PE Civil Structural Exam Prep

Practice Exam 6

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. Use reference materials equivalent to those provided in the NCEES exam software. Domain coverage mirrors the official NCEES PE Civil - Structural exam blueprint.

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

PRACTICE EXAM 6 -- QUESTIONS

DOMAIN 1 -- Loads (Q1-Q17)

1. Per ASCE 7-16 Table 4.3-1, passenger car garage floors must be designed for a concentrated wheel load of 3,000 lb positioned to produce the maximum effect in addition to the uniform live load. The minimum uniform live load for passenger car garages is:

A. 30 psf
B. 40 psf
C. 50 psf
D. 60 psf

2. In ASCE 7-16 ASD basic load combinations (Section 2.4.1), the combination for dead load plus occupancy live load only (no wind, seismic, snow, or lateral earth) is:

A. D + L
B. 1.2D + 1.6L
C. D + 0.75L
D. 1.4D + L

3. For a Risk Category I structure with SDS=1.0g and SD1=0.7g, ASCE 7-16 Table 11.6-1/11.6-2 assigns a Seismic Design Category (SDC) of:

A. SDC B
B. SDC C
C. SDC C (governed by SD1 table)
D. SDC D

4. Per ASCE 7-16 Table 4.3-1, occupied roofs used as greenhouses or for supported planting (without public access) have a minimum live load of:

A. 20 psf
B. 40 psf
C. 60 psf
D. 100 psf

5. Per ASCE 7-16 Table 4.3-1, public corridors, lobbies, and vestibules in office buildings require a minimum uniformly distributed live load of:

A. 40 psf
B. 100 psf
C. 60 psf
D. 80 psf

6. ASCE 7-16 Section 2.3.3 load combination 7 includes self-straining forces T (from temperature, settlement, creep). The load factor applied to T in the LRFD combination 1.2D + 1.0W + 1.0L + 1.2T is:

A. 0.5
B. 1.0
C. 1.2
D. 1.6

7. The ASCE 7-16 approximate fundamental period Ta for a 5-story reinforced concrete moment-resisting frame with hn=60 ft is Ta=Crxhn^x where Cr=0.016 and x=0.9. What is Ta?

A. 0.70 s
B. 0.96 s
C. 0.64 s
D. 1.13 s

8. A sloped gable roof with pg=25 psf (ground snow), Is=1.0, Ce=1.0, Ct=1.0 has a slope factor Cs=0.80 (moderate slope). What is the sloped-roof snow load Ps=Csxpf, where pf=0.7CeCtIspg?

A. 14.0 psf
B. 20.0 psf
C. 17.5 psf
D. 25.0 psf

9. Per ASCE 7-16 Table 12.2-1, the deflection amplification factor Cd for a Steel Special Moment Frame (SMF) is:

A. 3.0
B. 4.0
C. 5.5
D. 8.0

10. For the leeward wall of a building under ASCE 7-16 MWFRS analysis, the external pressure coefficient Cp for a building with a length-to-width ratio of L/B=2 is approximately:

A. +0.8
B. -0.5
C. -0.8
D. +0.5

11. Per ASCE 7-16 Section 4.9.3, the lateral force (perpendicular to the runway girder) from a bridge crane -- the 'side thrust' -- shall be taken as a minimum of what percentage of the sum of the weights of the lifted load and the crane trolley?

A. 4% of the total lifted load plus crane trolley weight, applied as a lateral force perpendicular to the runway
B. 10% of the total lifted load plus crane bridge weight, applied as a lateral force parallel to the runway
C. 20% of the maximum lifted load -- same as the side-thrust formula used for overhead industrial cranes
D. 10% of the maximum wheel loads on the rail -- the minimum lateral force perpendicular to the crane runway girder

12. A horizontal structural irregularity exists in a structure when the maximum story drift (including accidental torsion) at one end of the diaphragm is more than a specified multiple of the average drift. Per ASCE 7-16 Table 12.3-1, the Type 1a (torsional) irregularity threshold is:

A. The maximum drift exceeds 1.2 times the average drift
B. The maximum drift exceeds 1.4 times the average drift
C. The maximum drift exceeds 2.0 times the average drift
D. The diaphragm has any re-entrant corners that exceed 15% of the plan dimension

13. Per ASCE 7-16 Table 4.3-1, balconies (exterior) must be designed for a live load equal to:

A. 60 psf minimum -- same as the promenade roof category regardless of the adjacent interior floor LL
B. 80 psf -- as specified in the commentary to ASCE 7-16 for exterior elevated platforms
C. 1.5 times the live load for the area served, but not less than 60 psf per ASCE 7-16 Table 4.3-1
D. 100 psf -- the public corridor LL, which applies to any elevated exterior walkway adjacent to a corridor

14. A Seismic Design Category D structure with a nonparallel system (the vertical lateral force resisting elements are not parallel or symmetric about the major orthogonal axes) is classified as having a Type 5 horizontal irregularity. The consequence per ASCE 7-16 Section 12.7.3 is:

A. The structure must be analyzed for seismic forces acting in the direction of the nonparallel elements and for forces acting simultaneously in both orthogonal directions

B. The system must be redesigned to eliminate the irregularity -- nonparallel systems are not permitted in SDC D

C. A penalty factor of 1.25 is applied to the base shear V in the irregular direction

D. The redundancy factor rho must be set to 1.3 regardless of the layout

15. When designing exterior cladding (windows, curtain wall, roof panels) for wind loads, ASCE 7-16 requires the use of:

A. MWFRS pressures -- the same values used for the lateral frame design

B. The average of MWFRS and C&C pressures -- always the governing design basis

C. C&C pressure coefficients only -- MWFRS values are specifically for the primary lateral system

D. Components and Cladding (C&C) pressures, which are generally higher than MWFRS for small tributary areas

16. Per ASCE 7-16 Table 1.5-2, the seismic importance factor Ie for a Risk Category IV essential facility (hospitals, fire stations, emergency command centers) is:

A. Ie=1.0

B. Ie=1.5

C. Ie=1.25

D. Ie=2.0

17. Per ASCE 7-16 Table 12.12-1, the allowable story drift ratio Deltaa/hsx for a Risk Category II structure with a special reinforced concrete moment frame is:

A. 0.010 hsx

B. 0.020 hsx

C. 0.025 hsx

D. 0.015 hsx

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

18. A rectangular hollow section has outer dimensions 12 in x 20 in and inner dimensions 8 in x 14 in. The strong-axis moment of inertia is I = b_o h_o3/12 – b_i h_i3/12. What is I?

A. 6,171 in4

B. 4,800 in4

C. 8,000 in4

D. 1,829 in4

19. The plastic section modulus Z for a solid rectangular section (b=12 in, h=18 in) is Z=bh2/4. What is Z?

A. 486 in3

B. 648 in3

C. 1,296 in3

D. 972 in3

20. A T-section beam (flange: b=8 in, tf=1.5 in; web: hw=10 in, tw=1.0 in) has its centroid at y_c=8.14 in from the bottom, I=266 in4, and carries a shear V=50 kips. The shear flow at the flange-to-web junction (Q_flange=31.36 in3) is q=VQ/I. What is q?

A. 3.2 kip/in

B. 7.8 kip/in
C. 5.9 kip/in
D. 11.6 kip/in

21. A cracked reinforced concrete beam (b=12 in, d=17 in, As=3.0 in2, n=8, kd=6.49 in, Icr=3,744 in4) carries a service moment M=1,800 kip-in. The concrete compressive stress at the top fiber is fc=Mxkd/Icr. What is fc?

A. 2.0 ksi
B. 4.5 ksi
C. 3.12 ksi
D. 1.6 ksi

22. A simply supported steel beam (E=29,000 ksi, I=200 in4) spans L=20 ft (240 in) under a uniform load w=2 kip/ft (1/6 kip/in). Using y(x)=wx(L3–2Lx2+x3)/(24EI), what is the deflection at x=L/4=60 in?

A. 1.241 in
B. 0.885 in
C. 0.414 in
D. 1.657 in

23. In the AISC alignment chart (Jackson-Moreland nomograph) for column effective length factor K, a sidesway-uninhibited frame (columns free to sway laterally) produces K values:

A. Always equal to 1.0 -- K is independent of sidesway
B. Between 0.5 and 1.0 -- sidesway reduces the effective length
C. Between 1.0 and 2.0 in most practical cases -- fixed-fixed sway column has K=1.2
D. Greater than or equal to 1.0, approaching 2.0 for a cantilever column

24. In a pin-jointed plane truss, a zero-force member is identified when:

A. At a joint with two non-collinear unloaded members, both are zero-force; if one pair is collinear and unloaded, the non-collinear member carries zero

B. A member has zero force when its length is shorter than any connected member -- shorter lengths attract proportionally less load

C. The member has a high slenderness ratio and would buckle before the joints displace, so it is treated as a zero-force member

D. A zero-force member is identified when it connects exactly two joints that are already stable -- any such member is redundant and zero-force

25. When a beam-end far node in moment distribution analysis is pinned (free to rotate), the adjusted stiffness used in place of K=4EI/L is K'=3EI/L, and the carry-over factor becomes:

A. 0.5 -- same as for a fixed far end
B. 1.0 -- pin ends always carry over the full moment
C. Zero -- no moment is carried over to a pinned far end
D. 2/3 -- intermediate value for a semi-rigid pin connection

26. A beam is fixed at the left end A and simply supported (pinned) at the right end B. It carries a uniform load w=2 kip/ft over the full span L=16 ft. The fixed-end moment at A for this condition is FEM_A=wL2/8. What is FEM_A?

A. 64 kip.ft
B. 42.7 kip.ft
C. 32 kip.ft
D. 85.3 kip.ft

27. In the moment distribution method for a sway frame, after balancing the joints for the no-sway condition, the sway correction is applied by:

A. Doubling the carry-over moments from all previous distribution cycles

B. Applying assumed sway displacements, computing FEMs from sway, distributing them, then scaling the sway moments to satisfy horizontal equilibrium

C. Reducing all fixed-end moments by the sway stiffness ratio

D. Applying the lateral shear as an additional fixed-end force at the story level

28. A 2-panel Warren truss (span L=12 ft, height h=6 ft, A=2.0 in2 each member, E=29,000 ksi) carries P=20 kips at the midspan top joint. Using the virtual work method with n=member forces per unit load at midspan, the vertical deflection at midspan is:

A. 0.024 in

B. 0.095 in

C. 0.071 in

D. 0.048 in

29. Yield line theory for two-way reinforced concrete slabs is a plastic analysis method that:

A. Predicts the failure load by assuming rigid-plastic slab segments separated by yield lines where full plastic moment is reached, forming a kinematic mechanism

B. Finds the elastic deflection distribution at every point of the slab surface under the factored service load condition

C. Distributes the design moments according to the tributary area surrounding each column or support in the flat plate system

D. Computes the ultimate punching shear capacity at the critical perimeter around each slab-column connection at failure load

30. For a propped cantilever (fixed at A, pinned at B, span L=10 ft) under uniform load w=1.5 kip/ft, Castigliano's theorem applied at the redundant reaction at B gives R_B=3wL/8. What is R_B?

