



Fundamentals of structural analysis 5th edition solutions manual

1 2 2 3 15' 3 20' P15.3 Analyze by Setting up the Structure Stiffness Matrix. C B 10' 30 kips 10' A 14' 14' P10.6 Unknown: θ B FEMs PL (30)(20) == -75 k · ft, 8 8 2 2 wL (2)(28) 392 == FEM BC = k · ft, 12 12 3 Member end Moments Segment AB 2 EI M AB = (0 + θ B) - 75 20 2 EI M BA = (2 θ B + 0) + 75 20 2 EI 392 M BC = (2 θ B + 0) 28 3 2 EI 392 M BC = (2 θ B + 0) 28 3 2 EI 392 M BC = (2 θ B + 0) 28 3 2 EI 392 M BC = (2 θ B + 0) + 75 20 2 EI 392 M BC = (2 θ B + 0) + 75 20 2 EI 392 M BC = (2 θ B + 0) 28 3 2 EI 392 M BC = (2 θ B + 0) 28 3 2 EI 392 M BC = (2 θ B + 0) 28 3 2 EI 392 M BC = (2 θ B + 0) 28 3 2 EI 392 M BC = (2 θ B + 0) + 75 20 2 EI 392 M BC = (2 θ B + 0) + 75 20 2 EI 392 M BC = (2 θ B + 0) 28 3 2 EI 392 M BC = (2 θ B + 0) 28 3 2 EI 392 M BC = (2 θ B + 0) + 75 20 2 EI 392 M BC = (2 θ B + 0) 28 3 2 EI 392 M

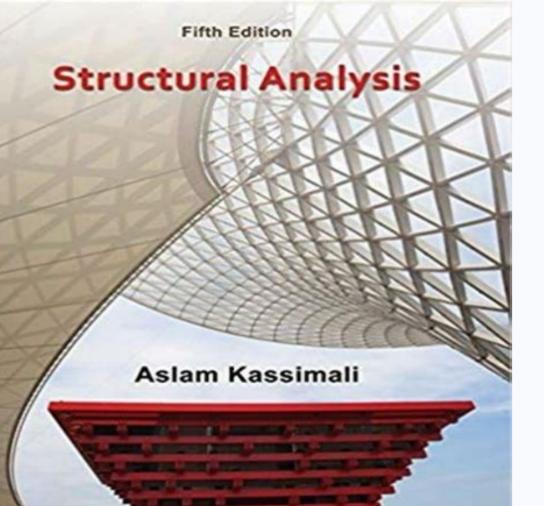
Draw the shear and moment curves and sketch the deflected shape. Determine the reactions in the rigid frame in Figure P9.47.) and the moment of inertia of all girders equals 4 12,000 in. Determine all bars forces. P9.33. C 1' D B hinge 18' Ax A Ex E 24' 24' Ey Ay P12.27 + Ax : $\Sigma MC = 0 @ x = 24 & c + Ax (18) + Ay (24 & c) = 0$ Where Ay = 0.5k Max. w = 4 kips/ft 60 kips B A 6' 12 kips x1 8 kip • ft x2 6' C 7' 5' P5.10 æ 7ö $\Sigma M A = cc \div (4)7 + 7(60) - 8 - 12C y = 0$ $ce^{2} \div oC y = 42.5$ kips $\Sigma Fy = Ay - 4(7) - 60 + 42.5 = 0$ Ay = 45.5 kips $\Sigma Fy = Ax + 12 = 0$ Ax = 12 kips Cut of segment AB $\Sigma Fy = 45.5 - 4x + V = 0$ V = 45.5 - 4 x kips æxö $\Sigma M = M + cc \div + 4x - x (45.5) = 0$ $ce^{2} \div oM x = 45.5 x - 2 x 2$ kip · ft Cut segment CB æ 169 $\ddot{o} \div \div \cdot C = 46.04$ kips C y $e^{2} = ccc y$ $ce^{2} + 46.04 = 0$ V = -46.04 kips $\Sigma MC = M x - x (46.04) = 0$ M x = 46.04 x kip · ft 5-11 Copyright @ 2018 McGraw-Hill Education. A C B 24' 16' P9.16 (a) Compatibility Equation: $\Delta A = -1.2 ec$ $\Delta A + \delta A X A = 0 \delta A A$: From Table A.3 $\delta A = Pa 2 1(24)2$ (L + a) = (16 + 24) 3EI Select RA as the Redundant 7680(1728) = 1.525 in/k (29,000)(300) Compatibility Equation: = -1.2 ec A = 0 Released Structure w/Support "B" low. B 12' C 8' VA, MB, MC, and RC P12.10 Müller-Breslau 12-11 Copyright @ 2018 McGraw-Hill Education. 4 Compute all reactions.

