3.0 to 4.0 -- the range for coarser natural sands approaching aggregate grading

C. FM = 2.3 to 3.1 -- ASTM C33 acceptance range for structural concrete fine aggregate

D. FM = 4.0 to 6.0 -- the range for coarse aggregate including gravel and crushed stone

55. Per ASTM A500, Grade B round Hollow Structural Section (HSS) has the following minimum mechanical properties:

A. Fy=46 ksi, Fu=62 ksi -- for square and rectangular A500 Grade B

B. Fy=42 ksi, Fu=58 ksi -- the minimum yield and tensile strength for ASTM A500 Grade B round HSS

C. Fy=50 ksi, Fu=65 ksi -- same as A992 wide flange steel

D. Fy=36 ksi, Fu=58 ksi -- A36 equivalent for HSS

56. The hydraulic conductivity (permeability) k of a soil is typically expressed in cm/s in geotechnical practice. For a fine sand, k is typically in the range of:

A. 10■3 to 10■1 cm/s -- the range for fine to coarse sand, corresponding to moderate to high permeability

B. 10■7 to 10■5 cm/s -- the range for silts and fine-grained soils

C. 10■1 to 10 cm/s -- the range for gravel

D. 10■9 to 10■7 cm/s -- the range for intact clays

57. Per TMS 402-16 Section 8.3.3.1 (ASD), the maximum allowable tensile stress in reinforcing steel for non-prestressed reinforced masonry is limited to:

A. 0.5fy -- used in some prestressed concrete allowable stress specifications

B. 0.6fy -- the allowable tensile stress limit for structural steel in some ASD frameworks

C. 0.33fy -- a very conservative limit sometimes used for unreinforced masonry only

D. 0.4fy -- the TMS 402-16 ASD allowable tensile stress for reinforcing steel in reinforced masonry

58. Per ASTM A615, the minimum elongation requirement for Grade 60 reinforcing bars (#8 and smaller) is:

A. 5% in 8 inches -- below the minimum required for standard structural applications

B. 7% in 8 inches -- the ASTM A615 Grade 60 minimum elongation for bars #3 through #6

C. 10% in 8 inches -- a higher ductility requirement for seismic Grade 60 (A706) bars

D. 12% in 8 inches -- the elongation requirement for Grade 40 (lower strength) reinforcing bars

59. Per ASTM C494, a Type C accelerating admixture is used in concrete to:

A. A water-reducing admixture (Type A) -- reduces water demand without changing the setting time

B. A retarding admixture (Type B) -- extends setting time for hot weather or long-distance transport

C. An accelerating admixture -- Type C specifically accelerates initial and final set for cold weather or early strength applications

D. An air-entraining admixture -- creates microscopic air bubbles for freeze-thaw durability

60. Per NDS 2018 Section 3.9.2, the interaction equation for a wood member in combined bending and axial tension is:

A. ft/F't + fb/F'b <= 1.0 -- linear interaction of combined tension and bending stresses

B. (ft/F't)2 + fb/F'b <= 1.0 -- quadratic tension term, used for combined compression and bending

C. ft/F't x fb/F'b <= 1.0 -- multiplicative interaction formula for combined loading

D. ft/F't – fb/F'b <= 0 -- tension stress must exceed bending contribution

61. Per ACI 318-14 Table 19.3.3, the target air content for concrete exposed to SEVERE freezing and thawing conditions with 3/4-inch maximum aggregate size is:

A. 6.0% +/- 1.5% -- the total air content for freeze-thaw durability with 3/4-inch aggregate

B. 4.5% +/- 1.5% -- for mild freeze-thaw exposure

C. 3.0% +/- 1.0% -- the moderate exposure air content

D. 7.5% +/- 1.5% -- for 3/8-inch maximum aggregate size

62. Per AISC Commentary Section C-J3.1 and industry standards, A490 high-strength bolts must NOT be hot-dip galvanized because:

A. The zinc coating reduces the slip coefficient below the required mu=0.50

B. Hot-dip galvanizing of A490 bolts is prohibited due to the risk of hydrogen embrittlement -- the pickling (acid cleaning) process prior to galvanizing can introduce atomic hydrogen into the steel, which can cause delayed brittle fracture in high-strength bolts under sustained tension

C. The galvanizing temperature (about 830°F) anneals the A490 steel and reduces its strength below the 150-ksi minimum

D. The zinc coating makes it impossible to achieve the required bolt pretension

63. The AASHTO soil classification system (M145) classifies soils primarily by their:

A. Origin and particle shape -- geological descriptors not used in AASHTO M145 classification

B. Grain size distribution and Atterberg limits -- the AASHTO M145 system rates A-1 through A-7 for pavement subgrade suitability

C. Laboratory-measured engineering properties including strength, compressibility, and permeability

D. Geological formation and depositional environment -- alluvial, glacial, or residual origin

DOMAIN 5 -- Component Design (Q64-Q100)

64. Per ACI 318-14 Section 8.3.1.1, for a flat plate (no beams, no drop panels), the minimum slab thickness to control deflection for an interior panel with clear span ln=18 ft and Grade 60 reinforcement (fy=60,000 psi) is h=ln/30:

A. 6.0 in -- below ACI minimum for flat plates without drop panels

B. 6.8 in -- using ln/31.7 instead of the correct ln/30 formula

C. 7.8 in -- using ln/27.7, a value applicable to flat plates with drop panels

D. 7.2 in -- h=18x12/30=7.2 in, the minimum for Grade 60 flat plates without drop panels

65. Per ACI 318-14 Section 25.2.3, the minimum beam width for two #9 bars (db=1.128 in) per layer with 1.5-in clear cover, #3 stirrups (ds=0.375 in), and 1-in minimum clear spacing between bars is:

- **A.** 9.0 in -- using 1.5-in cover but excluding stirrup diameter and bar spacing
- **B.** 11.2 in -- including stirrup diameter but using 0.75-in minimum bar spacing
- **C.** 12.7 in -- with 1.5-in cover, #3 stirrups, two #9 bars, and 1.0-in minimum clear spacing
- **D.** 10.0 in -- a rounded practical minimum for beams with two bars per layer

66. Per ACI 318-14 Section 18.6.4 (Special Moment Frames -- Beams), the maximum hoop spacing within the region of potential plastic hinging (within 2h from the column face) for a beam with d=20 in and #8 longitudinal bars (db=1.0 in) is:

- **A.** 3 in -- for very heavily reinforced beams in special seismic zones
- **B.** 4 in -- when d/4 < 6 for very deep beams
- **C.** 6 in -- the absolute maximum regardless of bar size
- **D.** 5 in -- the minimum of d/4=5 in, 6xdb=6 in, and 6 in per ACI 318-14 Section 18.6.4.4

67. For a post-tensioned concrete beam (Ep=28,500 ksi, Es=29,000 ksi, f'c=5,000 psi, Ec=4,031 ksi), the prestress loss due to elastic shortening (for a single post-tensioned tendon in a singlespan simply-supported beam) is:

- **A.** No loss for post-tensioned beams -- elastic shortening losses occur only in pretensioned beams
- **B.** DeltafpES = Epx(fcgp/Ec) -- for a single post-tensioned tendon, elastic shortening loss exists but is small; for multi-strand post-tensioned, it equals the average prestress for the nth strand
- **C.** DeltafpES = Ep/Es x DeltafsES -- the prestress loss equals the modular ratio times the steel stress loss
- **D.** DeltafpES = 0.25x(initial prestress) -- a simplified 25% loss factor

68. Per ACI 318-14 Section 25.8.6, for prestressed concrete, the transfer length for seven-wire strands (the distance from the end of the member over which prestress is transferred from the strand to the concrete by bond) is:

- **A.** 50db -- the ACI 318-14 transfer length for 7-wire prestressing strands under transfer of prestress
- **B.** 150db -- the ACI development length for prestressing strands
- **C.** fsexdb/3 -- the direct formula using effective prestress
- **D.** 60db -- the ACI upper bound for transfer length under unfavorable conditions

69. Per AISC 360-16 Table D3.1, for a welded plate connection where the plate is welded with longitudinal welds only (no transverse weld at the end of the element) and the weld length L_w=4w (four times the plate width w), the shear lag factor U is:

- **A.** U = 0.75 -- for weld length L_w between 1.5w and 2w per AISC Table D3.1
- **B.** U = 0.80 -- for weld length L_w between 2w and 4w per AISC Table D3.1
- **C.** U = 0.87 -- for weld length L_w >= 4w, plates with longitudinal welds only per AISC Table D3.1
- **D.** U = 1.00 -- when CJP groove welds are present, giving full section efficiency

70. For a W18x46 (Zx=90.7 in3, Sx=78.8 in3, ry=1.29 in, rts=1.50 in, c=1.0, J=0.358 in4, Cw=3160 in6, Fy=50 ksi) with unbraced length Lb<Lp, the nominal flexural strength Mn per AISC 360-16 Section F2 is:

- **A.** Mn = Mp – (Mp–0.7FySx)xreduction -- the LTB reduction formula for Lp<Lb<Lr gives Mn<Mp
- **B.** Mn = 0.7FySx -- the limiting elastic LTB stress
- **C.** Mn = FcSx -- buckling stress times elastic section modulus
- **D.** Mn = FyZx = Mp = 50x90.7/12 = 378 kip.ft -- full plastic moment when Lb < Lp

71. Per AISC 360-16 Section H1-1b, for a beam-column with Pu/phicPn=0.15 (less than 0.2, so H1-1b controls), the combined interaction check is:

- **A.** Pu/(2phicPn) + Mux/phibMnx <= 1.0 -- the H1-1b equation with the half-axial term
- **B.** Pu/(2phicPn) + (Mux/phibMnx + Muy/phibMny) <= 1.0
- **C.** Pu/phicPn + (Mux/phibMnx) <= 1.0 -- the linear interaction for low axial load
- **D.** Pu/phicPn + 0.9x(Mux/phibMnx) <= 1.0 -- with reduction factor for combined loading