A. 7.50 kips

B. 5.625 kips

C. 3.75 kips

D. 11.25 kips

31. A fixed-fixed steel bar (A=4.0 in2, E=29,000 ksi, L=120 in) is subjected to a uniform temperature rise DeltaT=100°F. With alpha=6.5x10■6/°F, the thermal stress that develops is sigma=ExalphaxDeltaT. What is sigma?

A. 10.0 ksi

B. 13.0 ksi

C. 18.85 ksi

D. 25.0 ksi

32. A point in a structural element has normal stress sigmax=12 ksi and sigmay=4 ksi with shear stress tauxy=6 ksi. The von Mises equivalent stress sigma_VM=sqrt(sigmax2–sigmaxsigmay+sigmay2+3tauxy2) is:

A. 12.0 ksi

B. 16.0 ksi

C. 10.0 ksi

D. 14.8 ksi

33. In the direct stiffness (matrix displacement) method, the global stiffness matrix [K] is assembled by:

A. Inverting the flexibility matrix [F] computed from unit loads at each DOF

B. Applying boundary conditions to the compatibility matrix

C. Computing the element stiffness matrices in local coordinates, transforming to global, and adding contributions at shared DOF locations

D. Multiplying the mass matrix by the acceleration response spectrum ordinates

34. A two-span continuous beam (equal spans L=20 ft=240 in, EI=4,350,000 kip-in2) has a uniform load w=1.5 kip/ft on the left span only. Using published tables, the midspan deflection of the loaded span (propped cantilever analogy) is δ~wL4/(185EI). What is δ?

A. 0.516 in

B. 0.129 in

C. 0.515 in

D. 0.064 in

35. A single concentrated moving load P traverses a simply supported beam of span L. The maximum support reaction at A occurs when:

A. The load is at support A (x=0) -- the reaction at A equals P

B. The load is at midspan -- both reactions are equal at P/2

C. The load is at the quarter-point x=L/4

D. The load position that produces maximum reaction at A also produces maximum shear in the member

36. An eccentrically loaded solid circular column (d=8 in, A=50.3 in2, I=201 in4) carries P=50 kips at eccentricity e=1.5 in. The bending moment M=Pe=75 kip-in. What is the maximum bending stress fb=Mc/I at the extreme fiber (c=4 in)?

A. 0.497 ksi

B. 1.49 ksi

C. 0.248 ksi

D. 2.987 ksi

37. A thin-walled closed rectangular tube (6 in x 10 in centerline, wall thickness t=0.5 in uniform) carries pure torque T=60 kip-in. The Bredt-Batho shear flow is q=T/(2A■) where A■ is the enclosed area. What is q?

A. 0.25 kip/in

B. 0.375 kip/in

C. 0.50 kip/in

D. 0.75 kip/in

38. A stress element has sigmax=12 ksi, sigmay=–4 ksi, and tauxy=8 ksi. The angle thetap of the principal plane from the x-axis (measured counterclockwise) is tan(2thetap)=2tauxy/(sigmax–sigmay). What is thetap?

A. 45.0°

B. 30.0°

C. 15.0°

D. 22.5°

39. The fundamental relationship between bending moment M, flexural rigidity EI, and radius of curvature R of the elastic curve (for small deflections) is:

A. Both A and C -- they express the same relationship in different forms

B. M = EI x R -- the moment is proportional to the radius

C. M = EI x d2v/dx2 -- the moment equals EI times the second derivative of deflection

D. M = EI/R, so curvature 1/R = M/(EI)

40. When applying the moment-area method to a non-prismatic beam (varying EI along the length), the M/EI diagram (not just the M diagram) must be used because:

A. The curvature at each cross-section is M/EI, so the moment-area theorems apply to the M/EI diagram to correctly account for the varying stiffness

B. Non-prismatic beams always have higher moments near the supports, requiring amplification

C. The moment-area theorem is invalid for non-prismatic beams -- only the conjugate beam method applies

D. The moment in a non-prismatic beam must first be scaled by EI_max before applying the theorem

41. A cable with horizontal span L=100 ft sag f=8 ft carries a uniform load of w=0.5 kip/ft (per horizontal foot). The horizontal cable tension H=wL2/(8f) is:

A. 39 kips

B. 78.1 kips

C. 156 kips

D. 26 kips

42. A two-story, three-column plane frame with rigid beams (one bay) and fixed bases has the following degrees of freedom when analyzed by the stiffness method (sidesway permitted): how many independent kinematic DOFs govern the sway analysis?

A. 1 -- the single story shear determines all motions

B. 3 -- three column base rotations

C. 2 -- one horizontal sway per floor level

D. 6 -- three joints times two rotations and one translation each

DOMAIN 3 -- Temporary Structures (Q43-Q49)

43. During construction of a composite floor system (steel deck + concrete topping), horizontal shear between the deck and the wet concrete is resisted primarily by:

A. Friction between the steel deck ribs and the fresh concrete

B. The shear stud connectors welded through the deck to the steel beam flanges during the concrete pour

C. Diagonal tension bars placed in the deck ribs at 2-ft intervals

D. The dead weight of the concrete acting as a normal force on the deck surface

44. Per OSHA 29 CFR 1926.452(l), mobile scaffolds (caster-wheeled towers) shall be manually propelled only on surfaces within 3° of level and shall not be moved when workers are on them if the height-to-base ratio exceeds:

A. 2:1 height-to-base ratio -- scaffold shall not be moved with workers on it when this ratio is exceeded

B. 3:1 height-to-base ratio -- workers may stay on board during movement below this threshold with outriggers

C. 4:1 height-to-base ratio -- the theoretical tip-over threshold when all wheels are simultaneously loaded equally

D. No restriction on height-to-base ratio -- OSHA only prohibits movement on surfaces inclined more than 3 degrees

45. Per 30 CFR Part 816 (Surface Mining) and common state regulations, vibration from blasting near structures is typically monitored by measuring Peak Particle Velocity (PPV). The generally accepted residential structure safe limit for PPV is:

A. 0.1 in/sec -- the threshold of human perception

B. 10 in/sec -- only visible structural distress matters

C. 0.5 to 2.0 in/sec, depending on frequency -- most residential limits are <=2.0 in/sec

D. 5.0 in/sec -- the threshold for cosmetic damage

46. Per OSHA 29 CFR 1926.652 Appendix B, the maximum allowable slope for an unclassified soil (unable to be classified as Type A, B, or C) during excavation is:

A. 3/4H:1V slope -- the steepest slope permitted for Type A cohesive soils

B. 1H:1V slope -- for Type B, moderately cohesive soils

C. 1.5H:1V slope -- the most conservative slope, required for unclassified or Type C soils

D. Vertical cut is permitted -- unclassified soils are treated as rock and allowed to stand vertically

47. To prevent formation of cold joints in a large concrete placement, the time between successive concrete lifts placed against each other must be limited so that the first layer has not yet reached initial set. Per ACI 304R, the acceptable elapsed time between layers is typically:

A. No time limit -- properly mixed concrete at the same temperature always bonds

B. A maximum of 4 hours between the initial placement and the adjacent lift, regardless of conditions

C. 30 minutes -- the maximum time concrete remains workable at 90°F

D. Governed by the time of initial set (typically 1-2 hours without retarder), extended by retarding admixtures or cooler temperatures -- field testing should verify

48. The primary purpose of form release agents (form oils) applied to concrete formwork before placing concrete is to:

A. Provide a chemical barrier that prevents the concrete paste from bonding to the form, allowing clean stripping without surface damage

B. Seal the form surface against water absorption from the fresh concrete mix

C. Increase the compressive strength of the concrete by reducing water migration into the form

D. Comply with OSHA requirements for form reuse -- form oil is mandatory per 29 CFR

49. When precast concrete wall panels are erected and temporarily braced, the bracing must resist lateral forces during the period before permanent connections are completed. Per ACI 550.1R and industry practice, the temporary braces must be designed for:

A. 10% of the panel weight applied laterally

B. The seismic design base shear for the panel tributary mass only

C. Wind loads based on the local code, plus a minimum brace force from panel self-weight eccentricity, applied simultaneously

D. Gravity loads only -- temporary bracing is not required for wind during construction

DOMAIN 4 -- Materials (Q50-Q63)

50. The modulus of elasticity of structural steel (E=29,000 ksi) compared to aluminum alloy (E~10,000 ksi) indicates that:

A. Steel is approximately 3 times stiffer per unit cross-sectional area, so aluminum members deflect about 3 times more under the same load for the same cross-section

B. Aluminum has higher strength per unit weight -- aluminum members can always be designed lighter than steel

C. Steel and aluminum produce identical deflections if the cross-section is scaled to achieve the same moment of inertia

D. The higher E of steel is offset by its higher self-weight -- both materials perform equally in deflection-controlled designs

51. Portland cement Type III is used in applications requiring:

A. Low heat of hydration -- used in mass concrete placements where internal temperature rise must be limited

B. Sulfate resistance -- preferred for footings and slabs exposed to high-sulfate soils or groundwater

C. General-purpose strength -- standard construction where 28-day strength is acceptable and no special conditions apply

D. High early strength -- cement is ground finer so concrete reaches 3-day strength comparable to the 7-day strength of Type I cement

52. Per NDS 2018, the reference design values for sawn lumber apply to 'dry service conditions' with moisture content not exceeding 19%. For exterior protected applications (under roof overhangs, covered porches), the typical equilibrium moisture content of wood in most US climates is approximately:

A. 5-8% (always below dry service threshold)

B. 12-15% (within the dry service limit of 19%)

C. 20-25% (exceeds dry service threshold -- wet service factors apply)

D. 0% -- properly kiln-dried lumber stays bone dry in all covered applications

53. In the Unified Soil Classification System (USCS), well-graded sand SW must satisfy two criteria in addition to less than 5% passing the No. 200 sieve. The criteria are:

A. Cu >= 4 AND 1 <= Cc <= 3 (coefficient of uniformity and coefficient of curvature)

B. PI < 4 AND plots below the A-line on the Casagrande plasticity chart

C. Median grain size D50 > 0.5 mm AND less than 12% passing No. 200

D. Cu >= 6 AND 1 <= Cc <= 3 -- same Cc criterion as gravel but higher Cu requirement for sand

54. Reducing the water-cement ratio (w/cm) in concrete design affects durability primarily by:

A. Increasing the modulus of elasticity and reducing creep under sustained long-term load in the structure

B. Reducing capillary porosity in the hardened cement paste, making the concrete less permeable to water, chloride ions, and aggressive chemicals

C. Increasing cement paste content, which fills voids throughout the mix, increasing density of the hardened concrete matrix

D. Slowing the hydration reaction, which gives more time for proper curing before the concrete reaches design strength

55. ASTM A36 structural steel has a minimum yield strength of 36 ksi and a minimum tensile strength Fu of:

A. 45 ksi

B. 50 ksi

C. 58 ksi

D. 65 ksi

56. Per NDS 2018 Supplement Table 5A, the wet service factor CM for reference bending design value Fb of glulam used in exposed exterior conditions (wet service, MC > 16%) is:

A. CM = 0.80 for bending design value Fb of structural glulam in wet service conditions

B. CM = 0.85 for Fb of structural glulam in wet service, same as sawn lumber

C. CM = 0.90 for bending in glulam -- glulam is less sensitive to moisture than sawn lumber

D. CM = 1.00 -- glulam is inherently resistant to moisture effects by virtue of its laminated assembly

57. Per TMS 602-16 Section 7.3.2, the minimum compressive strength of grout used in masonry construction is:

A. 1,500 psi

B. 1,800 psi

C. 3,000 psi

D. 2,000 psi

58. A structural steel mill test report (MTR), also called a certified material test report (CMTR), documents which information for each heat of steel?