are shown in Figure P2.12. The windward and leeward wind pressure profiles in the long direction of the warehouse are also shown. Establish the wind forces based on the following information: basic wind speed = 40 m/s, wind exposure category = $C_r K_r = 0.85$, $K_r = 1.0$, $G = 0.85$, and $C_r = 0.8$ for windward wall and -0.2 for leeward wall. Use the K_r values listed in Table 2.4. What is the total wind force acting in the long direction of the warehouse?	P2.12
Use I = 1	Total Windforce, Fr., Windward Wall
$q_r = 0.613 V^2$ (Eq. 2.4b)	$F_{w} = 481.8[4.6 \times 20] + 510.2[1.5 \times 20]$
= 0.613(40) ² = [980.8 N/m ²]	+ 532.9[1.5 × 20] + 555.6[1.4 × 20]
$q_i = q_j I K_i K_i K_j$	F _e = 91,180 N
$q_{\rm c} = 980.8(1)(K_{\rm c})(1)(0.85) = 833.7 K_{\rm c}$	For Leeward Wall
0-4.6 m: q = 833.7(0.85) = 708.6 N/m ²	p = q GC = q (0.85)(-0.2)
4.6-6.1m; g = 833.7(0.90) = 750.3 N/m ²	g = g at 9m = 817.1 N/m ² (above)
	q = q at $sm = s(r, r)qm$ (above)
6.1 = 7.6 m; q, =833.7(0.94) = 783.7 N/m ²	$p = 817.1 (0.85)(-0.2) = -138.9 \text{ N/m}^2$
7.6 = 9 m; q, = 833.7(0.98) = 817.1 N/m ²	
For the Windward, Wall	Total Windforce, F., on Leeward, Wall
$p = q, GC_{e}$ (Eq. 2.7)	$F_1 = 20^{-9}(1-138.9) = -25,003 \text{ N}^2$
	Total Force = $F_w + F_z$
where $GC_{\rho} = 0.85(0.8) = 0.68$ p = 0.68 q.	= 91,180N + 25,003
24.000	= 116, 183.3 N
$0-4.6 \text{ m}$ $p = 481.8 \text{ N/m}^2$	*Both F, and F, Act in Same Direction.
4.6-6.1 m p = 510.2 N/m ²	aver r are r we name orecton.
$6.1-7.6 \text{ m} p = 532.9 \text{ N/m}^4$ $7.6-9 \text{ m} p = 555.6 \text{ N/m}^4$	

ki ps ki ps ki ps ki ps ki ps ki ps vi ps ki ps 9' A 10 kips 10 @ 12' = 120' ΣP 5k 9 + 2.5k 2 = 0.417 kips/ft L 120 ft I of truss considering flanges only: I = 10 (54)2 2 = 58,320 in 4 Δf = bottom chord A = 10 in 2, d = 9' B P13.12 Load/ft w = top chord A = 10 in 2 2. 8m w = 5 kN/m A C B I 1.2I 9m 12 m P11.2 Distribution Factors I 10 I 10 = DFBA = 9 90 19 1.2 I 9I 9 K BC = DFBC = 12 90 19 19 I $\Sigma K = 90$ Fixed end Moments (see Table 2) front overleaf K AB = 5 (9) wL2 = 33.75 kN · m 12 12 2 5 (12) 40 (64) 4 wL2 PB 2 8 = 2 = -131.11 kN · m 2 2 12 12 L (12) 2 FEM BA = FEM AB = FEM AB = FEM BC 40 (8)16 wL2 PB 2 82 (12) + = + 95.55 kN · m 2 2 12 12 L (12) 2 FEM CB = Moment Distribution A 10/19 B 9/10 - 33.75 + 33.75 - 131.11 + 51.24 + 46.12 84.99 84.99 + 25.62 \leftarrow -8.13 C \rightarrow VAB = 13.96 kN VBC = 53.86 kN VBA = 31.04 kN VCB = 46.14 kN + 95.55 FEM'S 23.06 DEM'S 118.61 End Moments (kN·m) RB = VBA + VBC = 31.04 + 53.86 RB = 84.90 kN 11-3 Copyright © 2018 McGraw-Hill Education. The width of the footing is controlled by the allowable soil pressure and does not affect the analysis. Determine the reactions and draw the shear and moment curves for the beam. Use the computer program to plot the deflected shape. P14.6. Analyze the beam in Figure P14.6. Compute all reactions and draw the shear and moment diagram. What is the minimum required value of I if the deflection of point C is not to exceed 0.4 in.?