72. Per AISC 360-16 Table K3.1, for a rectangular HSS-to-HSS T-connection (K-gap connection) under tension in the branch, the governing failure mode is typically:

A. Punching shear through the chord face -- the branch creates localized punching failure in the hollow chord wall at the branch footprint

B. Chord face plastification -- the chord walls yield in a pattern controlled by the branch width ratio beta and chord stress; Pn depends on wall thickness and beta

C. Chord side wall failure by local buckling under the compressive component of the branch force

D. Branch local yielding -- the branch tube itself yields at the weld toe before any connection failure

73. Per AISC 360-16 Section J3.8 and RCSC Specification, the nominal slip resistance per bolt for a Class A faying surface (mu=0.35), standard holes, with 3/4-in A325 bolt (Pt=28 kips) in single shear is rn=muxDuxhfxPtxns:

A. 11.1 kips -- rn=0.35x1.13x1.0x28x1=11.07 kips

B. 8.4 kips

C. 14.0 kips

D. 22.0 kips

74. Per AISC 360-16 Section J8, a square column base plate (NxB) under a factored column load Pu=480 kips bearing on A36 plate (Fy=36 ksi) and a concrete support with f'c=4 ksi. The bearing strength per AISC is: phicxPp=phicx0.85xf'cxA1 (no confinement) = 0.65x0.85x4xA1. For A1=NxB and Pu<=phicPp, the minimum plate area A1 is:

A. 155 in2

B. 172 in2

C. 217 in2 -- A1=Pu/(phix0.85f'c)=480/(0.65x0.85x4)=480/2.21=217 in2

D. 131 in2

75. Per NDS 2018 Section 3.3.3, the beam stability factor CL for a wood beam depends on the effective slenderness ratio RB=sqrt(lexd/b2) where le is the effective bending length (from NDS Appendix A). For a simply supported beam (le=1.63L+3d for uniformly loaded conditions), CL:

A. CL = 1.0 -- when RB <= 10 and the beam satisfies the depth-to-breadth ratio requirements without any lateral bracing

B. CL < 1.0 when RB > 10 -- slender unbraced beams have reduced allowable bending stress; CL is computed from the formula CL=[(1+FbE/F*b)/(1.9)] – sqrt([(1+FbE/F*b)/(1.9)]2 – FbE/(0.95xF*b))

C. CL = 0 -- all unbraced beams collapse laterally before reaching design stress

D. CL = 0.85 -- a fixed 15% reduction for all unbraced beams in the NDS simplified method

76. Per AF&PA SDPWS 2021, the diaphragm unit shear capacity for a blocked wood structural panel diaphragm (10d nails at 6-in spacing at all edges and 12-in at intermediate framing) exceeds the unblocked capacity by approximately:

A. 30% -- blocking provides only a modest improvement when diagonal sheathing is already present

B. 50% -- a typical estimate for the benefit of blocking in metal-plate-connected truss systems

C. 100% -- blocking doubles the unblocked capacity, sufficient for medium seismic zones

D. More than 100% -- adding blocking more than doubles the capacity by providing nail lines at every panel edge, far exceeding the unblocked allowable

77. Per TMS 402-16 Section 8.3.4.1.1 (ASD for unreinforced masonry in flexure), the allowable tensile stress for flexural tension normal to the bed joints in running bond (Type S or M mortar, hollow CMU) is:

A. 38 psi -- for Type S mortar, fully grouted hollow CMU with continuous bond beam reinforcement

B. 19 psi -- for Type N mortar, hollow ungrouted CMU at limited masonry unit strength

C. 50 psi -- for solid or fully grouted CMU with Type M mortar

D. 25 psi -- TMS 402-16 ASD allowable tensile stress normal to bed joints for hollow CMU in Type S/M mortar

78. Per TMS 402-16 Section 9.3.4.3, the nominal shear strength of a reinforced masonry shear wall with both masonry (Vm) and steel (Vs) contributions is Vn=Vm+Vs. The masonry contribution Vm for a wall with P=50 kips, An=96 in2, and f'm=2,000 psi (sqrtf'm=44.7 psi) is:

A. Vm = [4.0–1.75(M/Vdv)]xAnxsqrtf'm + 0.25P -- per TMS 402-16 Eq. 9-20 for reinforced masonry shear walls
B. Vm = sqrtf'm x An -- the simplified masonry shear contribution for unreinforced analysis
C. Vm = 2xsqrtf'mxbwxd -- the concrete beam analogy for masonry shear strength
D. Vm = 0.4P/An x bwxd -- a friction-based formula analogous to dowel action

79. Per ACI 318-14 Section 13.2.7.3, for a strip (wall) footing, the critical section for one-way shear is located at:

A. At the face of the supported wall -- no d-offset is used for strip footings under concentrated line loads
B. At a distance d from the wall face -- the same d-offset rule applies to strip footings one-way shear as for beams
C. At midspan of the footing between column reactions
D. At B/4 from the footing edge -- a simplified location for the critical shear section

80. For a square footing (B=4 ft=48 in) on cohesive soil (c=500 psf, phi=20°), with Df=3 ft, gamma=115 pcf, and Terzaghi bearing capacity factors Nc=14.83, Nq=6.40, Ngamma=3.64, the ultimate bearing capacity for a square footing (qu=1.3cxNc+qxNq+0.4gammaxBxNgamma) is:

A. 9.67 ksf
B. 11.28 ksf
C. 12.5 ksf -- qu=1.3x0.5x14.83+0.345x6.40+0.4x0.115x4x3.64=9.64+2.21+0.67=12.52 ksf
D. 14.8 ksf

81. For a drilled concrete pier bearing on sound rock, the point bearing resistance may be estimated using the unconfined compressive strength qu_rock. For qu_rock=10,000 psi and a reduction factor alpha=0.3, the allowable unit point bearing pressure qa is:

A. 3,000 psi -- but applying FS=2.0, giving qa=1,500 psi for the allowable value
B. 1,500 psi -- applying alpha=0.15 to account for very poor rock quality
C. 3,000 psi -- allowable point bearing = alphaxqu_rock = 0.3x10,000 = 3,000 psi applied as the allowable
D. 1,000 psi -- a blanket conservative limit used for all rock types regardless of testing

82. For the heel of a cantilever retaining wall, the flexural tension in the footing slab acts on the TOP face (tension from upward net bearing pressure – weight of soil above heel). The design moment for the heel reinforcement per foot of wall is computed at the:

A. Back face of the wall stem -- the critical section for heel bending is at the back face of the stem, where the full heel cantilever arm applies
B. Mid-length of the heel -- average moment for distributed loading
C. Footing centerline -- the maximum moment at the center of the footing
D. Front face of the stem -- for the heel side, the distance from front of stem to heel edge gives the maximum moment arm

83. For a pile group (4x4=16 piles, pile diameter dp=16 in, center-to-center spacing s=3.5 ft, L=40 ft embedded in clay Su=1,200 psf), the block failure capacity Qblock assumes the group acts as a single large block. The perimeter shear of the block Qs_block:

A. Qs_block = Su x perimeter x L -- using a simplified circular/rectangular perimeter
B. Qs_block = Su x 2x(B_group+L_group) x L -- block perimeter x length x skin friction unit resistance
C. Qs_block = Su x 9 x A_tip -- the end bearing formula for block failure at the base
D. Qs_block = Su x 2x(B+L)_group x L = 1.2x4x10.5x40 = 2,016 kips -- for the 4x4 pile group

84. For a drilled shaft in clay using the alpha (total stress) method, the unit skin friction is fs=alphaxSu where alpha is the adhesion factor. For Su=1,500 psf (0.75 ksf), the typical alpha from the API or O'Neill-Reese approach is approximately:

A. alpha = 1.0 -- full adhesion factor for all cohesive soils regardless of Su or interface conditions

B. alpha = 0.55 -- based on O'Neill and Reese (1999) for drilled shafts in cohesive soils with Su~1,500 psf

C. alpha = 0.30 -- a very conservative value used when soil-interface properties are poorly characterized

D. alpha = 0.80 -- for stiff over-consolidated clays above 2,000 psf undrained shear strength

85. Per ACI 318-14 Section 22.6.5.2, for a flat mat foundation with an edge column (L-shaped critical perimeter on two sides), the punching shear critical perimeter bo and the applicable alphas factor are:

A. bo = 4x(c+d), alphas = 40 -- same as interior column

B. bo = 3x(c+d), alphas = 30 -- two sides of the punch perimeter for edge column -- not the L-shape but the three-sided critical perimeter

C. bo = 2x(c+d), alphas = 20 -- for corner columns

D. bo = 3 sides, alphas = 30 -- the edge column critical perimeter has three sides (two parallel to the edge + one perpendicular), and alphas=30 per ACI 318-14 Section 22.6.5.2

86. Per Terzaghi's consolidation theory, the relationship between primary and secondary consolidation settlement is:

A. Primary consolidation occurs under excess pore pressure; secondary consolidation follows after dissipation, continuing under constant effective stress due to soil skeleton creep

B. Primary and secondary consolidation both occur simultaneously and cannot be separated during oedometer testing

C. Secondary consolidation settlement always exceeds primary settlement for all fine-grained saturated soils

D. Primary consolidation is reversible when load is removed; secondary consolidation is permanent and non-recoverable

87. For a spread footing straddling two soil types (the footing plan overlaps both competent soil and a weak zone), the foundation design should:

A. Use the weaker soil's bearing capacity uniformly -- the weakest zone controls the entire footing design

B. Design using only the stronger soil, ignoring the weak zone in the bearing area calculation

C. Proportion the footing or provide a reinforced transfer slab to place all load on competent soil

D. Increase the footing embedment depth until all soil below is the stronger material

88. For an infinite slope of cohesionless soil (c=0) with slope angle beta=22° and friction angle phi=32°, the factor of safety against shallow translational sliding is FS=tan(phi)/tan(beta):

A. 1.10 -- barely stable

B. 1.55 -- FS=tan(32°)/tan(22°)=0.6249/0.4040=1.55

C. 1.82 -- for the same slope with c=50 psf cohesion added

D. 2.10 -- adding seepage effects

89. Per ACI 318-14 Section 13.4.8, for a two-pile pile cap (pile diameter dp=16 in, cap depth h=30 in, d=25 in), the critical perimeter for punching shear at each pile is at d/2 from the pile face. The critical perimeter bo for a circular pile is:

A. pix(dp+d/2) = pix24 = 75 in -- using d/2 as the equivalent diameter addition

B. 4x(dp+d) = 4x41 = 164 in -- rectangular perimeter approximation for a circular pile

C. 2x(dp+d) = 2x41 = 82 in -- partial perimeter for a two-sided critical section

D. pix(dp+d) = pix41 = 129 in -- circular critical perimeter at d/2 from the pile face

90. Per Rankine's passive earth pressure theory, the passive pressure coefficient Kp for a cohesionless backfill with phi=30° is:

A. Kp = 0.33 -- the active pressure coefficient for phi=30°

B. Kp = 1.0 -- no amplification at rest

C. Kp = 3.0 -- (1+sinphi)/(1−sinphi) = 1.5/0.5 = 3.0 for phi=30°

D. Kp = 4.0 -- for friction angles above 25°

91. For a mat foundation subjected to hydrostatic uplift (groundwater table at the mat base level) with mat size 60 ft x 80 ft, the net upward hydrostatic force on the mat when the mat is 2 ft thick (concrete unit weight 150 pcf) is:

A. The upward pressure at the mat base = gamma_waterxh_water -- with WT at the mat base, h_water=0, so upward pressure = 0; net uplift = 0 for this condition

B. Net uplift is zero when WT is at mat base -- uplift only develops when WT rises above the mat base elevation

C. Net uplift = 62.4x60x80 = 300 kips -- this calculation assumes 1 ft of water head above the mat base

D. Full hydrostatic pressure acts when WT reaches the top of the mat face, using the full mat thickness as water head

92. Per ACI 318-14 Section 15.4.1, the minimum temperature and shrinkage reinforcement in a wall footing (strip footing) in the longitudinal direction is:

A. rho = 0.0018 -- temperature and shrinkage steel for Grade 60 in the long direction of a strip footing

B. rho = 0.0020 -- based on the structural reinforcement minimum

C. rho = 0.0014 -- the minimum for Grade 60 bars in slabs on grade

D. No minimum -- strip footings have no temperature and shrinkage reinforcement requirement in the longitudinal direction

93. The Standard Proctor Test (ASTM D698) uses a specific compaction energy per unit volume. In standard Proctor testing, the optimum moisture content (OMC) is the moisture content at which:

A. The moisture content at maximum wet density -- wet density always increases with increasing moisture content

B. The moisture content maximizing friction angle -- internal friction is highest at dry or slightly moist conditions

C. The moisture content minimizing hydraulic conductivity -- minimum permeability occurs slightly above OMC

D. The moisture content producing maximum dry unit weight -- above OMC, added water displaces soil solids, reducing dry density

94. Per ACI 318-14 Section 13.5.1.2, for a spread footing on a wall (strip footing) bearing on concrete or masonry walls, the critical section for FLEXURAL design is at:

A. Halfway between the wall face and the footing edge -- a compromise critical section for masonry on concrete

B. The wall face -- the same as a concrete wall on soil

C. One-fourth the wall thickness inside the wall face -- the ACI critical section for masonry wall footings

D. The edge of the base plate or bearing area -- for steel column footings

95. Per Tomlinson's alpha method for pile skin friction in clay, the adhesion coefficient alpha is typically:

A. alpha = 1.0 for all clays -- full adhesion between concrete pile and clay

B. alpha = 0.5 for soft clays (Su<250 psf) and decreasing toward 0.25 for stiff clays (Su>2000 psf) -- the adhesion factor reduces for stiffer, over-consolidated clays

C. alpha ranges from approximately 1.0 (very soft clay) to 0.5 (stiff clay), generally decreasing as the undrained shear strength Su increases -- for practical design, alpha=0.6-0.8 is common for typical soft to medium clays

D. alpha = 0 -- pile skin friction in clay is only developed through downdrag, never through positive adhesion

96. For a combined footing spanning between two columns (L=20 ft total, Column 1 at 0, Column 2 at 20 ft), the shear between the two columns changes sign at the point of zero shear. The location of maximum positive moment is:

A. At Column 1 face -- the maximum moment is negative (hogging) at the first column face for the typical combined footing case

B. At the point of zero shear between the columns -- the positive moment maximum occurs at V=0 by integration of the net upward pressure

C. At the geometric centroid of the combined footing

D. At the midpoint between the two columns -- only valid when the two column loads are exactly equal

97. Stone column ground improvement increases bearing capacity by:

A. Replacing the weak soil with granular fill in discrete locations -- stone columns are purely displacement elements

B. Acting as stiff reinforcing elements within the soft soil matrix (composite composite reinforcement) while also providing radial drainage paths to accelerate consolidation of the surrounding clay

C. Providing vertical drainage paths only -- stone columns are drainage elements with negligible structural contribution

D. Vibro-compacting the surrounding soil during installation -- densification alone provides all the improvement

98. For a warehouse building, the permissible total and differential settlement criteria are typically governed by:

A. Angular distortion δ/L = 1/300-1/500 to prevent frame and facade damage, with absolute total settlement limit of 1-2 in for typical warehouse column footings

B. Groundwater table criterion -- settlement limits only apply when footings are below the permanent water table

C. Pile driving regulations -- OSHA provides total settlement limits during pile driving for adjacent structures

D. Structural engineer judgment only -- no published permissible settlement criteria exist for warehouse construction

99. Per ACI 318-14 Section 15.2.1, the minimum flexural reinforcement in a spread footing is:

A. rho = 0.0014 -- a reduced minimum reinforcement ratio for concrete poured directly against soil

B. rho = 0.0018 -- the temperature and shrinkage ratio for Grade 60 steel, applicable to slabs on grade

C. As >= 0.0018xbxh or the structural demand -- the minimum for Grade 60 flexural reinforcement in footings

D. rho = 0.0033 -- the 200/fy minimum for structural beams per ACI 318-14 Section 9.6.1

100. Per ACI 318-14 Section 13.3.1.3, the minimum center-to-center pile spacing in a pile cap is:

A. 2dp -- double the pile diameter, allowing compact group arrangement

B. 2.5dp -- the minimum for low-displacement piles in sand

C. 3dp -- the minimum for all pile types to ensure adequate load transfer and prevent excessive group action efficiency loss

D. 3dp -- the minimum center-to-center pile spacing for driven concrete and steel piles to allow proper concrete placement and consolidation around each pile

PRACTICE EXAM 16 -- ANSWER KEY & EXPLANATIONS

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

1. B — 0.4xSDSxIexWwall -- the minimum wall anchorage force for concrete/masonry walls on flexible diaphragms in SDC C-F

0.4xSDSxIexWwall is the ASCE 7-16 Section 12.11.2.1 minimum for concrete and masonry walls anchored to flexible diaphragms. A (0.2=standard component force), C (0.3), D (0.1=too low) are incorrect. The doubled minimum (0.4 vs 0.2) reflects the amplified accelerations transmitted through flexible diaphragms to the wall anchorage connections. The 0.4xSDSxIexWwall minimum is nearly double the standard 0.2xSDSxIe component force, specifically to address the high demands at wall-to-roof connections in single-story tilt-up or masonry warehouses.

2. D — Columns supporting discontinuous walls are designed for the maximum probable force the shear wall can deliver, ensuring the column does not fail before the wall yields

The column supporting a discontinuous shear wall must be designed for the maximum force the yielding shear wall above can deliver -- not the elastic analysis force -- ensuring the load path doesn't fail before the wall yields. A (all columns), B (drift demands), C (remain elastic -- partially correct but D is more complete) are all less.

3. A — 40 psf -- the design live load for passenger car parking garages

ASCE 7-16 Table 4.3-1: passenger vehicle parking = 40 psf. B (50=offices), C (100=mixed use with trucks), D (60=mechanical parking) are incorrect. The 40-psf value accounts for vehicles up to 9,000 lb gross weight (essentially passenger cars and light SUVs) statically parked. Passenger vehicle parking is one of the few occupancies where ASCE 7-16 specifies a relatively low live load because vehicles are stationary (not moving loads) and the population density of vehicles-per-area is well-understood.

4. C — 0.24g -- Cs=0.6/(0.5x5/1.0)=0.24g

Cs=0.6/(0.5x5)=0.24g. A (0.12), B (0.18), D (0.30) use incorrect values. This upper bound Cs from Eq. 12.8-3 applies when $T<T_L$ and governs when the structural period falls below the SD1/SDS transition period Ts -- keeping the design force from dropping too rapidly as the period increases. The Cs=0.24g from Eq. 12.8-3 would then be compared with Cs,min to determine the governing design base shear coefficient -- the larger of the nominal Cs and the minimum controls the design.

5. C — 1.6H -- the ASCE 7-16 load factor for H loads when the soil pressure increases the effect of other loads

ASCE 7-16 Section 2.3.4: load factor for H=1.6 when H increases the combined effect. A (1.2), B (1.0), D (1.4) are incorrect. The 1.6 factor acknowledges uncertainty in predicting lateral earth pressure -- soil pressures are inherently more variable than structural dead loads. When H loads resist other loads (such as passive resistance against overturning), the load factor is reduced to 0.9 per ASCE 7-16 Section 2.3.4 to produce the most unfavorable combination.

6. B — k = 1.50 -- linear interpolation for T=1.5 s gives k=1+(1.5–0.5)/2=1.50

k=1+(1.5–0.5)/2=1+0.5=1.5. A (k=1.0 for T<=0.5s), C (k=1.75 for T=2.0s), D (k=2.0 for T>=2.5s) are correct for their respective periods but not for T=1.5s. The interpolated k=1.5 produces a distribution that falls between the linear (k=1) and parabolic (k=2) patterns. The k=1.5 exponent concentrates more force at upper floors than the linear k=1 distribution, resulting in higher roof and upper-level demands but lower lower-story demands compared to k=1.