A. Ultrasonic test results for internal flaws -- the MTR only covers non-destructive inspection

B. Fabrication drawings and welding procedures for the structural components

C. Chemical composition (ladle analysis) and mechanical properties (yield strength, tensile strength, elongation, Charpy toughness) from the heat of steel

D. The structural engineer's certification that the steel meets AISC 360-16 design requirements

59. Per ACI 232.2R and ACI 318-14 for general structural concrete (non-exposure classes requiring high durability), the maximum permitted replacement of Portland cement by Class F fly ash is typically:

A. Up to 25% by mass of cementitious materials for standard structural concrete

B. Up to 50% for all exposure conditions without restriction
C. 15% maximum -- ACI prohibits higher replacement rates
D. No limit -- fly ash content is determined solely by the mix design engineer

60. Per NDS 2018 Appendix N, the load duration factor CD for a member subjected only to dead load (permanent loads acting continuously throughout the design life) is:

A. CD = 1.0
B. CD = 0.9
C. CD = 0.8
D. CD = 1.15

61. ASTM C494 Type G admixtures combine which two functions?

A. Air-entraining agent and accelerated set -- shortens time-of-set while entraining protective freeze-thaw air bubbles
B. Normal-range water reducer and set retardation -- modestly reduces w/cm while extending the workable period
C. High-range water reducer and accelerated set -- maximizes workability while also speeding early strength gain
D. High-range water reduction (superplasticizer) and retarding -- reducing w/cm significantly while extending the workable placement period

62. In Atterberg limit testing, which statement correctly describes the relationship between the plastic limit (PL) and liquid limit (LL)?

A. PL is the MC at which soil transitions from solid to plastic; LL is the MC at which it becomes liquid -- and LL always exceeds PL
B. PL always exceeds LL because lower moisture corresponds to higher plasticity in cohesive clay soils
C. LL and PL are always equal for pure clays -- the plasticity index PI equals zero for all clays
D. PL is always exactly half the LL for all cohesive soils, by definition of the Casagrande test procedure

63. ASTM A615 Grade 60 deformed reinforcing bars have a minimum specified tensile strength Fu of:

A. 60 ksi -- same as yield strength, a common error when Fy and Fu values are confused for reinforcing bars
B. 75 ksi -- minimum tensile strength for A615 Grade 40 bars, incorrectly applied to Grade 60
C. 90 ksi -- minimum tensile strength specified for A615 Grade 60 deformed reinforcing bars
D. 100 ksi -- applies to A615 Grade 75 bars, not Grade 60

DOMAIN 5 -- Component Design (Q64-Q100)

64. A reinforced concrete T-beam has a span of 24 ft, web width bw=12 in, slab thickness hf=4 in, and center-to-center spacing of beams s=8 ft. Per ACI 318-14 Section 6.3.2.1, the effective flange width beff for an interior beam is the smallest of: L/4, bw+16hf, and c/c spacing. What is beff?

A. 48 in
B. 72 in
C. 76 in
D. 96 in

65. Per ACI 318-14 Section 8.4.4.2.3, the fraction of unbalanced moment transferred by eccentricity of shear (gammav) at a square interior column slab-column connection is gammav=1−gammaf, where gammaf=1/(1+(2/3)sqrt(b1/b2)). For a square column (b1=b2), what is gammav?

A. 0.600
B. 0.500
C. 0.400
D. 0.333

66. Per ACI 318-14 Table 9.3.1.1, the minimum thickness h for a simply supported non-prestressed reinforced concrete beam (fy=60,000 psi) to avoid deflection calculation is L/16. For a 24-ft span (288 in), what is the minimum h?

A. 24 in
B. 18 in
C. 14.4 in
D. 12 in

67. Per ACI 318-14 Section 24.2.4, the long-term deflection multiplier lambdaDelta=ξ/(1+50rho') applies to the immediate dead load deflection to compute additional long-term deflection. For a 60-month sustained duration (ξ=2.0) and compression steel ratio rho'=As'/(bwd)=1.2/(12x20)=0.005, what is lambdaDelta?

A. 2.000
B. 1.333
C. 1.818
D. 1.600

68. A beam with bw=12 in, d=21 in carries Vu=60 kips with phiVc=40 kips. Stirrups are #3 U-bars (Av=0.22 in2) in Grade 60 steel. Using the ACI 318-14 transverse reinforcement equation s=Avfyd/Vs where Vs=(Vu/phi)–Vc, what is the required stirrup spacing from the design equation (before checking ACI maximum spacing limits)?

A. 10.4 in
B. 8.0 in
C. 12.0 in
D. 24.0 in

69. In a pre-tensioned prestressed concrete beam, the decompression moment M_decomp is the moment at which the net flexural stress at the precompressed tensile fiber reaches zero. For a simply supported beam, M_decomp equals:

A. The cracking moment Mcr -- decompression and cracking occur simultaneously
B. Half the cracking moment (Mcr/2) by ACI definition
C. Pe x e_p, where Pe is the effective prestress force and e_p is its eccentricity from the centroid
D. Pe x e_p + Pe/A x S_bottom, combining the prestress force resultant with the section modulus

70. For a #8 bar (db=1.0 in, fy=60,000 psi, f'c=4,000 psi) with cover and spacing such that (c+Ktr)/db=2.5 (maximum allowed), the basic development length ld per ACI 318-14 is ld=3fyψt ψe ψs/(40lambdasqrtf'cx(c+Ktr)/db)xdb. The Class B tension lap splice length is 1.3xld. What is the Class B splice length?

A. 28 in
B. 22 in
C. 37 in
D. 45 in

71. Per ACI 318-14 Section 22.7.4.1, a member subjected to factored torsional moment Tu less than the threshold value phiTth=philambdasqrtf'c(Acp2/pcp)/4 may be designed by ignoring torsion. The primary reason for this provision is:

A. Below this threshold, the torsion can be redistributed to adjacent members in a statically indeterminate system without distress, so designing for it adds unnecessary cost
B. Torsion below the threshold is always resisted by the shear steel already provided for beam shear
C. The concrete alone provides sufficient torsional capacity at this level -- no steel is needed
D. The formula was developed only for rectangular sections; other shapes may not use it

72. Per ACI 318-14 Section 6.6.4.5.1, when performing the moment magnifier method for non-sway columns (δns), ACI requires a minimum eccentricity be used to prevent unconservative results. The minimum first-order moment for a non-sway column is taken as:

A. Pu x 0.1h (10% of the column dimension times the axial load)

B. Pu x 0.6 in + 0.03h (a linearly varying minimum eccentricity)

C. The larger of Pu x (0.6 + 0.03h) in or Pu x 1.0 in

D. Zero -- ACI 318-14 has no minimum eccentricity requirement for non-sway frames

73. A reinforced masonry or concrete shear wall with hw/lw <= 2 (where hw is the wall height and lw is the horizontal wall length) is classified as a 'squat' shear wall. The primary design implication is:

A. Squat walls are less stiff than slender walls -- they attract less seismic force

B. Squat walls are dominated by shear rather than flexure, requiring special consideration of diagonal tension capacity and horizontal reinforcement

C. Squat walls cannot be used in SDC D and above -- the aspect ratio is too low for ductile behavior

D. Squat walls require only nominal reinforcement -- their small hw/lw ratio provides inherent overstrength

74. Per ACI 318-14 Section 11.3.1.1, the limiting wall slenderness to ensure adequate column behavior (rather than plate-buckling instability) requires that the ratio kLc/h (unsupported height to wall thickness) not exceed:

A. Wall slenderness limit is kLc/h not to exceed 18 per ACI 318-14 Section 11.7.2

B. Wall slenderness limit is kLc/h not to exceed 24 per ACI 318-14 Section 11.7.2

C. Wall slenderness limit is kLc/h not to exceed 32 per ACI 318-14 Section 11.7.2

D. Wall slenderness limit is kLc/h not to exceed 25 per ACI 318-14 Section 11.7.2 for the simplified method

75. Per AISC 360-16 Section F2.5, the limiting unbraced length Lp (below which a beam achieves its full plastic moment Mp) for a W16x45 with A992 steel (ry=1.57 in, Fy=50 ksi) is Lp=1.76rysqrt(E/Fy). What is Lp?

A. 5.55 ft

B. 7.20 ft

C. 4.00 ft

D. 9.80 ft

76. A W16x45 beam (A992, Fy=50 ksi, Zx=82.3 in3) has an unbraced length Lb<Lp (in the plastic range). What is the available flexural strength phibMn?

A. 240 kip.ft

B. 380 kip.ft

C. 308.6 kip-ft

D. 450 kip.ft

77. For an HSS-to-HSS T-connection (branch attaching to the face of a chord), AISC 360-16 Chapter K requires checking the chord wall capacity for:

A. Combined axial force and in-plane shear capacity at the chord web -- governs HSS shear and compression limit states

B. Weld throat shear between the branch and chord only -- the HSS wall is not an independent design limit state per Chapter K

C. Branch member plastic moment -- the chord wall ductility is always sufficient to transfer full branch plastic moment

D. Local yielding and crippling of the chord wall face -- the chord wall is punched by the branch force and must be checked for these limit states

78. For a composite beam designed per AISC 360-16 Chapter I, the maximum center-to-center spacing of headed shear studs along the beam is the lesser of 8 times the total slab thickness and:

A. 24 in

B. 36 in

C. 48 in

D. 8 in

79. A column carries Pu=400 kips on a concrete pedestal (f'c=4 ksi). Using AISC Design Guide 1, the minimum base plate area A1=Pu/(phicx0.85f'c) with phic=0.65. What is the required bearing area A1?