B P8.25. w = 12 kN/m B A 10 m C 14 m P10.4 θ A = θ C = 0 ü 2 EI 24 (10 ii (0 + θ B) ii 10 8 i 2 EI 24 (10 ii M BA = (2 θ B + 0) + ii 14 12 ii 2i 2 EI 12 ((14) ii M BA = (2 θ B + 0) + ii 14 12 ii 2i 2 EI 12 ((14) ii M BA = (2 θ B + 0) + ii 14 12 ii 2i 2 EI 12 ((14) ii M BA = (0 + θ B) + ii 14 12 ii 2i 2 EI 12 ((14) ii M BA = (2 θ B + 0) + ii 10 8 i 2 EI 24 (10 ii M BA = (2 θ B + 0) + ii 10 8 i 2 EI 24 (10 ii M BA = (2 θ B + 0) + ii 10 8 i 2 EI 24 (10 ii M BA = (2 θ B + 0) + ii 10 8 i 2 EI 24 (10 ii M BA = (2 θ B + 0) + ii 10 8 i 2 EI 24 (10 ii M BA = (2 θ B + 0) + ii 10 8 i 2 EI 24 (10 ii M BA = (2 θ B + 0) + ii 10 8 i 2 EI 24 (10 ii M BA = (2 θ B + 0) + ii 10 8 i 2 EI 24 (10 ii M BA = (2 \theta B + 0) Substi. Draw the influence lines for the horizontal and vertical reactions at support A and the forces or components of force in members BC, CM, and ML. C 48 kips 6' D 24 kips B 6' A E F 8' 8' P8.8 E = 29,000 ksi and A = 2 in.2 for all members. Continued Case 3: $\Delta D = 0.272$ in. A w = 2 kips/ft 12' 2 kips D B C 8' E 16' 4' P10.33 + Unknowns: Δ , θ B θC , θE , (B) Joint C: $\Sigma MC = 0 \Delta \Delta \Psi AB = -$, $\Psi CE = 12.8 \Psi AB - \Delta/12 = \Psi CE \Delta/8.2 \Psi AB = -\Psi CE 3 wL2 (2(16))^2 = = 42.66 \text{ kip} \cdot \text{ft FEM BC} = 12.12 (1) \text{ Express end moments in terms of displacements } 2 \text{ EI M AB} = (\theta - 3\Psi AB) 12.8 \text{ EI M BA} = (\theta - 3\Psi AB) 12.8$ $\theta E - 3\Psi CE$) 8 2 EI M EC = [2 $\theta E + \theta C - 3\Psi CE$] 8 (A) Joint B: $\Sigma M B = 0$ Joint C (C) Joint E: $\Sigma M E = 0$ Shear Equation Where V1 = (D) 2 - (2) Equilibrium Equations M BA + M BA 12 & V2 = 2 k - V1 + V2 = 0 M CE + 0 + 2Fx = 0 M AB + M BA 12 & V2 = 2 k - V1 + V2 = 0 M CE + 0 + 2Fx = 0 M AB + M BA 12 & V2 = 2 k - V1 + V2 = 0 M CE + 0 + 2Fx = 0 M AB + M BA 12 & V2 = 2 k - V1 + V2 = 0 M CE + 0 + 2Fx = 0 M AB + M BA 12 & V2 = 2 k - V1 + V2 = 0 M CE + 0 + 2Fx = 0 M AB + M BA 12 & V2 = 2 k - V1 + V2 = 0 M CE + 0 + 2Fx = 0 M AB + M BA + M BA + 0 + 2Fx = 0 M AB + M BA 12 & V2 = 2 k - V1 + V2 = 0 M CE + 0 + 2Fx = 0 M AB + M BA 12 & V2 = 2 k - V1 + V2 = 0 M CE + 0 + 2Fx = 0 M AB + M BA + 0 + 2Fx = 0 M AB + M BA 12 & V2 = 2 k - V1 + V2 = 0 M CE + 0 + 2Fx = 0 M AB + M BA + 0 + 2Fx = 0 M AB + M BA + 0 + 2Fx = 0 M AB + M BA + 0 + 2Fx = 0 M AB + M BA + 0 + 2Fx = 0 M AB + M BA + 0 + 2Fx = 0 M AB + M BA + 0 + 2Fx = 0 M AB + M BA + 0 + 2Fx = 0 + Values in Step1 into Equil EQ's and Solving for the Unknown Displacements. Given: I = 360 in. C P9.26. (a) (b) (c) (d) b = 11 r = 5 n = 7 b = 5 (e) r = 3 n = 4 (b + r = 16) > (2n = 14) (b + r = 8) > (2n = 8) 2 deg. Determine the maximum tension in the cable and the reactions at supports A and D. P6.3. The cable in Figure P6.3 supports girder DE uniformly loaded with 4 kips/ft. P12.25. Consider the truss in Figure P9.34 without the applied loads. Neglect axial deformations. PINNED COLUMN BASES Approximate method yielded exact results. A five-story building is shown in Figure P2.10. (b) To avoid ponding* of rainwater on the roof, the girder is to be fabricated with a camber equal to the deflection at midspan of the roof girder produced by the uniform load. too long and support D settles 0.25 in., compute the vertical