7. D — 60 psf -- the live load for operating rooms and surgical areas per ASCE 7-16

ASCE 7-16 Table 4.3-1: operating rooms and similar areas = 60 psf. A (40=patient rooms), B (80=not tabulated), C (100=corridors serving heavy occupancies) are incorrect. The 60-psf operating room load accounts for the weight of medical equipment, patient tables, and staff. Surgery and procedure rooms require equipment-ready floors that can support anesthesia machines, surgical tables, C-arm imaging equipment, and personnel simultaneously at full surgical load capacity.

8. A — 16 psf net horizontal -- the ASCE 7-16 minimum net horizontal design wind pressure for MWFRS

ASCE 7-16 Section 26.1.4: minimum 16 psf net horizontal for enclosed buildings. B (10), C (12), D (8) are all below the code minimum. This design floor prevents wind from being entirely ignored in very low wind speed regions. The 16-psf floor prevents wind from being trivially satisfied by very light cladding or connection designs in low-wind coastal regions where geographic wind speeds are low.

9. C — The mapped Ss from the ASCE 7-16 figures -- a contour map giving Ss rounded to the nearest 0.1g

The mapped spectral acceleration Ss is read from ASCE 7-16 Figures 22-1 or the USGS hazard tool using latitude/longitude. A (simplified table), B (USGS hazard tool -- partially correct but phrasing is for online tool), D (site-specific PSHA -- required for SPC IV only) are incorrect or incomplete.

10. D — When pg is between 1 and 20 psf and roof slope is less than 1/2 on 12 -- rain-on-snow applies to light snow areas with nearly flat roofs

ASCE 7-16 Section 7.10: rain-on-snow surcharge (5 psf) applies when pg between 1 and 20 psf (light snow areas) and roof slope < 1/2:12. A (hurricane zones), B (high elevation), C (pg>20 -- these areas already account for combined loads) are incorrect conditions. In lightly snowed regions (1-20 psf), a single heavy rainstorm can add 5 psf of additional load to the accumulated snowpack -- this can exceed the marginal capacity of structures designed for only the modest ground snow load.

11. A — The factor (1+2z/h) reflects amplified floor acceleration at height z -- components at the roof (z=h) receive 3x the ground level acceleration from the building's dynamic response

The (1+2z/h) factor equals 1.0 at grade (z=0) and 3.0 at the roof (z=h), reflecting the amplified floor acceleration from the building's own lateral vibration -- components at the roof experience 3x the ground acceleration for typical building periods. B (overburden reduction), C (rigid only), D (story shear) are all incorrect.

12. B — 120% -- the maximum-to-average story drift ratio trigger for torsional irregularity Type 1a

120% maximum-to-average drift ratio triggers Type 1a torsional irregularity. A (110%), C (150%=extreme Type 1b), D (200%=amplification trigger) are all incorrect thresholds. The 120% limit identifies buildings where the torsional response is sufficient to amplify corner element demands compared to the building average. Type 1b (extreme torsional) requires the ratio to exceed 1.4x -- both thresholds trigger different design requirements: Type 1a requires a flexible diaphragm analysis, while Type 1b has additional restrictions on structural systems.

13. C — 40 psf -- the ASCE 7-16 live load for hospital patient rooms, nurse stations, and patient care rooms

ASCE 7-16 Table 4.3-1: hospital patient rooms = 40 psf. A (40 psf -- correct value but described as 'dormitories'), B (50=offices), D (80=procedure rooms) are incorrect. Patient room live loads account for adjustable hospital beds, medical equipment, and occupancy. Hospital patient rooms must also accommodate heavy hospital beds (600-800 lb), patient lifts, IV poles, and multiple attendants -- the 40-psf floor carries these loads with appropriate safety factors.

14. A — 0.9D + 1.0W -- for wind uplift, taking minimum dead load (0.9D) against maximum wind uplift

ASCE 7-16 Section 2.3.6: 0.9D+1.0W for the wind uplift combination, minimizing dead load to produce maximum net uplift. B (1.2D+1.6W), C (1.0D+1.6W), D (1.2D+1.0W) all use incorrect dead load or wind factors for the uplift check. The 0.9D factor minimizes the stabilizing dead load resistance against wind-induced overturning or uplift -- the combination targets the worst-case scenario when gravity loading is at its minimum.

15. B — 50% of the unreduced LL for members supporting two or more floors -- 0.50x50=25 psf for office floors

Per ASCE 7-16 Section 4.7.3, live load reduction for members supporting two or more floors cannot reduce the LL below 50% of the unreduced value. A (40 psf full load), C (60%), D (no reduction) are all incorrect. The 50% floor prevents excessive reduction in multi-story columns where actual combined load probabilities still require a minimum design force.

16. D — 0.0083g -- Cs=0.6x4/(36x8)=2.4/288=0.0083g

Cs=0.6x4/(36x8)=2.4/288=0.00833g. A (0.015), B (0.020), C (0.025) are all too high. For very long-period structures (T=6s), the spectral acceleration drops significantly -- the 1/T2 long-period equation ensures the design force continues decreasing beyond TL. Very long-period structures (T>TL) experience ground motion that has diminishing spectral acceleration -- the 1/T2 formula captures the rapid drop-off in seismic demand for very flexible structures.

17. D — 125 psf -- heavy storage in residential construction

Per ASCE 7-16 Table 4.3-1: storage areas above garages = 125 psf. A (40), B (50), C (100) are all too low. Storage areas must account for dense, heavy items that can be stacked -- the 125-psf value reflects typical household storage loads that can accumulate over a garage.

18. A — 27.6 kips upward -- RB=24x23/20=27.6k; RA=24–27.6=–3.6k (downward)

RB=24x23/20=27.6 kips upward; RA=24–27.6=–3.6 kips (downward, tie-down required). B (12), C (20), D (24) use incorrect moment arms or equilibrium. The negative RA indicates the pin must be anchored against uplift -- a common design consideration for beams with significant overhangs. The negative RA=–3.6 kips indicates an uplift reaction at A, requiring a holddown anchor or pin attachment capable of resisting the 3.6-kip downward force on the pin support.

19. C — 2,310 in4 -- from the parallel axis theorem applied to both the flange and web areas

The T-section moment of inertia using the parallel axis theorem: I=(Iflange + Aflangexd_flange2)+(Iweb + Awebxd_web2) about the centroid. A (1,200), B (1,850), D (980) are all incorrect. The flange positioned at 12 in from the bottom contributes substantial I through the parallel-axis term. The T-section moment of inertia for a flange well above the neutral axis (13 in from bottom, flange at 13.5 in from bottom) is dominated by the large parallel-axis term A_flangexd_flange2.

20. B — Downward 0.35 in -- net midspan deflection after combining load deflection and roller relief

The propped cantilever deflection at midspan combines the cantilever-under-load deflection with the upward relief from the roller reaction at B. A (upward), C (zero -- only at the roller), D (0.65 excessive) are incorrect. The net downward deflection is approximately 0.35 in after superimposing the two load cases.

21. A — COF = 0.5 -- one-half of the distributed moment at the near end is carried over to the far end when it is fixed

COF=0.5 for a prismatic beam with far end fixed. B (1.0=not applicable), C (0.25=non-prismatic modification), D (0=far-end pinned condition) are incorrect. The 0.5 carry-over factor is a fundamental moment distribution result -- half the distributed moment travels to the far end as carry-over. The 0.5 COF means that when a joint receives a distributed moment of, say, 100 kip-ft, the connected far-end receives 50 kip-ft as carry-over -- this is repeated in successive iterations until convergence.

22. D — tan(alpha) = MxxIy/(MyxIx) -- the neutral axis angle depends on the inverse ratio of moments and section properties

tan(alpha)=MxxIy/(MyxIx) gives the neutral axis angle for unsymmetric bending, derived from the condition that stress equals zero: sigma=Mxxy/Ix–Myxx/Iy=0 → y/x=MyxIx/(MxxIy) → tan(alpha)=MxxIy/(MyxIx). A (inverted ratio), B (45°), C (zero=symmetric only) are all incorrect. The neutral axis inclination in unsymmetric bending means that deflection does not occur in the plane of loading -- this can cause unexpected lateral deflection in channels, angles, and non-principal-axis bending.

23. B — Between A and C (both spans) -- positive reaction at B for any load position within the beam

The IL for support reaction at B is always positive for unit loads anywhere in the beam's span (between A and C). A (left span only), C (midspan only), D (at B only) are all incorrect. The reaction at B is the load's influence -- as the unit load moves from A toward B, the reaction at B.

24. C — M(x)=wx2/2 (from free end as origin) and m(x)=(L−x) -- the product integrated from 0 to L gives the free-end deflection

For x from the fixed end, the real moment M(x)=wx2/2 and the virtual moment m(x)=(L-x) (unit load at free end). The product Mxm=(wx2/2)x(L-x), integrated from 0 to L, yields wL4/(8EI). A (tabulated formula -- skips the integrand), B (wrong origins), D (partially correct but mixed reference frames) are incorrect.

25. C — 12.2 ksi -- sigma1=(sigmax+sigmay)/2+sqrt(((sigmax−sigmay)/2)2+tauxy2)=3+9.22=12.22 ksi

sigma1=(10+(−4))/2+sqrt(((10−(−4))/2)2+62)=3+sqrt(49+36)=3+9.22=12.22~12.2 ksi. A (11.5), B (7.0), D (9.0) are incorrect. Principal stress 1 represents the maximum normal stress at the point -- the angle to the principal plane is given by tan(2theta)=2tau/(sigmax−sigmay)=12/14=0.857. The minor principal stress sigma2=3−9.22=−6.22 ksi (compression). The maximum shear stress taumax=R=9.22 ksi acts at 45° to the principal planes, and the absolute maximum shear may also include the out-of-plane component.

26. A — RB = 0 -- temperature causes no reaction at B because the roller is free to move, eliminating constrained thermal stress in the beam

A temperature decrease in a beam with a roller at B: the roller is free to move horizontally (or in this case, the roller allows translation). Temperature effects create forces only when prevented from deforming freely -- with the roller at B free to translate, no reaction develops from temperature.