- **A.** 100 in2
- **B.** 150 in2
- **C.** 181 in2
- **D.** 225 in2

80. AISC 360-16 Section E2 recommends that the slenderness ratio KL/r for compression members (other than built-up members) generally should not exceed:

- **A.** KL/r <= 200 -- a practical guideline to limit susceptibility to accidental loads and vibration
- **B.** KL/r <= 120 -- the absolute code limit
- **C.** KL/r <= 300 for all steel columns
- **D.** No limit -- only the available compressive strength matters

81. AISC 360-16 Chapter J distinguishes between 'snug-tight' and 'fully pretensioned' bolted connections. Fully pretensioned connections are required when:

- **A.** All beam-to-column connections in any building frame with more than 3 stories above grade require full pretensioning
- **B.** When the live-to-dead load ratio at the joint exceeds 0.50 -- indicating highly variable loading conditions
- **C.** Gravity-only connections in low-seismic zones -- standard practice for efficiency and reduced installation cost
- **D.** Connections that are slip-critical, subject to significant tension and shear reversal, or in SDC C or higher requiring seismic detailing

82. For a compact W-shape beam in a braced frame (Lb < Lp), the available flexural strength phibMn equals phibMp regardless of:

- **A.** Shear must be checked for interaction with bending even below Lp -- combined shear and moment govern for stocky compact sections
- **B.** Beam span and tributary area affect the available moment at Lb < Lp because longer spans attract more load from the floor system
- **C.** The beam's unbraced length -- within Lb < Lp, full plastic moment is always available regardless of how much shorter Lb is than Lp
- **D.** Load pattern -- a concentrated midspan load produces a less favorable moment gradient than uniform load, reducing the Mn below Lp

83. For an A500 Grade B (Fy=46 ksi) round HSS used as a beam or brace, the compact D/t limit is 0.07E/Fy. For E=29,000 ksi, what is the compact D/t limit?

- **A.** 40.9
- **B.** 44.1
- **C.** 53.9
- **D.** 27.8

84. A 3/4-in diameter A325 bolt (Fu=120 ksi) passes through a 1/2-in-thick A36 plate (Fu=58 ksi). The bolt bears against the plate in single shear. Per AISC 360-16 Section J3.10, the bearing design strength for standard holes with deformation considered is phiRn=0.75x2.4xFuxdxt. What is phiRn for the plate (the controlling element)?

- **A.** 39.1 kips
- **B.** 65.0 kips
- **C.** 22.5 kips
- **D.** 52.5 kips

85. A 4x12 Douglas Fir-Larch Select Structural sawn beam (Fb=1,500 psi, dry service) is used under snow loads (CD=1.15) in a covered dry building (CM=1.0, Ct=1.0). The size factor CF=1.0 and beam stability factor CL=0.90. What is the adjusted bending design value F'b?

- **A.** 1,725 psi
- **B.** 1,380 psi
- **C.** 1,200 psi
- **D.** 1,552 psi

86. Per NDS 2018 Section 12.1.4.6, for a bolt loaded parallel to grain in a wood member, the minimum required end distance (distance from the center of the bolt to the end of the member toward which the bolt load acts) for full design value is:

- **A.** 4D (four bolt diameters)
- **B.** 7D (seven bolt diameters)
- **C.** 3.5D (three-and-a-half bolt diameters)
- **D.** 10D (ten bolt diameters)

87. Per NDS 2018 Table 12N, the reference lateral design value Z for a 16d common wire nail (D=0.162 in) driven through a 1.5-in Douglas Fir-Larch side member into a DFL main member, in single shear, is approximately:

- **A.** 90 lb
- **B.** 175 lb
- **C.** 141 lb
- **D.** 200 lb

88. Per TMS 402-16 Table 5.4.2, the maximum horizontal spacing of joint reinforcement or horizontal reinforcement in a reinforced masonry wall (for walls with vertical bars only) shall not exceed the smaller of one-third the wall height and:

- **A.** 48 in
- **B.** 24 in
- **C.** 32 in
- **D.** 16 in

89. Per TMS 402-16 Section 5.3.1.6.2, for a masonry column designed by ASD, the resultant compressive force must remain within the kern of the column cross-section. For a square column of side B, the kern eccentricity limit is:

- **A.** e <= B/4
- **B.** e <= B/3
- **C.** e <= B/6 from the centroid (kern condition)
- **D.** e <= B/8 for columns with rhog < 0.002

90. A masonry lintel (single-span, simply supported) has an 8-ft clear span and supports a uniform load from the masonry wall above of w=0.24 kip/ft. The maximum bending moment M=wL2/8 is:

- **A.** 1.92 kip.ft
- **B.** 3.84 kip.ft
- **C.** 0.96 kip.ft
- **D.** 5.76 kip.ft

91. Per TMS 602-16 Article 1.5, special inspection of masonry is required for all masonry in Seismic Design Category (SDC):

- **A.** A and B -- routine inspection applies; special inspection starts at B
- **B.** C, D, E, and F -- special inspection is required when seismic demands are elevated

C. D and above only -- routine inspection is permitted in SDC C

D. All SDC levels -- special inspection is mandatory for all masonry per TMS 602-16

92. Per ACI 318-14 Section 26.8.2, the minimum reinforcement ratio for footings in each direction is the same as for slabs on grade under temperature and shrinkage: rhomin=0.0018 for Grade 60 deformed bars. For a 5x5 ft footing, 18 in thick, the minimum As per foot of footing width is rhominx12x18. What is As/ft?

A. 0.25 in2/ft

B. 0.31 in2/ft

C. 0.18 in2/ft

D. 0.39 in2/ft

93. An expansive (swelling) soil is most reliably identified in the field by which test or measurement?

A. The free swell test (ASTM D4546 Method A) -- measuring volume change under no load when submerged in water

B. The Proctor compaction test -- high compaction energy produces swelling soils

C. Standard penetration test (SPT) N-values below 10 -- low blow counts indicate swelling potential

D. Grain size analysis -- soils with greater than 20% passing No. 200 sieve are always expansive

94. For a 12x12 in column at the edge of a flat plate slab (d=7 in), the critical punching shear perimeter bo at an edge column (two sides running along the edge, one perpendicular to the edge) is:

A. 26 in

B. 51 in

C. 52 in

D. 20 in

95. A contractor must dewater a deep excavation (30 ft below grade) in permeable sandy soils. Compared to wellpoint systems (limited to about 15 ft of drawdown per stage), deep well pumps can dewater to greater depths because:

A. Wellpoints are expressly prohibited by OSHA regulations for excavations deeper than 20 ft in all soil types and conditions

B. Deep well pumps have larger casing diameters allowing greater water volume per pump, which matters for high-inflow conditions

C. Wellpoints use vacuum-assisted suction lift (limited to about 15 ft by atmospheric pressure); deep wells use submersible pumps inside the well casing, so drawdown depth is unlimited

D. Deep wells inject treated water back into the aquifer while wellpoints simply pump groundwater away from the excavation zone

96. For a shallow footing on rock, the allowable bearing capacity may be estimated from the Rock Quality Designation (RQD) and uniaxial compressive strength of the rock. High RQD (>75%) generally indicates:

A. Highly fractured rock with many discontinuities per meter of core run -- low allowable bearing capacity governs the design

B. Moderate rock quality requiring conservative reduction factors applied to the uniaxial compressive strength of intact specimens

C. Rock that is specifically better suited for deep foundations than shallow footings due to its high lateral capacity

D. Good to excellent rock quality with generally high allowable bearing pressures -- design is often controlled by structural failure of the footing rather than by rock capacity

97. A building footing is designed for bearing on compacted structural fill. The primary geotechnical risk the engineer must investigate is:

A. Hydrostatic uplift from a perched water table in the fill, which always has higher permeability than the underlying natural soil

B. Bearing capacity failure because compacted structural fill always has lower shear strength than the undisturbed native soil below it

C. Uniform settlement from the fill consolidating under applied load -- typically manageable because fill consolidation is predictable

D. Differential settlement from non-uniform fill compaction, variable underlying soil, or inadequate quality control -- the most significant risk for footings on fill

98. For a driven pile installed in dense sand or gravel, an H-pile (HP section) is preferred over a displacement pile (closed-end pipe or concrete pile) primarily because:

A. H-piles have significantly higher axial load capacity than displacement piles of the same steel weight, making them more economical

B. H-piles can be field-spliced far more easily than displacement piles by using standard splice plates and groove welds

C. H-piles are low-displacement piles -- their open web lets soil flow between the flanges, minimizing lateral ground displacement and vibration to adjacent structures during driving

D. H-piles are consistently lighter per linear foot, allowing smaller driving equipment, lower mobilization cost, and reduced crew requirements

99. A gravity retaining wall has a 3-ft-deep passive zone at the toe (granular fill with phi=30°, gamma=115 pcf, Kp=3.0). The passive earth pressure resultant per foot of wall is Pp=0.5Kpgammah2. What is Pp?

A. 1.55 kip/ft

B. 3.11 kip/ft

C. 0.78 kip/ft

D. 4.66 kip/ft

100. Per ACI 318-14 Section 25.5.5, the minimum lap splice length for compression reinforcement in a tied column footing pedestal (fy=60,000 psi, f'c=4,000 psi) must satisfy the greater of 0.0005fydb or 12 in, but not less than a computed ld_comp. For a #8 bar (db=1.0 in), the minimum compression lap splice is:

A. 12 in

B. 30 in

C. 22 in

D. 18 in

PRACTICE EXAM 6 -- ANSWER KEY & EXPLANATIONS

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

1. B — 40 psf

ASCE 7-16 Table 4.3-1 assigns 40 psf uniform LL to passenger car garages. Options A (30) is too low; C (50) and D (60) exceed the requirement. The 3,000-lb concentrated wheel load is applied separately at the worst-case location per Table 4.3-1 footnote. Garage uniform LL reflects static vehicle weight distributed over a representative floor area, not the.

2. A — D + L

The ASCE 7-16 ASD basic combination for dead plus occupancy live load is simply D+L, with no load factors (Section 2.4.1, LC1). Options B (1.2D+1.6L) and D (1.4D+L) are LRFD combinations; C (D+0.75L) applies when wind or seismic loads are also present. ASD uses the nominal service-level loads without amplification -- the safety margin resides in the allowable.

3. D — SDC D

For SDS=1.0g and SD1=0.7g, ASCE 7-16 Tables 11.6-1 and 11.6-2 assign SDC D for Risk Category I when SDS>=0.50g AND SD1>=0.20g. Both tables must be checked independently, and the more severe SDC governs. Options A, B, C are less severe categories applicable to lower SDS or SD1 values.

4. C — 60 psf

ASCE 7-16 Table 4.3-1 assigns 60 psf to occupied roof areas used for greenhouse growing or supported planting (without intended public promenade access). Options A (20 psf) is for ordinary inaccessible roofs; B (40 psf) for light access; D (100 psf) for assembly use. Greenhouse and garden loads reflect the weight of saturated soil, containers, irrigation systems, and.

5. B — 100 psf

ASCE 7-16 Table 4.3-1 assigns 100 psf minimum LL to public corridors, lobbies, and vestibules in office buildings. Option A (40 psf) is for office floors; C (60 psf) is for assembly fixed seating; D (80 psf) is not a standard ASCE table value. The 100 psf reflects concentrated occupancy, emergency egress loading, and potential queuing in public.

6. D — 1.6

Self-straining forces T (from thermal expansion, settlement, fabrication errors) use load factor 1.2 in the LRFD combination 1.2D+1.0W+1.0L+1.2T per ASCE 7-16 Section 2.3.3. Options A (0.5), B (1.0), C (1.2 as the only factor) misstate the self-straining factor in context. The 1.2T factor reflects uncertainty in restraint conditions that generate thermal or settlement-induced forces in indeterminate structures.

7. C — 0.64 s

Ta=Crxhn^x=0.016x60^0.9=0.016x39.84=0.637 s~0.64 s per ASCE 7-16 Eq. 12.8-7. Options A (0.70) and B (0.96) use x=1.0; D (1.13) uses hn=80 ft. The approximate period formula uses 60^0.9, not 60 -- the exponent 0.9 for RC MRFs reflects the fact that taller frames do not scale linearly in period with height because shear deformations in shorter frames are proportionally.

8. A — 14.0 psf

pf=0.7xCexCtxIsxpg=0.7x1.0x1.0x1.0x25=17.5 psf. Ps=Csxpf=0.80x17.5=14.0 psf. Options B (20.0) omits the 0.7 factor and only applies Cs; C (17.5) is pf before slope reduction; D (25.0) is the ground snow. The slope factor Cs accounts for heat transmission through the roof and sliding -- Cs<1.0 reduces the balanced roof load for roofs warm enough to shed snow.

9. C — 5.5

ASCE 7-16 Table 12.2-1 assigns Cd=5.5 for Steel SMF. Options A (3.0) is Cd for ordinary MRF; B (4.0) is intermediate MRF; D (8.0) is the R-factor (not Cd). Cd amplifies design story drifts computed under the reduced (CsxW) seismic forces to obtain the true expected drift, used for checking drift limits and P-delta effects.

10. B — -0.5

ASCE 7-16 Figure 27.3-1 gives Cp=–0.5 for the leeward wall of a building with L/B=2. The negative sign indicates suction (pressure directed outward). Options A (+0.8) and D (+0.5) are windward wall pressures; C (–0.8) applies to a very short building (L/B<=1). Leeward wall suction reduces as building length increases relative to width -- at L/B>=4, Cp approaches.

11. D — 10% of the maximum wheel loads on the rail -- the minimum lateral force perpendicular to the crane runway girder

ASCE 7-16 Section 4.10.3 specifies that the minimum lateral force (side thrust) from bridge cranes shall be 10% of the maximum wheel loads on the rail at which the lateral force is applied -- not a percentage of the total lifted load. Option C (20% of trolley+lifted load) is the calculation for cross-aisle thrust in ASCE 7-16 Table.

12. A — The maximum drift exceeds 1.2 times the average drift

ASCE 7-16 Table 12.3-1 Type 1a torsional irregularity exists when the maximum story drift including accidental torsion exceeds 1.2 times the average story drift. Type 1b (extreme) occurs at 1.4x the average. Options B (1.4x) is extreme torsion; C (2.0x) and D (re-entrant corners) describe different irregularity types.

13. C — 1.5 times the live load for the area served, but not less than 60 psf per ASCE 7-16 Table 4.3-1

ASCE 7-16 Table 4.3-1 footnote requires balconies to be designed for the larger of 1.5x the adjacent interior floor LL or 60 psf minimum. Options A (60 psf only) and B (80 psf) omit the 1.5x rule; D (100 psf) is too high. The 1.5x multiplier accounts for the tendency of people to congregate at balcony edges and.

14. A — The structure must be analyzed for seismic forces acting in the direction of the nonparallel elements and for forces acting simultaneously in both orthogonal directions

ASCE 7-16 Section 12.7.3 for structures with non-parallel lateral systems requires orthogonal combination: design for seismic forces in the non-parallel direction and simultaneously for 30% of the orthogonal force (or full SRSS combination). Options B (prohibited), C (penalty factor), D (rho=1.3) are incorrect. The orthogonal combination ensures that diagonal lateral elements carry combined demands from both principal earthquake.

15. D — Components and Cladding (C&C) pressures, which are generally higher than MWFRS for small tributary areas

ASCE 7-16 requires Components and Cladding (C&C) pressure coefficients for the design of individual cladding elements (glazing, roof panels, wall cladding) because C&C coefficients capture the higher local peak pressures at edges, corners, and ridges that govern small tributary areas. Options A, B, C are incorrect -- C&C pressures are generally higher than MWFRS values for small areas.

16. B — Ie=1.5

ASCE 7-16 Table 1.5-2 assigns Ie=1.5 for Risk Category IV (post-disaster essential facilities). RC III uses Ie=1.25; RC I/II use Ie=1.0. Option A (1.0) is for ordinary occupancies; C (1.25) is for RC III; D (2.0) does not exist in ASCE 7-16. The higher Ie amplifies the design seismic forces to ensure essential facilities sustain significantly less damage.

17. B — 0.020 hsx

ASCE 7-16 Table 12.12-1 limits story drift to 0.020hsx for Risk Category II buildings with special concrete moment frames. RC SMFs are in the less-restrictive row because their ductility accommodates larger story drifts without collapse. Options A (0.010) applies to Risk Category IV essential facilities; C (0.025) applies to Risk Category I with special or intermediate steel MRFs;.

18. A — 6,171 in4

I=(12x203/12)–(8x143/12)=8,000–1,829=6,171 in4. Option D (1,829) is the inner rectangle's I alone; B (4,800) uses incorrect dimensions; C (8,000) is only the outer rectangle. Hollow sections have high moment of inertia relative to area because material is placed efficiently at maximum distance from the neutral axis. The wall thickness directly controls the I reduction for the hollow void.

19. D — 972 in3

Z=bh2/4=12x182/4=12x324/4=972 in3. Option A (486) uses bh2/8; B (648) uses bh2/6 (which is S, the elastic modulus); C (1,296) doubles the result. Plastic section modulus Z=bh2/4 for solid rectangles equals bhxh/2/2 -- half the cross-section area times the distance between the two half-area centroids. Z always exceeds S; their ratio S/Z=section shape factor (1.5 for rectangles).

20. C — 5.9 kip/in

q=VQ/I=50x31.36/266=5.90 kip/in at the flange-web interface. Options A (3.2) and D (11.6) use incorrect Q or I values; B (7.8) uses a different cross-section assumption. The shear flow q represents the longitudinal shear per unit length that must be transferred between the flange and web at their junction.

21. C — 3.12 ksi

fc=Mxkd/Icr=1800x6.49/3744=3.12 ksi. Option A (2.0) uses a wrong kd; B (4.5) overestimates; D (1.6) underestimates. The transformed section method converts steel area As to nAs and locates the neutral axis kd from the top by equating first moments: b(kd)2/2=nAs(d–kd). The cracked section I_cr is then used for all service-load stress checks, not the gross section Ig.

22. B — 0.885 in

y(L/4)=wx(L/4)x(L3–2Lx(L/4)2+(L/4)3)/(24EI)=(2/12)x60x(13,824,000–1,728,000+216,000)/139,200,000=0.884 in. Option A (1.241) is the midspan deflection (5wL4/384EI); C (0.414) and D (1.657) use incorrect x positions. The elastic curve equation applies continuously -- deflection at any position can be computed, but maximum deflection for non-symmetric loading occurs away from midspan. The deflection at L/4 is 0.884 in vs 1.241 in at midspan -- about 71%.

23. D — Greater than or equal to 1.0, approaching 2.0 for a cantilever column

For sidesway-uninhibited frames, K>=1.0 because lateral displacement at the top amplifies the column moments. A cantilever column (pinned base, free top) has K=2.0; a fixed-fixed sway column has K=1.2. Options A (K=1.0), B (K<1.0), C (K between 1.0 and 2.0 for fixed-fixed) partially characterize specific cases but miss the key principle.

24. A — At a joint with two non-collinear unloaded members, both are zero-force; if one pair is collinear and unloaded, the non-collinear member carries zero

After the swap, option A correctly identifies zero-force members: at a joint with only two non-collinear members under no external load, both must have zero force; if one pair is collinear, the non-collinear member carries zero force. Options B-D describe other structural situations. Zero-force members are important because they carry no load yet participate in stability -- removing.

25. C — Zero -- no moment is carried over to a pinned far end

When the far end is pinned, the modified stiffness is 3EI/L (not 4EI/L for fixed), and no moment is carried over -- the carry-over factor is zero. Options A (0.5) and B (1.0) misstate the carry-over for pinned ends. Eliminating carry-over to pinned ends speeds moment distribution convergence: the pin end joint is balanced in a single distribution.

26. A — 64 kip.ft

FEM_A=wL2/8=2x162/8=64 kip-ft for a fixed-near/pinned-far beam. Options B (42.7) applies the standard fixed-fixed formula wL2/12; C (32) halves the result; D (85.3) uses wL2/3. The larger FEM at the fixed end (compared to wL2/12 for fixed-fixed) reflects that pinning the far end reduces restraint there, forcing more curvature near the fixed end.

27. B — Applying assumed sway displacements, computing FEMs from sway, distributing them, then scaling the sway moments to satisfy horizontal equilibrium

In sway moment distribution: (1) distribute joints in a no-sway condition; (2) apply an assumed sway (unit chord rotation) to all columns, compute chord-rotation FEMs, distribute them; (3) scale the sway-cycle results so that the total horizontal shear equals the applied lateral force or satisfies equilibrium. Options A, C, D misstate the procedure.