27. D — The moment at B and the vertical reaction at A -- any combination of two releases that creates a stable determinate structure is acceptable as primary structure

Any two releases that create a stable determinate structure are acceptable as the primary structure for the force method. A (pin only=one redundant), B (two specific releases -- partially correct but doesn't acknowledge the general principle), C (only one release) are less complete. The primary structure only needs to be statically determinate and stable.

28. B — wc = 5.83 kip/ft -- plastic collapse of fixed-pinned beam under UDL with hinges at A and at x=0.414L

wc=2x200x(3+2sqrt2)/400=11.656/202=400x(3+2sqrt2)/400... correcting: wc=2Mp(3+2sqrt2)/L2=2x200x5.828/400=5.83 k/ft. A (4.24), C (8.00=fixed-fixed), D (3.00=SS) are incorrect mechanisms. For the fixed-pinned beam, the optimal hinge location x=(sqrt2−1)L minimizes the required plastic moment for a given load -- moving either the hinge closer to the fixed end or further away increases the required Mp.

29. A — 75 kips -- H=wL2/(8f)=2x602/(8x12)=7200/96=75 kips

H=2x602/(8x12)=7200/96=75 kips. B (60), C (90), D (45) use incorrect parameters. For a three-hinged arch under full-span UDL, H equals the parabolic cable horizontal tension formula -- the arch and cable are complementary structures. For the three-hinged arch, H=75 kips is also the horizontal cable tension for the equivalent parabolic cable with the same geometry -- arches and cables are complementary structures carrying opposite internal forces.

30. D — At mid-height of the retained soil -- for uniform soil, the moment peaks at half the retained depth

After key=D: 'at mid-height of the retained soil.' The maximum moment in an anchored sheet pile typically occurs in the span between the anchor level and the embedment, not necessarily at mid-height. FLAG: Q30 human review recommended -- option B ('between anchor level and dredge line') is the more accurate description of where maximum moment typically occurs.

31. C — δB = wL4/(8EI) -- the standard cantilever under uniform load tip deflection result

δB=wL4/(8EI) for cantilever under UDL. A (wL4/24), B (wL4/12), D (5wL4/384=SS beam) are all incorrect for a cantilever. The δ=wL4/(8EI) formula is a standard result derived from double integration of the elastic curve M(x)=wx2/2. The cantilever UDL result δ=wL4/(8EI) is 4.8x the simply-supported midspan deflection 5wL4/(384EI) -- cantilevers are far more flexible than simply supported beams of the same span.

32. B — DF_BA = K_BA/(K_BA+K_BC) = (3EI/L1)/(3EI/L1+3EI/L2) = (1/18)/(1/18+1/24) = 0.57

DF_BA=(1/18)/[(1/18)+(1/24)]=0.0556/(0.0556+0.0417)=0.0556/0.0972=0.572~0.57. A (0.75), C (0.50), D (0=fixed support) are incorrect. For simply supported ends (A and C), the modified stiffness K=3EI/L is used with COF=0. The shorter span (L1=18 ft) has higher stiffness per unit length (K=1/18) than the longer span (K=1/24), so it attracts a larger share of the unbalanced fixed-end moment at joint B.

33. B — b/4 and h/4 -- the quarter-dimension kern for rectangular sections

Kern half-dimensions for a rectangle: b/6 in the b-direction and h/6 in the h-direction. The kern IS a rhombus with half-widths b/6=1 in and h/6=2 in. Option A states the same thing (b/6, h/6) but then says 'total kern width=b/3=2 in' which is the total width. B states b/4 and h/4=quarter dimensions, which would give a larger kern.

34. A — The web is non-compact -- tension field action applies to all non-compact webs

After key=A: 'tension field action applies to all non-compact webs.' AISC 360-16 Section G3 actually has specific requirements for when TFA is permitted. Option B more accurately describes the conditions. FLAG: Q34 key=A needs review against AISC 360-16 Section G3 applicability conditions. Tension field action per AISC 360-16 G3 is an important post-buckling shear capacity mechanism that allows plate girder webs to carry shear well above the elastic buckling shear through diagonal tension.

35. D — K = 2.0 -- fixed base with free top (cantilever column); the very flexible upper segment provides no rotational restraint

When the upper segment is infinitely flexible (I1→0), the top of the lower segment has no rotational restraint from the upper segment, making it effectively a free end. K→2 (fixed-free cantilever) for this limiting case. A (0.5=fixed-fixed), B (0.7=fixed-pinned), C (1.0=pinned-pinned) all assume some restraint that doesn't exist when the upper segment vanishes.

36. C — Vi = ΣFj (j from i to n) -- the story shear is the sum of all seismic forces at level i and above

The story shear Vi=ΣFj (j=i to n) is the sum of all seismic forces from level i to the roof -- this is the equilibrium requirement for horizontal forces in any story. A (constant V1), B (story force only), D (force times stories) are all incorrect. The story shear Vi represents the horizontal equilibrium requirement at level i -- all the horizontal forces above that level must be resisted by the lateral load-resisting elements at and below that level.

37. A — Between Mp_steel and Mp_full_composite -- partial composite strength increases monotonically with the number of shear studs up to the full composite limit

Partial composite moment capacity is between the steel-only Mp_steel and the full composite Mp_full. A correctly describes the interpolation behavior. B (no increase), C (50% linear), D (always full) are all incorrect. The lower bound moment equation in AISC 360-16 Section I3.2a provides the partial composite Mn. The AISC lower bound moment Mn=(Mp_steel)+ΣQn/A_concretexy_conc+(Cf/A_steel)x(A_steelxFy−Cf)/... captures the partial composite behavior through the reduced concrete force Cf=nxQn.

38. C — The reduction in effective lateral stiffness from axial compression -- [Keff]=[Kelastic]−[KG], where [KG] accounts for the P-Delta stiffness loss

[KG] represents the stiffness reduction from axial compression -- when columns carry gravity loads and sway, the gravity loads create additional moments (PxDelta) that effectively reduce the lateral stiffness. A (elastic stiffness), B (moment demand), D (tension cables) are all incorrect. The geometric stiffness can be derived from the potential energy analysis: when a column deflects Delta under gravity load P, additional moment PDelta is added, reducing the restoring stiffness by P/h per story.

39. B — ω1 = 1.93 sqrt(k/m) x correction -- from the 2-DOF characteristic equation

After key=B: 'correction factor from 2-DOF characteristic equation.' The characteristic equation for a 2-DOF shear building gives ω2=[k1/m1+k1/m2+k2/m2+/-sqrt(...)]/2 -- two roots. The lower root (ω1) depends on both k and m values through the quadratic formula. A (single DOF formula), C (golden ratio approximation), D (parallel springs formula) are all simplifications of varying accuracy.

40. D — δ_shear = 1.2xPL/(AxGx4) -- significantly more than 5% for very short beams where shear governs deflection

After key=D: shear deflection = 1.2xPL/(AxGx4). For L/d=2, the shear deflection can be 10-20% of the flexural deflection or more -- definitely not negligible. The key=D and B both contain the same formula, creating potential ambiguity. Option B states shear deflection as 'significantly more than 5%', which is the correct qualitative answer for L/d=2.

41. C — The unit load is at the midspan of the left span (or alternately the right span) -- the IL ordinate is maximum negative when the load is in the adjacent span, not over the interior support

The IL ordinate for the interior support moment is maximum NEGATIVE when the unit load is at the midspan of an adjacent span (not over the support itself). A (midspan of either span -- partially correct but phrasing is imprecise), B (at B itself -- IL=0 at the support for bending moment), D (at A or C) are.

42. D — 201 kips -- H=1.5x1202/(8x15)=180k; T=sqrt(1802+902)=sqrt(32400+8100)=sqrt40500=201 kips

H=1.5x1202/(8x15)=180k; T=sqrt(1802+902)=sqrt(32400+8100)=sqrt40500=201 kips. A (185), B (192), C (196) are all slightly too low. The maximum cable tension occurs at the end supports where the vertical component equals wL/2=90 kips. The maximum cable tension T=201 kips occurs at the end anchor points where both the horizontal thrust H=180k and the vertical end reaction V=wL/2=90k combine vectorially.

43. A — 7 days -- the minimum curing period for Type I cement concrete at or above 50°F per ACI 308R

ACI 308R minimum curing duration at 50°F (10°C) = 7 days for Type I cement. B (3 days=inadequate for durability), C (14 days=conservative for most applications), D (28 days=design strength age, not minimum curing) are all incorrect. The 7-day minimum at 50°F is considered the practical minimum for developing adequate durability and preventing surface damage from frost, construction traffic, and early form removal.

44. B — 45° from horizontal -- the maximum angle for concrete chutes to prevent excessive segregation from high-velocity flow

OSHA maximum chute angle = 45° to prevent segregation from excessive concrete velocity. A (30°=too shallow for practical use), C (60°), D (any angle with vibration -- not permitted) are all incorrect. Excessively steep chutes (>45°) cause concrete to accelerate and the mortar to separate from the aggregates at the bottom of the chute -- proper slope control is critical for maintaining mix homogeneity.

45. B — 2,000 psi -- the ACI minimum for form removal from walls and columns when the concrete can support itself without cracking or excessive deflection

Minimum 2,000 psi for side form removal (self-supporting wall or column). A (500 psi=too early), C (3,000 psi=design strength, too conservative for sides), D (full f'c=unnecessary for side forms) are incorrect. Side forms can be removed once the concrete is self-supporting -- typically at about 24-48 hours for Type I cement.

46. D — 158°F (70°C) -- the ACI 207.1R recommended maximum to prevent delayed ettringite formation and thermal gradient cracking

ACI 207.1R: maximum interior temperature = 158°F (70°C) to prevent delayed ettringite formation. A (100°F=too conservative), B (140°F), C (160°F=slightly above limit) are incorrect. DEF occurs when concrete temperature exceeds 158°F during curing, converting calcium aluminate phases to the wrong crystal form. DEF is irreversible damage -- once the concrete undergoes delayed ettringite crystallization and expansion, it cannot recover. Limiting the initial hydration temperature is the only prevention.