28. D — 0.048 in

δ=Σ(nNL)/(AE)=[2x(N_TC/P)xN_TCxL_TC+(N_BC/P)xN_BCxL]/(AE). With N_TC=–14.14k, L_TC=101.82 in, N_BC=10k, L=144 in, A=2.0 in2, E=29,000 ksi: δ=[2x0.707x14.14x101.82+0.5x10x144]/58,000=[2,035+720]/58,000=0.0475 in. Options A (0.024), B (0.095), C (0.071) are incorrect. The virtual work method requires computing member forces for both the actual loading (N) and a unit virtual load (n), then summing nNL/AE. Virtual work requires computing member forces for both the actual loading N.

29. A — Predicts the failure load by assuming rigid-plastic slab segments separated by yield lines where full plastic moment is reached, forming a kinematic mechanism

Yield line theory is an upper-bound plastic method: the analyst postulates a kinematic failure mechanism (a pattern of rigid slab segments rotating about yield lines where the full plastic moment is reached) and applies virtual work to find the collapse load. Options B-D describe elastic analysis, tributary area distribution, and punching shear -- none of which are yield.

30. B — 5.625 kips

R_B=3wL/8=3(1.5)(10)/8=5.625 kips. Castigliano applied at the redundant B: the released structure is a cantilever from A; the deflection at B under w plus redundant R_B must be zero. Integrating ∂U/∂R_B=0 gives R_B=3wL/8. Options A (7.50) uses 5wL/8; C (3.75) uses wL/4; D (11.25) uses 3wL/4. This result matches the force method and confirms that Castigliano and the compatibility.

31. C — 18.85 ksi

sigma=EalphaDeltaT=29,000x6.5x10■6x100=18.85 ksi. For a fully restrained bar, no thermal deformation occurs -- the bar develops compressive stress equal to the thermal stress. Options A (10.0) uses alpha=3.4x10■6; B (13.0) uses alpha=4.5x10■6; D (25.0) uses alpha=8.6x10■6. The thermal stress is always compressive for heating with fixed ends. Partial end restraint reduces the stress proportionally to the restraint stiffness ratio.

32. D — 14.8 ksi

sigma_VM=sqrt(122–12x4+42+3x62)=sqrt(144–48+16+108)=sqrt220=14.83 ksi~14.8 ksi. Options A (12.0), B (16.0), C (10.0) apply incorrect formula components. Von Mises criterion predicts yielding when sigma_VM>=Fy -- it accounts for the combined effect of all stress components more accurately than the maximum normal or maximum shear criteria. For uniaxial tension sigma_VM=sigmax, recovering the simple tensile yield criterion.

33. D — Multiplying the mass matrix by the acceleration response spectrum ordinates

The global stiffness matrix [K] is assembled by transforming each element's local stiffness matrix to global coordinates and adding its contributions at the appropriate DOF rows and columns (direct stiffness assembly). Options A (invert flexibility), B (compatibility matrix), C (partially correct -- C correctly describes the procedure). Option D was placed at D after the swap.

34. C — 0.515 in

δ=wL4/(185EI)=(1.5/12)x2404/(185x4,350,000)=0.1250x3,317,760,000/805,000,000=0.515 in. Options A (0.516~A in original) rounds slightly differently; B (0.129) quarters the result; D (0.064) is 1/8 of the result. The factor 185 in the denominator (vs 384 for SS or 192 for fixed-fixed) reflects that the propped cantilever is stiffer than SS but less stiff than fully fixed -- verified by the result falling between.

35. A — The load is at support A (x=0) -- the reaction at A equals P

The maximum reaction at support A of a simply supported beam occurs when the moving load P is positioned directly at A (x=0), giving R_A=P and R_B=0. Options B (midspan), C (quarter-point), D (same as A for max shear) are incorrect. The influence line for reaction at A has an ordinate of 1.0 at A and 0 at.

36. B — 1.49 ksi

sigma_bending=Mc/I=(50x1.5)x4/201.06=300/201.06=1.49 ksi. Option A (0.497) uses c=d/2/3 incorrectly; C (0.248) halves the answer; D (2.987) doubles it. For eccentric axial loading, the total maximum stress is sigma_total=P/A+fb=0.995+1.49=2.49 ksi; the pure bending stress Mc/I accounts for the eccentricity contribution only. Circular columns have I=pid4/64 and c=d/2. The bending stress 1.49 ksi acts alone at the extreme fiber.

37. C — 0.50 kip/in

q=T/(2A■)=60/(2x6x10)=60/120=0.50 kip/in. Options A (0.25) and B (0.375) use larger or incorrect enclosed areas; D (0.75) uses 0.5x the correct A■. The Bredt-Batho shear flow is constant around the closed tube perimeter (unlike open sections where shear flow varies). The actual wall shear stress tau=q/t varies with wall thickness: thinner walls carry higher shear stress for the same.

38. D — 22.5°

tan(2thetap)=2tau/(sigmax−sigmay)=2x8/(12−(−4))=16/16=1.0 → 2thetap=45° → thetap=22.5°. Options A (45°), B (30°), C (15°) use incorrect applications of the principal plane formula. The principal angle 22.5° is measured counterclockwise from the x-axis to the plane on which maximum normal stress acts. At this angle, the shear stress is zero -- confirming it is a principal plane.

39. A — Both A and C -- they express the same relationship in different forms

The beam curvature relationship is M=EI/R (or equivalently 1/R=M/EI), where R is the radius of curvature. This is the same as M=EIxd2v/dx2 for small deflections. Both expressions are equivalent forms -- the differential form (C) is derived from the geometric definition of curvature (A). Option D (both A and C) was at D before; A now correctly states.

40. B — Non-prismatic beams always have higher moments near the supports, requiring amplification

The moment-area theorems require the M/(EI) diagram (curvature diagram), not the M diagram alone, because curvature κ=M/(EI) varies with both moment and stiffness. For non-prismatic beams, EI varies along the span, so M/(EI) must be constructed for each segment with its own EI value before computing areas and centroid moments.

41. B — 78.1 kips

H=wL2/(8f)=0.5x10,000/64=78.125 kips~78 kips for the parabolic cable approximation (load uniform per horizontal distance). Options A (39) halves the result; C (156) doubles; D (26) uses f=16 ft. The horizontal tension H is constant throughout the cable length (since no horizontal external forces act). Vertical reactions at the supports equal wL/2=25 kips each; the resultant cable tension at the.

42. C — 2 -- one horizontal sway per floor level

For a two-story, one-bay frame with fixed bases and sway permitted, there are 2 independent horizontal DOFs: one per floor level (story translations δ1 and δ2). Joint rotations at beam-column connections are also DOFs in a full analysis (6 joints x 1 rotation = 6), but for the sway analysis alone, 2 lateral DOF govern the story drift.

43. D — The dead weight of the concrete acting as a normal force on the deck surface

During construction of a composite floor, the shear studs welded to the steel beam transfer horizontal shear between the hardened concrete slab and the steel section at composite action. The question asked about transfer during the pour -- before composite action is achieved -- and the correct answer is D (the shear studs), not friction or gravity force.

44. A — 2:1 height-to-base ratio -- scaffold shall not be moved with workers on it when this ratio is exceeded

OSHA 29 CFR 1926.452(l)(5) prohibits manually propelling mobile scaffolds when workers are on them if the scaffold height-to-base ratio exceeds 2:1. Options B (3:1), C (4:1), D (no restriction) are incorrect. The 2:1 height-to-base limit prevents tip-over from caster wheel inertia when the scaffold is moved. If the ratio exceeds 2:1, workers must descend before the scaffold is.

45. C — 0.5 to 2.0 in/sec, depending on frequency -- most residential limits are <=2.0 in/sec

Standard residential damage thresholds are typically 0.5 to 2.0 in/sec PPV (peak particle velocity), depending on frequency -- the OSMRE/USBM RI-8507 standards use frequency-dependent safe limits with the structure-damage threshold generally <=2.0 in/sec. Options A (0.1 in/sec) is human perception; B (10 in/sec) is well above safe levels; D (5.0 in/sec) exceeds typical cosmetic damage thresholds for residential.

46. B — 1H:1V slope -- for Type B, moderately cohesive soils

OSHA 29 CFR 1926.652 Appendix B requires a maximum slope of 1.5H:1V for unclassified soil (which cannot be identified as Type A, B, or C). Options A (3/4:1) applies to Type A; C (1:1) is Type B; D (vertical) describes rock. For unclassified soil, the most conservative Type C slope of 1.5H:1V must be used because the soil's.

47. D — Governed by the time of initial set (typically 1-2 hours without retarder), extended by retarding admixtures or cooler temperatures -- field testing should verify

Cold joint prevention depends on placing fresh concrete before the earlier lift has reached initial set -- typically 1-2 hours for normal concrete at 70°F. Retarding admixtures and cooler temperatures extend this window; hot weather accelerates set. Options A (no time limit), B (fixed 4 hours), C (30 minutes at 90°F) either ignore field variability or mistate ACI.

48. A — Provide a chemical barrier that prevents the concrete paste from bonding to the form, allowing clean stripping without surface damage

Form release agents (also called mold oils or form oils) create a chemical boundary layer that prevents the cement paste from bonding to the form surface, allowing clean stripping without damage to form or concrete surface. Options B, C, D misidentify the purpose -- release agents are not sealers, strength modifiers, or regulatory requirements.

49. C — Wind loads based on the local code, plus a minimum brace force from panel self-weight eccentricity, applied simultaneously

Temporary braces for precast wall panels must resist wind and seismic forces based on local codes, combined with a minimum brace force from panel self-weight eccentricity (typically 1/100 of panel weight per ACI 550.1R), applied simultaneously. Options A (10% of weight) is the OSHA scaffold standard; B (seismic only) ignores wind; D (no bracing required) is dangerous.

50. A — Steel is approximately 3 times stiffer per unit cross-sectional area, so aluminum members deflect about 3 times more under the same load for the same cross-section

Steel (E=29,000 ksi) is approximately 2.9x stiffer than aluminum (E~10,000 ksi) per unit cross-sectional area, so an identical aluminum section deflects about 2.9x more than the same steel section under the same load. Options B (weight advantage) is irrelevant to the stiffness question; C (equal deflections for same I) ignores that equal I at different E gives different.

51. D — High early strength -- cement is ground finer so concrete reaches 3-day strength comparable to the 7-day strength of Type I cement

Type III Portland cement is ground more finely than Type I, achieving high early strength -- 3-day strength comparable to 7-day for Type I and 28-day strength at 7 days. Options A (low heat) describes Type IV; B (sulfate resistance) describes Type V; C (general purpose) describes Type I.

52. B — 12-15% (within the dry service limit of 19%)

In most US climates, exterior protected applications maintain equilibrium MC of 12-15%, which is below the 19% dry service threshold. Options A (5-8%) is only typical of interior heated and air-conditioned spaces; C (20-25%) would require wet service factors; D (0%) is physically impossible for wood exposed to air.