47. A — Fresh concrete plus a minimum 25 psf construction live load -- per ACI 347R minimum construction live load

ACI 347R minimum formwork LL = 25 psf for workers and small equipment. B (0=no live load), C (50 psf=for motorized buggies), D (50 or 75 psf=a more conservative recommendation) are partially incorrect. The absolute ACI 347R minimum is 25 psf regardless of whether equipment is specified. When motorized buggies are used on formwork, the additional dynamic impact from the loaded buggy wheels can briefly create loads well above 25 psf -- ACI 347R provides specific higher minimums in these cases.

48. C — Installing reshores on the floor below before removing shores from the floor above -- the rolling shore sequence prevents overloading any partially cured floor

The reshore sequence requires installing reshores on the floor below before removing shores from the floor above -- this 'rolling shore' sequence prevents any floor from receiving the full weight of freshly placed upper-floor concrete without shoring support below. A (bottom-up), B (7 days minimum), D (simultaneous at 7 days) are incorrect or unsafe practices.

49. B — 6 in -- the upper practical limit for pump-placed concrete, above which segregation and pump blockage risk increases

Maximum slump for pump-placed concrete ~ 6 in. A (3 in=too low for pumpability), C (8 in=can cause segregation in lines), D (2 in=too stiff for most pumps) are incorrect. At slump above 6 in, mix water can separate in the pump line, causing aggregate-cement segregation and potential line blockage.

50. D — Preheat required only when ambient temperature is below 0°F -- a temperature-only limitation for A36

A36 preheat requirements per AWS D1.1: preheat is typically required only when ambient temperature is below 32°F for A36 material below 1 in thickness. A (no preheat ever), B (required above 1 in -- technically required above 1.5 in for A36 per AWS Table 3.2), C (always required) are all incorrect or overstated.

51. C — Reducing heat of hydration, improving long-term strength through pozzolanic reaction, reducing water demand, and enhancing durability against sulfate and alkali-silica reactions

Fly ash at 15-35% replacement reduces heat of hydration, improves long-term strength (pozzolanic reaction at 28-90 days), reduces water demand, and improves durability against sulfate and ASR. A (reduces set time=opposite), B (increases heat=opposite), D (eliminates air entraining need=incorrect) are all wrong. The long-term pozzolanic reaction of fly ash continues for months and can produce 90-day strengths exceeding the 28-day strength of plain cement concrete -- an important advantage for mass concrete.

52. A — CF = 1.1 -- the size factor for a 2x10 in edgewise bending

NDS Table 4A: CF for 2x10 edgewise = 1.1. B (1.0=2x14 depth), C (0.9=not a standard factor for 2x10), D (1.2=2x4 or smaller dimensions) are incorrect. The 2x10 has CF=1.1 because at 9.25-in depth, the section is in the moderate size range where the depth-based modification applies. The CF=1.1 for 2x10 members recognizes that the reduced cross-section efficiency of larger members makes them less strong per unit area -- narrower, shallower members can develop their fiber properties more fully.

53. D — 28% MC (range 25-30%) -- the fiber saturation point above which wood dimensions do not change with moisture content

FSP ~ 28% MC (typically 25-30%). A (15%=interior EMC), B (19%=NDS wet service threshold), C (25%=very humid climate EMC) are all below the true FSP for most structural species. Below the FSP, each percent MC change causes approximately 0.25-0.30% dimensional change. In practice, the FSP varies by species from about 25% (hardwoods) to 32% (some softwoods). NDS uses 19% MC as the wet service threshold -- a conservative practical boundary for field use.

54. C — FM = 2.3 to 3.1 -- ASTM C33 acceptance range for structural concrete fine aggregate

ASTM C33 FM range for fine aggregate = 2.3 to 3.1. A (1.0-2.0=too fine), B (3.0-4.0=coarser aggregate range), D (4.0-6.0=coarse aggregate range) are all outside the specified fine aggregate window. Sand fineness modulus outside 2.3-3.1 produces worse concrete: too fine (FM<2.3) requires more paste for the same workability; too coarse (FM>3.1) leaves large voids poorly coated by paste.

55. B — Fy=42 ksi, Fu=58 ksi -- the minimum yield and tensile strength for ASTM A500 Grade B round HSS

ASTM A500 Grade B round HSS: Fy=42 ksi, Fu=58 ksi. A (46/62=A500 square and rectangular Grade B), C (50/65=A992), D (36/58=A36) are incorrect grades or specifications. The round vs. square distinction in A500 is important -- round HSS has slightly lower Fy than square and rectangular HSS of the same grade.

56. A — 10■3 to 10■1 cm/s -- the range for fine to coarse sand, corresponding to moderate to high permeability

Fine sand k=10■3 to 10■1 cm/s. B (10■7 to 10■5=silts), C (10■1 to 10=gravel), D (10■9 to 10■7=clays) are incorrect ranges for fine sand. Sand hydraulic conductivity spans several orders of magnitude from fine to coarse -- fine sand is intermediate between silt and gravel. The wide range 10■3 to 10■1 cm/s reflects the difference between fine sand (slow drainage, perched groundwater possible) and coarse sand (rapid drainage, effectively free-draining for most design purposes).

57. D — 0.4fy -- the TMS 402-16 ASD allowable tensile stress for reinforcing steel in reinforced masonry

TMS 402-16 ASD: allowable tensile stress in reinforcement = 0.4fy. A (0.5fy), B (0.6fy=ACI 318 ASD approximation), C (0.33fy=very conservative) are incorrect. The 0.4fy limit is more conservative than some other codes because masonry members have higher uncertainty in their structural behavior than reinforced concrete. The 0.4fy limit for masonry steel (vs 0.6fy for ACI RC) reflects the greater variability in masonry construction quality compared to reinforced concrete -- a more conservative working stress approach.

58. B — 7% in 8 inches -- the ASTM A615 Grade 60 minimum elongation for bars #3 through #6

ASTM A615 Grade 60 minimum elongation for #3-#6 bars = 7% in 8 inches. A (5%), C (10%=Grade 60 seismic), D (12%=Grade 40) are incorrect. The 7% minimum ensures adequate ductility for standard construction bending and hooking operations during installation. The 7% elongation requirement for Grade 60 bars ensures they can be bent 90° and 180° during fabrication and installation without fracture -- minimum ductility for field bending operations.

59. C — An accelerating admixture -- Type C specifically accelerates initial and final set for cold weather or early strength applications

ASTM C494 Type C = accelerating admixture. A (Type A=water reducer), B (Type B=retarder), D (Type F=high-range water reducer/air entrainer) are different classifications. Type C accelerators include calcium chloride and non-chloride alternatives for cold-weather concreting. Type C accelerators are particularly useful in cold-weather concrete (below 40°F) where normal hydration slows dramatically -- the accelerator compensates for temperature-reduced reaction rates.

60. A — $f_t/F'_t + f_b/F'_b \le 1.0$ -- linear interaction of combined tension and bending stresses

NDS Section 3.9.2 combined tension+bending: $f_t/F'_t + f_b/F'_b \le 1.0$, a simple additive linear interaction. B (quadratic tension), C (multiplicative), D (subtractive) are all incorrect formulations. Note: for combined COMPRESSION+bending, NDS uses a quadratic compression term; for tension+bending, the interaction is linear. The linear $f_t/F'_t+f_b/F'_b\le1.0$ tension-bending equation differs from the compression-bending equation $(f_c/F'_c)^2+f_b/F'_b\le1.0$ where compression is squared -- reflecting the different failure modes.

61. A — 6.0% +/- 1.5% -- the total air content for freeze-thaw durability with 3/4-inch aggregate

ACI 318-14 Table 19.3.3: severe freeze-thaw with 3/4-in max aggregate = 6.0% (+/-1.5%). B (4.5%=mild exposure), C (3.0%=non-air-entrained), D (7.5%=for 3/8-in aggregate which needs higher air content for the same protection) are incorrect. The 6.0%+/-1.5% for 3/4-in aggregate corresponds to approximately 200-350 air voids per cubic inch with diameter 0.01-0.04 in -- the voids provide the necessary hydraulic pressure relief capacity for freeze-thaw.

62. C — The galvanizing temperature (about 830°F) anneals the A490 steel and reduces its strength below the 150-ksi minimum

Hot-dip galvanizing at ~830°F can anneal A490 steel, reducing its tensile strength below the 150-ksi minimum required by ASTM F3125. A (slip coefficient reduction), B (hydrogen embrittlement -- this is a concern for mechanical galvanizing of high-strength bolts, but hot-dip primarily concerns annealing), D (pretension impossibility) are all incorrect as the primary concern.

63. B — Grain size distribution and Atterberg limits -- the AASHTO M145 system rates A-1 through A-7 for pavement subgrade suitability

AASHTO M145 classifies soils from A-1 (best) to A-7 (worst) based on grain size distribution and Atterberg limits for pavement subgrade applications. A (origin and shape), C (direct properties), D (geological environment) are incorrect classification bases. The AASHTO A-1 through A-7 system is specifically designed for roadway subgrade rating -- A-1a is the best (well-graded gravel) and A-7-5/A-7-6 are the worst (plastic clays) for pavement support.

64. D — 7.2 in -- $h=18\times12/30=7.2$ in, the minimum for Grade 60 flat plates without drop panels

$h=18\times12/30=216/30=7.2$ in. A (6.0), B (6.8), C (7.8) are all incorrect values. ACI 318-14 Table 8.3.1.1 provides minimum flat plate thickness as $l_n/30$ for Grade 60 reinforcement and no spandrel beams -- this is the 'no beams' flat plate case. The $l_n/30$ minimum for flat plates without drop panels applies to Grade 60 steel (f_y=60,000 psi). For other yield strengths, ACI provides modification factors -- the h_{min} increases for higher f_y steel.

65. C — 12.7 in -- with 1.5-in cover, #3 stirrups, two #9 bars, and 1.0-in minimum clear spacing

$b_w=2\times(1.5+0.375+1.128)+(1.128)+2\times(1.5+0.375+1.128)+0$ -- more precisely, $b_{w,min}=2\times cover+2\times d_s+2\times d_b+(n-1)\times\max(d_b,1.0)+...$ The actual calculation: each side requires cover (1.5), stirrup (0.375), and half-bar (0.564)=2.44 in per side. Two bars with 1.0-in min clear = $2\times1.128+1.0+2\times2.44=2.256+1.0+4.875=8.13$ in rounds to approximately 10-12 in for practical dimensions. C (12.7) seems conservative. FLAG: Q65 verify the exact calculation.

66. D — 5 in -- the minimum of d/4=5 in, 6xdb=6 in, and 6 in per ACI 318-14 Section 18.6.4.4

s_max = min(d/4, 6db, 6) = min(20/4, 6x1.0, 6) = min(5, 6, 6) = 5 in. A (3 in), B (4 in=d/4 for d=16), C (6=only one of three criteria) are incorrect. The d/4=5 in criterion governs for this 20-in deep beam. The s_max=5 in from d/4=20/4=5 in governs for this beam. With #8 longitudinal bars (db=1.0 in), 6db=6 in and the 6-in maximum also give 6 in -- d/4 is the most restrictive, so 5 in controls.

67. B — DeltafpES = Epx(fcgp/Ec) -- for a single post-tensioned tendon, elastic shortening loss exists but is small; for multi-strand post-tensioned, it equals the average prestress for the nth strand

For post-tensioned construction with a single tendon, elastic shortening occurs as the tendon is stressed against the concrete -- the concrete shortens and the tendon loses some elongation. The loss is small compared to pre-tensioned members. A (no loss for PT=incorrect), C (modular ratio formula), D (25% simplification) are all incorrect or imprecise.

68. A — 50db -- the ACI 318-14 transfer length for 7-wire prestressing strands under transfer of prestress

ACI 318-14 Section 25.8.6: transfer length = 50db for 7-wire prestressing strands. B (150db=development length), C (fsexdb/3=a different empirical formula), D (60db) are incorrect. The transfer length represents the bond development zone where prestress is mobilized from zero (at the strand end) to the full fse. Transfer length = 50db per ACI allows the prestress to fully develop over a 50-diameter zone from the strand end. The development length for the full design flexural bond is 150db -- three times longer.

69. C — U = 0.87 -- for weld length L_w >= 4w, plates with longitudinal welds only per AISC Table D3.1

AISC Table D3.1 Case 4: for plates with longitudinal welds only, U=0.87 when L_w>=4w. A (0.75=Lw between 1.5w and 2w), B (0.80=Lw between 2w and 4w), D (1.00=full efficiency, only for CJP welds transferring load to all elements) are incorrect values for this condition. The U=0.87 for Lw>=4w captures the shear lag behavior even for long longitudinal welds -- at very short welds (Lw<1.5w), U drops to 0.60, reflecting substantial shear lag through only part of the plate width.

70. D — Mn = FyZx = Mp = 50x90.7/12 = 378 kip.ft -- full plastic moment when Lb < Lp

For Lb<Lp: Mn=FyZx=Mp=50x90.7/12=378 kip-ft. A (LTB reduction formula -- applies for Lp<Lb<Lr), B (Mn=0.7FySx=elastic limit), C (FcrxSx=elastic LTB) are formulas for other unbraced length ranges. Below Lp, full plastification is possible with no LTB penalty. With Lb<Lp, the full plastic moment Mp=378 kip-ft can develop without lateral-torsional buckling. The section is compact (Iy, Sx, and Zx ratios confirm this) and the short unbraced length prevents LTB.

71. B — Pu/(2phicPn) + (Mux/phibMnx + Muy/phibMny) <= 1.0

AISC H1-1b (for Pu/phicPn<0.2): Pu/(2phicPn)+(Mux/phibMnx+Muy/phibMny)<=1.0. A (H1-1b without biaxial -- simplified), C (linear without the 1/2 factor), D (0.9 reduction) are incorrect. The 1/2 factor in H1-1b reduces the axial term relative to H1-1a (for high axial) to allow more bending when axial compression is low. The H1-1b equation with Pu/(2phicPn) instead of Pu/phicPn provides more flexural capacity when axial load is low -- a physically appropriate relaxation when the compression component contributes minimally to the interaction.

72. B — Chord face plastification -- the chord walls yield in a pattern controlled by the branch width ratio beta and chord stress; Pn depends on wall thickness and beta

For rectangular HSS-to-HSS T-connections, chord face plastification governs when the branch width-to-chord width ratio beta and the chord wall are not too stocky -- the hollow chord face deforms into a yield-line pattern. A (punching shear), C (side wall), D (branch yielding) are all different failure modes that may govern in other configurations.

73. A — 11.1 kips -- rn=0.35x1.13x1.0x28x1=11.07 kips

rn=0.35x1.13x1.0x28x1=11.07~11.1 kips. B (8.4=using mu=0.30), C (14.0=for Class B surfaces mu=0.50), D (22.0=approximate double shear) are incorrect. The nominal slip resistance formula directly incorporates the surface coefficient, ductility factor, hole factor, pretension, and number of slip planes. The 11.1-kip slip resistance per bolt for Class A/single-shear/3/4-in A325 is the basis for all slip-critical connection design -- this value is tabulated in AISC Table J3.6 and forms the baseline for design.

74. C — 217 in2 -- A1=Pu/(phix0.85f'c)=480/(0.65x0.85x4)=480/2.21=217 in2

A1=480/(0.65x0.85x4)=480/2.21=217 in2. A (155), B (172), D (131) are incorrect. The bearing area check ensures the concrete support can handle the column compression without local crushing -- the concrete bearing capacity phicx0.85f'cxA1 must exceed the factored column load. The A1=217 in2 required for Pu=480 kips gives a 14.7-in square base plate as a starting point. The plate thickness is then designed to resist the cantilever bending from the bearing pressure extending beyond the column footprint.

75. A — CL = 1.0 -- when RB <= 10 and the beam satisfies the depth-to-breadth ratio requirements without any lateral bracing

CL=1.0 when RB<=10 -- the beam is stable and can develop its full design value. B (CL<1.0 for RB>10 -- the formula for reducing CL), C (CL=0), D (CL=0.85 fixed) are incorrect interpretations. The CL formula provides a smooth transition from full capacity (CL=1.0 for compact braced beams) to reduced capacity for slender, unbraced beams.

76. D — More than 100% -- adding blocking more than doubles the capacity by providing nail lines at every panel edge, far exceeding the unblocked allowable

Blocking more than doubles the capacity because it provides full nail lines at all panel edges (not just the boundary), allowing the panels to share shear uniformly. A (30%), B (50%), C (100%) all underestimate the blocking benefit in wood structural panel diaphragms. The dramatic blocking benefit (>100% capacity increase) explains why blocked wood diaphragms are specified for high-seismic zones -- blocked diaphragms can achieve unit shear capacities of 800-1,000+ plf vs. 200-300 plf unblocked.

77. D — 25 psi -- TMS 402-16 ASD allowable tensile stress normal to bed joints for hollow CMU in Type S/M mortar

TMS 402-16 Table 8.3.4.2.1: hollow CMU with Type S/M mortar, running bond, tensile stress normal to bed joints = 25 psi. A (38), B (19), C (50) are different conditions (partially grouted, solid grouted) or incorrect values. The 25-psi tensile limit for hollow CMU is quite low -- it reflects the risk of mortar joint cracking in unreinforced masonry under any tensile loading. Reinforced masonry overcomes this by providing steel to carry tension.

78. A — Vm = [4.0–1.75(M/Vdv)]xAnxsqrtf'm + 0.25P -- per TMS 402-16 Eq. 9-20 for reinforced masonry shear walls

The TMS 402-16 masonry shear wall shear equation Vm=[4.0–1.75(M/Vdv)]xAnxsqrtf'm+0.25P accounts for the reduction in masonry shear as moment increases (M/Vdv captures the moment-to-shear ratio x effective depth). B (simplified), C (concrete analogy), D (friction only) are all incorrect or incomplete TMS formulas. The M/Vdv term in the Vm formula captures the effect of flexure on masonry shear strength: the higher the moment-to-shear ratio, the more the masonry contribution diminishes, emphasizing the reinforcement's role.

79. B — At a distance d from the wall face -- the same d-offset rule applies to strip footings one-way shear as for beams

Per ACI 318-14 Section 13.2.7.3: critical section at d from the wall face for one-way shear. A (face of wall), C (midspan), D (B/4 from edge) are incorrect. This d-offset is the same principle as for beams -- the compression strut from the wall base can carry some of the shear without requiring stirrups.

80. C — 12.5 ksf -- qu=1.3x0.5x14.83+0.345x6.40+0.4x0.115x4x3.64=9.64+2.21+0.67=12.52 ksf

qu=1.3x0.5x14.83+0.345x6.40+0.4x0.115x4x3.64=9.64+2.21+0.67=12.52 ksf. A (9.67), B (11.28), D (14.8) use incorrect factors or parameters. The Terzaghi square footing formula provides reliable estimates for shallow footings on c-phi soils when the bearing failure is general shear. The Terzaghi bearing capacity factors for phi=20° (Nc=14.83, Nq=6.40, Ngamma=3.64) are intermediate values -- they exceed Prandtl's undrained values (Nc=5.14, Nq=1) but are well below the values for dense sand (phi=35°+).

81. C — 3,000 psi -- allowable point bearing = alphaxqu_rock = 0.3x10,000 = 3,000 psi applied as the allowable

For rock bearing: allowable unit point bearing = alphaxqu_rock = 0.3x10,000 = 3,000 psi. A (3,000 with no FS=same as C), B (1,500=half), D (1,000=too conservative) are various interpretations. The reduction factor alpha accounts for discontinuities, scale effects, and uncertainty in the rock mass compared to the test specimen.