53. D — Cu >= 6 AND 1 <= Cc <= 3 -- same Cc criterion as gravel but higher Cu requirement for sand

For the USCS symbol SW (well-graded sand), the criteria are Cu>=6 AND 1<=Cc<=3 (higher Cu requirement than for gravels, which use Cu>=4). Option A correctly describes GW, not SW -- the Cu threshold differs. Options B (plasticity criteria) applies to fine-grained soils; C (D50 criterion) is not a USCS criterion.

54. B — Reducing capillary porosity in the hardened cement paste, making the concrete less permeable to water, chloride ions, and aggressive chemicals

Lower w/cm reduces the capillary porosity of the hardened cement paste, making it less permeable to water, chloride ions, CO2 (carbonation), and sulfate solutions that cause rebar corrosion, freeze-thaw damage, and alkali-silica reaction. Options A (E and creep), C (paste content increase), D (slower hydration) are secondary or incorrect effects.

55. C — 58 ksi

ASTM A36 has a minimum tensile strength Fu=58 ksi (and minimum yield Fy=36 ksi). Options A (45), B (50), D (65) are incorrect. Fu=58 ksi is the property used for calculating net-section rupture capacity and weld matching requirements. For connection design, the 58-ksi tensile strength -- not the 36-ksi yield strength -- governs fracture and block shear limit.

56. A — CM = 0.80 for bending design value Fb of structural glulam in wet service conditions

NDS 2018 Supplement Table 5A gives CM=0.80 for Fb of structural glulam in wet service conditions (MC>16%). Options B (0.85) is for horizontal shear Fv; C (0.90) is for some other properties; D (1.00) applies only to dry service. Unlike sawn lumber (CM=0.85 for Fb), glulam uses CM=0.80, reflecting that laminated assemblies are more sensitive to moisture cycling.

57. D — 2,000 psi

TMS 602-16 Section 7.3.2.1 requires masonry grout to have a minimum compressive strength of 2,000 psi at 28 days. Options A (1,500) and B (1,800) are below the minimum; C (3,000) is conservative but not the required minimum. High-strength grout (3,000 psi) is often specified to match masonry unit strength and improve durability, but 2,000 psi is the.

58. C — Chemical composition (ladle analysis) and mechanical properties (yield strength, tensile strength, elongation, Charpy toughness) from the heat of steel

A mill test report (MTR/CMTR) from the steel producer documents the heat-specific chemical composition (carbon, manganese, silicon, etc.) and mechanical test results (Fy, Fu, elongation, CVN if required) for that heat of steel. Options A (UT results), B (fabrication drawings), D (engineer certification) are not part of the MTR.

59. A — Up to 25% by mass of cementitious materials for standard structural concrete

ACI 232.2R and typical state-of-practice limit Class F fly ash replacement to 25% by mass of cementitious content for standard structural concrete without specialized durability mix design. Options B (50% without restriction) applies only to specific high-volume fly ash designs with testing; C (15% max) is too conservative; D (no limit) contradicts ACI guidelines.

60. B — CD = 0.9

NDS 2018 Appendix N assigns CD=0.9 for permanent dead loads acting at the design life duration (50+ years). Options A (1.0) is for 10-year loads (floor LL); C (0.8) is not a standard NDS value; D (1.15) is snow load. Dead load has the lowest CD because wood strength decreases significantly under very long-term sustained loading due to.

61. D — High-range water reduction (superplasticizer) and retarding -- reducing w/cm significantly while extending the workable placement period

ASTM C494 Type G admixtures combine high-range water reduction (superplasticizer function) with set retardation -- useful for large, hot-weather placements that need both reduced w/cm and extended workability. Options A (air+accelerator), B (normal WR+retarder=Type D), C (accelerator+air) describe different combinations. Type G provides the maximum water reduction of all ASTM C494 types while preventing premature stiffening in long-haul.

62. A — PL is the MC at which soil transitions from solid to plastic; LL is the MC at which it becomes liquid -- and LL always exceeds PL

The plastic limit (PL) is the MC at which the soil transitions from solid/semi-solid to plastic behavior (when rolled into threads begins to crumble); the liquid limit (LL) is the MC at which the soil transitions from plastic to liquid state. LL always exceeds PL, and PI=LL−PL>=0. Options B (PL>LL) is physically impossible; C (PI=0 for pure clays).

63. C — 90 ksi -- minimum tensile strength specified for A615 Grade 60 deformed reinforcing bars

ASTM A615 Grade 60 deformed reinforcing bars have a minimum specified tensile strength Fu=90 ksi (Fy_min=60 ksi). Options A (60 ksi) confuses Fu with Fy; B (75 ksi) is the A615 Gr 40/60 tensile minimum for smaller diameters incorrectly; D (100 ksi) overstates it. The ductility requirement Fy/Fu<0.75 (minimum elongation) must also be satisfied.

64. B — 72 in

beff=min(L/4, bw+16hf, c-to-c)=min(72, 12+64=76, 96)=72 in. Options A (48) uses L/6; C (76) gives bw+16hf without the L/4 check; D (96) is only the beam spacing. ACI 318-14 Section 6.3.2.1 requires checking all three criteria and using the smallest. The L/4 limit prevents the effective slab from extending so far that non-uniform shear lag effects compromise the composite.

65. C — 0.400

gammav=1−gammaf=1−1/(1+(2/3)sqrt(b1/b2))=1−1/(1+2/3)=1−0.60=0.40 for a square column (b1=b2). Options A (0.600) is gammaf; B (0.500) applies to an aspect ratio of b1/b2=2.25; D (0.333) applies when b1/b2 is very large. gammavxMsc determines the unbalanced moment transferred by shear eccentricity, contributing to the punching shear stress on the critical perimeter. The remaining gammaf fraction is transferred by flexure through the column.

66. B — 18 in

h_min=L/16=288/16=18 in per ACI 318-14 Table 9.3.1.1 for a simply supported non-prestressed beam with Grade 60 steel. Options A (24 in) applies to a cantilever (L/8); C (14.4 in) uses L/20 (two-way slab); D (12 in) is L/24. The L/16 limit is calibrated to keep long-term deflection under L/360 for typical service conditions.

67. D — 1.600

lambdaDelta=ξ/(1+50rho')=2.0/(1+50x0.005)=2.0/1.25=1.600. Options A (2.000) sets rho'=0; B (1.333) uses ξ=2.0/(1+50x0.010)=1.333 (rho'=0.010); C (1.818) uses rho'=0.001. Compression reinforcement rho' reduces creep by sharing the compression with the concrete. Without compression steel (rho'=0), lambdaDelta=2.0, meaning 100% additional long-term deflection on top of the immediate dead load deflection for 5 years or more.

68. A — 10.4 in

Vc=phiVc/phi=40/0.75=53.3 k. Vs=(Vu/phi)−Vc=60/0.75−53.3=26.7 k. s=Avfyd/Vs=0.22x60x21/26.7=277.2/26.7=10.4 in. Options B (8.0) and C (12.0) use different Av or Vs; D (24.0) uses a larger Av. ACI limits maximum stirrup spacing to d/2=10.5 in or 24 in -- here, 10.4 in from the design equation approximately matches d/2, indicating the spacing is governed by both the design equation and the ACI.

69. D — Pe x e_p + Pe/A x S_bottom, combining the prestress force resultant with the section modulus

M_decomp = Pexe_p + (Pe/A)x(S_bottom), combining the direct prestress P/A that shifts the entire stress upward with the eccentricity moment Pexe_p that shifts it further -- together they offset the tensile flexural stress. Option C (Pexe_p alone) omits the axial precompression term. Options A and B misidentify decompression.

70. C — 37 in

ld=3(60,000)/[40(1.0)sqrt4000/2.5]x1.0=3x60,000/(40x63.25/2.5)=180,000/(1012)=177.8/... let me state: ld=28.5 in. Class B splice=1.3x28.5=37.0 in. Options A (28) is ld itself; B (22) is too short; D (45) uses 1.58xld. Class B splices (1.3ld) apply when more than half the bars are spliced at a section or when As_provided<2xAs_required. The extra 30% accounts for reduced splice reliability when joints are not staggered.

71. A — Below this threshold, the torsion can be redistributed to adjacent members in a statically indeterminate system without distress, so designing for it adds unnecessary cost

Torsion below the threshold phiTth can be neglected in statically indeterminate structures because the torsion can redistribute to adjacent members -- the structure is torsionally redundant, and the member redistributes torsion until it reaches adjacent members. Options B, C, D misidentify the reason. In statically determinate members, torsion cannot redistribute and must always be fully designed for regardless.

72. B — Pu x 0.6 in + 0.03h (a linearly varying minimum eccentricity)

ACI 318-14 Section 6.6.4.5.3 requires a minimum first-order moment for non-sway column design: Mmin=Pux(0.6+0.03h) in., ensuring meaningful eccentricity even when analysis gives a nearly concentric load. Option A (0.1h) applies to ACI 318-99 and older codes; C uses 1.0 in absolute; D (no minimum) is incorrect. The minimum eccentricity prevents the column from being designed as a pure.

73. B — Squat walls are dominated by shear rather than flexure, requiring special consideration of diagonal tension capacity and horizontal reinforcement

Squat shear walls (hw/lw<=2) are governed by shear failure rather than flexure-dominated behavior seen in slender walls. Diagonal tension capacity and horizontal reinforcement distribution are the primary design concerns. Options A (less stiff), C (not permitted in SDC D), D (nominal reinforcement) are all incorrect. Squat walls can be used in SDC D with special reinforced masonry detailing.

74. D — Wall slenderness limit is kLc/h not to exceed 25 per ACI 318-14 Section 11.7.2 for the simplified method

ACI 318-14 Section 11.7.2 (structural walls) limits the ratio kLc/h <= 25 for walls designed using the simplified design method. Options A (18) is the TMS 402 masonry h/t limit; B (24) and C (32) are not the ACI criterion. Beyond kLc/h=25, the slender wall provisions of ACI 318-14 Section 11.8 must be used, which include magnified moments.

75. A — 5.55 ft

Lp=1.76rysqrt(E/Fy)=1.76x1.57xsqrt580=1.76x1.57x24.08=66.6 in=5.55 ft. Options B (7.20), C (4.00), D (9.80) use incorrect ry or coefficients. Below Lp, the beam can develop its full plastic moment without lateral buckling. The Lp formula is derived by setting the LTB capacity equal to Mp; it depends on ry (not rx) because lateral buckling involves out-of-plane (weak-axis) bending and twisting.

76. C — 308.6 kip-ft

phibMn=phibMp=phibxFyxZx=0.9x50x82.3=3,703.5 kip-in=308.6 kip-ft~309 kip-ft. Options A (240), B (380), D (450) use incorrect Zx or phi. For compact sections with Lb<Lp, no LTB reduction applies and the full plastic moment Mp is available. The W16x45 has Zx=82.3 in3 per AISC Steel Construction Manual; phi=0.9 for flexure; Fy=50 ksi for A992.

77. D — Local yielding and crippling of the chord wall face -- the chord wall is punched by the branch force and must be checked for these limit states

For an HSS-to-HSS T-connection (branch perpendicular to chord), AISC Chapter K requires checking local yielding and local crippling (punching) of the chord wall face under the branch force. Options A (chord web shear), B (weld only), C (branch moment) are not the controlling Chapter K limit states for hollow section connections.

78. B — 36 in

AISC 360-16 Section I8.2d limits composite stud maximum center-to-center spacing to the lesser of 8 times the total slab thickness or 36 in. Options A (24), C (48), D (8) do not match the AISC limit. The 36-in maximum prevents large unshored spans between studs that would allow excessive interface slip and reduce the composite action efficiency.

79. C — 181 in2

A1_req=Pu/(phicx0.85f'c)=400/(0.65x0.85x4)=400/2.21=181 in2. Options A (100) uses phi=1.3; B (150) uses phi=0.70; D (225) uses phi=0.60. The base plate bearing area transfers the column load to the concrete at an allowable bearing stress of phicx0.85f'c. The plate dimensions (NxB) must provide at least A1_req and are typically chosen to minimize plate thickness while maintaining compact proportions around the column.

80. A — KL/r <= 200 -- a practical guideline to limit susceptibility to accidental loads and vibration

AISC 360-16 Commentary Section E2 recommends that KL/r<=200 for compression members to avoid excessive susceptibility to accidental loads, vibration, and construction-phase loads. It is a guideline, not a mandatory code limit. Options B (120), C (300), D (no limit) are all incorrect. Very slender compression members with KL/r>200 have critical loads Fcr that are extremely low, making them.

81. D — Connections that are slip-critical, subject to significant tension and shear reversal, or in SDC C or higher requiring seismic detailing

Fully pretensioned A325/A490 bolts are required per AISC 360-16 Table J3.1 for: slip-critical connections, connections subject to tension and shear reversal, connections in SDC C or higher with seismic requirements, and connections per AISC 341 seismic provisions. Options A (all multi-story connections), B (LL>50% DL), C (gravity-only loads) are incorrect criteria.

82. C — The beam's unbraced length -- within Lb < Lp, full plastic moment is always available regardless of how much shorter Lb is than Lp

For compact beams with Lb<Lp, phibMn=phibMp regardless of how much shorter Lb is than Lp -- the full plastic moment is always achievable. Options A (shear interaction), B (span dependence), D (load pattern) are all irrelevant to phibMn in the plastic range. Lp is the threshold below which no additional LTB reduction applies -- the beam simply achieves.

83. B — 44.1

Compact D/t=0.07E/Fy=0.07x29,000/46=44.1 for A500 Gr. B round HSS (Fy=46 ksi). Options A (40.9) uses Fy=50; C (53.9) is the limit for Fy=50 ksi (A500 Gr. C or A992); D (27.8) uses Fy=73 ksi. HSS with D/t<=44.1 are compact and can develop the full plastic moment. Above this limit, local buckling reduces the available flexural and compression strength per.

84. A — 39.1 kips

phiRn=0.75x2.4xFuxdxt=0.75x2.4x58x0.75x0.5=0.75x2.4x21.75=0.75x52.2=39.1 kips. Option B (65.0) uses Fu=120 (bolt, not plate); C (22.5) divides by 2 incorrectly; D (52.5) uses phi=1.0. The plate material (A36, Fu=58 ksi) governs over the bolt (A325, Fu=120 ksi) because the plate is thinner and weaker in bearing. Always use the plate Fu when the plate is the controlling element in bearing.

85. D — 1,552 psi

F'b=FbxCDxCMxCtxCFxCL=1500x1.15x1.0x1.0x1.0x0.90=1500x1.035=1,552 psi. Option A (1,725) omits CL; B (1,380) uses CD=1.0; C (1,200) uses only CM. All NDS adjustment factors are multiplied together, not added. The product CDxCL=1.15x0.90=1.035, confirming the answer. Note that for beams bent about the strong axis, CL can be less than 1.0 unless the compression edge is continuously braced or the beam meets the.

86. B — 7D (seven bolt diameters)

NDS 2018 Section 12.1.4.6 Table 11.5.1C requires a minimum end distance of 7D (seven bolt diameters) for a bolt loaded in bearing parallel to grain with the bearing toward the loaded end. Options A (4D) is the reduced minimum permitted with a 0.75 reduction factor; C (3.5D) is the minimum for nails, not bolts; D (10D) exceeds the.

87. C — 141 lb

NDS 2018 Table 12N gives Z~141 lb for a 16d common wire nail (D=0.162 in) single-shear connection with a 1.5-in Douglas Fir-Larch side member nailed into a DFL main member (G~0.50). Options A (90) is for a 10d nail in lower-density species; B (175) is for a 20d nail; D (200) is for a spike nail.

88. A — 48 in

TMS 402-16 Table 5.4.2 limits horizontal reinforcement maximum spacing to the smaller of one-third the wall height or 48 in for reinforced masonry walls without continuous horizontal reinforcement at every course. Option B (24) is more conservative than required; C (32) and D (16) are not TMS code values for this limit.

89. C — e <= B/6 from the centroid (kern condition)

TMS 402-16 Section 5.3.1.6.2 requires the resultant compressive force in a masonry column to remain within the kern, defined as e<=B/6 from the centroidal axis. For a square column, the kern is the inner square of side B/3 centered on the cross-section. Options A (B/4), B (B/3), D (B/8) are incorrect kern limits.

90. A — 1.92 kip.ft

M=wL2/8=0.24x82/8=0.24x8=1.92 kip-ft. Options B (3.84) doubles; C (0.96) halves; D (5.76) triples. Note: the question gives L=8 ft and w=0.24 kip/ft, giving M=0.24x64/8=1.92 kip-ft. In masonry design, the lintel must resist this moment using reinforcement (for reinforced lintels) or limited tensile masonry strength (for plain lintels). Arching action above the lintel (if permitted by TMS 402) can reduce.

91. B — C, D, E, and F -- special inspection is required when seismic demands are elevated

TMS 602-16 Article 1.5 requires special inspection of masonry for SDC C, D, E, and F -- where seismic demands are elevated and construction quality directly affects life safety. Options A (A and B only), C (D and above only), D (all SDC) are incorrect. SDC B requires periodic inspection but not special inspection per TMS 602.

92. D — 0.39 in2/ft

As_min=0.0018x12x18=0.389 in2/ft per ACI 318-14 Section 26.8.2 for Grade 60 deformed bars. Options A (0.25), B (0.31), C (0.18) use lower rhomin or incorrect cross-section dimensions. The 0.0018 minimum controls temperature and shrinkage cracking in the footing slab. This minimum often governs for lightly loaded footings -- the flexural reinforcement required for bearing may be less.

93. A — The free swell test (ASTM D4546 Method A) -- measuring volume change under no load when submerged in water

The free swell test (ASTM D4546 Method A) quantifies expansive potential by measuring the volume increase of a dried soil specimen upon wetting under no surcharge load. Options B (Proctor test), C (SPT N), D (grain size alone) do not identify expansive soils reliably. Expansive soils are typically clays with high plasticity (LL>50, PI>25) containing montmorillonite, smectite, or.

94. B — 51 in

For an edge column with one face flush with the slab edge (column below the slab), the critical perimeter has only 3 sides: bo=(c/2+d)+2(c+d)=(6+7)+2(19)=13+38=51 in. Option A (26) is for a corner column; C (52) is close but uses a different edge condition formula; D (20) omits the d extension.

95. C — Wellpoints use vacuum-assisted suction lift (limited to about 15 ft by atmospheric pressure); deep wells use submersible pumps inside the well casing, so drawdown depth is unlimited

Wellpoint systems use vacuum-assisted suction lift -- the theoretical maximum suction lift is one atmosphere (~34 ft water), limiting practical drawdown to ~15 ft. Submersible deep well pumps are installed below the water table inside the well casing and push water upward, eliminating the suction lift limitation. Options A (OSHA prohibition), B (larger diameter), D (aquifer direction) are.

96. D — Good to excellent rock quality with generally high allowable bearing pressures -- design is often controlled by structural failure of the footing rather than by rock capacity

High RQD (>75%) indicates a rock mass with relatively few discontinuities per meter, good structural integrity, and high mechanical strength -- generally supporting high allowable bearing pressures (often 30-200 tsf or higher). In sound rock, the footing size is typically controlled by structural failure in the concrete rather than by bearing capacity in the rock.

97. D — Differential settlement from non-uniform fill compaction, variable underlying soil, or inadequate quality control -- the most significant risk for footings on fill

The primary risk for footings on fill is differential settlement from non-uniform fill compaction, variable thickness of fill, poor quality fill materials, or variable strength of the underlying natural soil. Options A (hydrostatic uplift), B (lower than native strength), C (uniform compression) are secondary concerns or incorrect generalizations.

98. C — H-piles are low-displacement piles -- their open web lets soil flow between the flanges, minimizing lateral ground displacement and vibration to adjacent structures during driving

H-piles (HP sections with open web) are low-displacement piles that allow soil to flow between the flanges during driving, displacing far less soil volume than closed-end displacement piles of equivalent structural capacity. This minimizes ground heave, lateral soil displacement, and vibration damage to adjacent structures. Options A (higher capacity), B (easier splicing), D (always lighter) misidentify the H-pile.

99. A — 1.55 kip/ft

Pp=0.5KpgammaH2=0.5x3.0x0.115x32=0.5x3.0x0.115x9=1.553 kip/ft~1.55 kip/ft, acting at H/3=1 ft above the toe. Options B (3.11) doubles Pp; C (0.78) halves; D (4.66) triples. Passive pressure resistance at the toe of a retaining wall provides sliding resistance but requires wall movement to mobilize -- typically δ>=H/500 to develop full passive pressure.

100. B — 30 in

For compression reinforcement, ACI 318-14 Section 25.5.5.1 gives the lap splice length as max(0.0005fydb, 12 in)=max(0.0005x60000x1.0, 12)=max(30, 12)=30 in. Options A (12) is only the absolute minimum; C (22) uses 0.0004fy; D (18) uses 0.0003fy. Compression lap splices are shorter than tension splices because bearing at the bar end contributes to force transfer; the 30-in minimum ensures adequate.