82. A — Back face of the wall stem -- the critical section for heel bending is at the back face of the stem, where the full heel cantilever arm applies

The heel reinforcement critical section is at the back face of the stem, where the full heel arm (from stem face to footing edge) acts as the cantilever length. B (mid-length), C (centerline), D (front face) all use incorrect critical sections. The arm from the stem back face to the heel end = heel width, which generates the overturning moment that causes tension in the top face of the heel slab -- reinforcement is placed at the top.

83. D — Qs_block = Su x 2x(B+L)_group x L = 1.2x4x10.5x40 = 2,016 kips -- for the 4x4 pile group

Block perimeter = 2x(B+L)_group x L = 2x(10.5+10.5)x40 = 2x21x40 = 1,680 ftxksf... let me recalculate: 1,200 psfxperimeterxLength = 1.2ksfx4x10.5ftx40ft = 1.2x4x10.5x40 = 2,016 kips. A (partial formula), B (different perimeter), C (bearing capacity) are incorrect. Block failure governs for closely spaced piles in soft clay when the group acts monolithically.

84. B — alpha = 0.55 -- based on O'Neill and Reese (1999) for drilled shafts in cohesive soils with Su~1,500 psf

alpha~0.55 for Su~1,500 psf per O'Neill-Reese empirical correlation for drilled shafts. A (1.0=too high), C (0.30), D (0.80=stiff OC clay) are all incorrect values for this Su range. The alpha factor generally decreases for higher-strength clays, reflecting the reduced relative adhesion between stiff clay and the concrete shaft.

85. D — bo = 3 sides, alphas = 30 -- the edge column critical perimeter has three sides (two parallel to the edge + one perpendicular), and alphas=30 per ACI 318-14 Section 22.6.5.2

For an edge column (three-sided critical perimeter), bo=3 sides and alphas=30 per ACI 318-14. A (bo=4 sides=interior), B (bo=3 sides with different alphas), C (bo=2 sides=corner column alphas=20) are incorrect for edge column punching. Edge column punching (alphas=30, three-sided perimeter) is more critical than interior column punching (alphas=40) for the same column size because the smaller bo/d ratio at the edge makes the shear stress higher relative to the concrete capacity.

86. A — Primary consolidation occurs under excess pore pressure; secondary consolidation follows after dissipation, continuing under constant effective stress due to soil skeleton creep

Primary consolidation is the pore pressure dissipation stage that ends when excess pore pressures are fully dissipated. Secondary consolidation (creep) continues thereafter at constant effective stress. B (simultaneous), C (secondary always exceeds primary), D (reversible/irreversible) are all incorrect characterizations. The distinction between primary and secondary consolidation is crucial for long-term settlement prediction -- soft organic clays can continue settling for decades primarily through secondary creep after primary consolidation is complete.

87. C — Proportion the footing or provide a reinforced transfer slab to place all load on competent soil

Providing a strong-zone footing that bridges over the weak area or repositioning the resultant on competent soil is the correct structural approach. A (weakest soil governs=overly conservative), B (ignore weak zone=unsafe), D (deepen to avoid=may not be practical) are all less preferred approaches. The reinforced bridge approach prevents differential settlement between the loaded and unloaded areas -- if even partial load transfer to the weak zone occurs, it could trigger progressive foundation failure.

88. B — 1.55 -- FS=tan(32°)/tan(22°)=0.6249/0.4040=1.55

FS=tan(32°)/tan(22°)=0.6249/0.4040=1.546~1.55. A (1.10), C (1.82), D (2.10) are incorrect values or different conditions. FS=1.55 is acceptable for temporary cuts but borderline for permanent slopes -- typical design requires FS>=1.3 to 1.5 depending on consequence of failure. FS=1.55 represents a marginally stable slope for permanent conditions -- most codes require FS>=1.3 to 1.5 for permanent cut slopes. The infinite slope analysis is appropriate for shallow translational failures.

89. D — pix(dp+d) = pix41 = 129 in -- circular critical perimeter at d/2 from the pile face

bo=pix(dp+d)=pix(16+25)=pix41=129 in. A (75=using dp+d/2), B (164=4x(dp+d)), C (82=2x(dp+d)) are all incorrect perimeter formulas. The circular pile critical perimeter is based on an equivalent circle with diameter dp+d. The circular pile critical perimeter pidp+pid=pi(dp+d)=pix41=129 in is used for punching shear checks in pile caps -- distinct from the rectangular critical perimeter used for column punching in slabs.

90. C — Kp = 3.0 -- (1+sinphi)/(1−sinphi) = 1.5/0.5 = 3.0 for phi=30°

Kp=(1+sin30°)/(1−sin30°)=(1+0.5)/(1−0.5)=1.5/0.5=3.0. A (0.33=Ka), B (1.0=K0), D (4.0) are all incorrect. Kp=3.0 for phi=30° is the passive pressure coefficient -- the passive pressure acts 9x the active pressure (Ka=1/3, Kp=3, ratio=9:1) for phi=30°. The Kp=3.0 value for phi=30° is 9x the active coefficient Ka=1/3, illustrating why passive resistance is so valuable for resisting sliding of retaining structures when sufficient displacement is available.

91. A — The upward pressure at the mat base = gamma_waterxh_water -- with WT at the mat base, h_water=0, so upward pressure = 0; net uplift = 0 for this condition

When the water table is at the mat base level, the water pressure at the base is zero (datum) -- net uplift = 0. Uplift occurs only when the water table is ABOVE the mat base, creating a net upward pressure on the base. B (same as A), C (1-ft water head assumed), D (water at mat top).

92. B — rho = 0.0020 -- based on the structural reinforcement minimum

ACI 318-14 Section 15.2.1 requires As,min in the long direction of a strip footing = 0.0020xbwxh (or the structural calculation, whichever is greater) -- using 0.002 for temperature and shrinkage in the longitudinal direction. A (0.0018=for slabs under 20 in thick), C (0.0014=alternative for Grade 40), D (no requirement) are incorrect.

93. D — The moisture content producing maximum dry unit weight -- above OMC, added water displaces soil solids, reducing dry density

The OMC is defined as the MC producing the maximum dry unit weight at a given compaction energy. Below OMC, insufficient lubrication prevents full particle rearrangement; above OMC, water occupies space that could otherwise be solid particles. A (wet density), B (friction angle), C (permeability=minimum at slightly above OMC, not at OMC exactly) are all incorrect definitions.

94. A — Halfway between the wall face and the footing edge -- a compromise critical section for masonry on concrete

For masonry wall footings, ACI 318-14 Section 13.5.1.2 specifies the critical section at halfway between the wall face and footing edge. B (wall face=concrete wall assumption), C (quarter wall thickness inside=masonry option), D (edge of base plate=steel column option) are all for different wall types. For masonry wall footings, the critical flexure section is midway between the wall face and footing edge per ACI 318-14 -- this reflects the ability of masonry to spread the column load laterally.

95. C — alpha ranges from approximately 1.0 (very soft clay) to 0.5 (stiff clay), generally decreasing as the undrained shear strength Su increases -- for practical design, alpha=0.6-0.8 is common for typical soft to medium clays

alpha ranges from ~1.0 (very soft clay, Su<500 psf) to ~0.5 (stiff clay, Su>2,000 psf). For typical soft to medium clay, alpha=0.6-0.8 is common in practice. A (1.0 always), B (0.5 and 0.25), D (0=no adhesion) are all overly simplified or incorrect. The decreasing alpha for stiffer clays reflects the reduced compatibility between the rigid clay structure and the concrete shaft -- very stiff clays have lower interface friction relative to their strength.

96. B — At the point of zero shear between the columns -- the positive moment maximum occurs at V=0 by integration of the net upward pressure

The V=0 point between columns is where the shear changes sign, and by integration, the bending moment is maximum at that point. A (at column face=maximum negative moment), C (centroid), D (midpoint between columns=only if columns are equal and symmetrically placed) are incorrect. The maximum positive moment location (V=0) can be determined by finding where the integrated net load (upward soil pressure minus downward column loads) changes sign between the two columns.

97. B — Acting as stiff reinforcing elements within the soft soil matrix (composite composite reinforcement) while also providing radial drainage paths to accelerate consolidation of the surrounding clay

Stone columns provide both composite reinforcement (stiff stone columns carry higher stress fraction due to stress concentration) and drainage paths (radial drainage to the stone column accelerates clay consolidation). A (displacement only), C (drainage only), D (densification of surrounding clay=stone columns don't densify clay) are all incomplete. The composite reinforcement effect (stress concentration ratio n=sigma_stone/sigma_clay typically 3-5) allows a small fraction of stone columns to carry a disproportionate fraction of the applied load.

98. A — Angular distortion δ/L = 1/300-1/500 to prevent frame and facade damage, with absolute total settlement limit of 1-2 in for typical warehouse column footings

The angular distortion limit δ/L=1/300 to 1/500 is the standard criterion for frame buildings including warehouses, along with total settlement limits of 1-2 in. B (water table), C (pile regulations), D (judgment only) are all incorrect frameworks. The 1/300-1/500 angular distortion range for warehouses balances structural performance (frame not cracking), operational requirements (floor flatness for forklifts), and rack system tolerances.

99. C — As >= 0.0018xbxh or the structural demand -- the minimum for Grade 60 flexural reinforcement in footings

ACI 318-14 Section 15.2.1: As,min from temperature and shrinkage ratio (rho=0.0018 for Grade 60 bars) or the structural requirement, whichever is larger. A (0.0014), B (0.0018=the ratio for Grade 60, but C is more complete by stating structural vs minimum), D (0.0033=200/fy for beams) are all less complete or incorrect for footings.

100. D — 3dp -- the minimum center-to-center pile spacing for driven concrete and steel piles to allow proper concrete placement and consolidation around each pile

ACI 318-14 Section 13.3.1.3: minimum center-to-center pile spacing = 3dp for driven piles. A (2dp=too tight), B (2.5dp), C (3dp=same as D but stated without the rationale) -- D is the correct ACI minimum pile spacing for standard pile types. The 3dp minimum pile spacing also ensures that pile installation (driving or drilling) doesn't damage adjacent piles -- too-close spacing can cause pile heave, soil disturbance, and loss of capacity in adjacent installed piles.