displacement of joint E. and 4 I = 320 in. w = 2.4 kips/ft D P11.32. The beam shown in Figure P12.46 is subjected to a moving concentrated load of 80 kN. Determine the support reactions at A and C. A B 12' C 6' P9.14 Geometry $\Delta BO = 0$ (a) Use RB as Redundant $0 = \Delta BO + \delta BB$ [X B] 0 = -0.2 + 0.115 X B X B = 10.5k (b.) Redundant Applied as Force Release Structure with Original Loads K AY = -0.16 k M A = -34.1ft k RBY = 6.16 k RDY = 6 k Release Structure w/Unit Value of RB 9-16 Copyright © 2018 McGraw-Hill Education. MA B RA C D hinge RC 6' 8' P12.4 (a) (b) Uniform Load of Variable Length 1.2 k/ft é ù 1 + RA max. B C 12' A D 9' 10' P3.12 Figure (b): 19 (19) = 0 2 180.5 = 10 (1.33Bx) + 12 Bx $\Sigma M d = 10 By + 12 Bx - Bx = 7.125 kips$ Ax = Bx = 7.125 kips Ay = By = 9.5 kips Ay = By = 0.5 kips Ay = 0. $\Sigma Fx = Bx + Dx = 0$ $\Sigma Fy = -19 + By + Dy = 0$ Dx = -7.125 kips Dy = 9.5 kips 3-13 Copyright © 2018 McGraw-Hill Education. Select the origin at D. w = 2 kips/ft P5.20. w = 4 kN/m P9.22. 15 kips P9.6. Compute the reactions and draw the shear and moment curves for the beam in Figure P9.6. EI is constant. lightweight reinforced concrete masonry units with an average weight of 90 psf. Classify the structures in Figure P3.32. 16' P15.2. Continued Substitute Stiffness Coefficients into Equations (1) and (2), Solve Δx and $\Delta y \approx 43$ AE $\ddot{o} \div \dot{c} \approx 32$ AE $\ddot{o} \div \ddot{c} \approx 32$ AE $\ddot{c} \approx 32$ AE $\ddot{c} \approx 32$ AE $\ddot{c} \div \ddot{c} \approx 32$ $(\sin fx) Dy = (1) cc$ AE = 4 + cc AE = 4 + cc AE = 3 + cc C + ccc C + cc constant for all members. 2 and E = 29,000 kips/in. B A hinge $r = 20 \text{ m} \theta = 45^{\circ} \text{ C} \text{ P6.19}$ Determine Reactions: Equation (1): + $\Sigma \text{ MC} = 0$; Ax (14.14) + Ay (34.14) - 56.56(27.07) - 80(10) = 0 Equation (2): $\Sigma \text{ MB}$ for segment AB, + $\Sigma \text{ M} \text{ B} = 0$; Ay (14.14) - 56.56(7.07) = 0 Solve EQ'S (1) and (2) Ax = 48.3 \text{ kN} Ay = 48.3 kN $\Sigma \text{ Fx} = 0$; C x = $48.3 \text{ kN} \neg \Sigma Fy = 0$; C y = 80 + 56.56 - 48.3 = 88.26 kN 6-21 Copyright © 2018 McGraw-Hill Education. For all bars, area = 2400 mm and E = 200 GPa. (b) Assuming that no load acts, determine the horizontal displacement of joint B if support A moves 20 mm to the right and 30 mm down and support E moves downward 36 mm., area of all truss 2 2 bars = 1 in. 200 lb/ft P10.13. E P12.9. Beam AD is connected to a cable at C. 2 Given: E = 29,000 kips/in. 2 kips/ft P11.25. Equations at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 At Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 At Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BC = 0 At Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 At Joint A: M • m w = 6 kN • m A C B RA = 28.31 kN RB = 28.69 8m P5.36 5-38 Copyright © 2018 McGraw-Hill Education. L = 10' A = 1.2 in.2 120 kN P14.10. Δ AD + RA δ AA = 0 980 88.2 + RA = 0 AE AE RA = 11.11 kN - P-System Releved Structure Q-System for Δ AD P & Q System for Δ AD P & Q System for δ AA Bar Force and Reactions 9-40 Copyright © 2018 McGraw-Hill Education. L = 10' A = 1.2 in.2 120 kN P14.10. Δ AD + RA δ AA = 0 980 88.2 + RA = 0 AE AE RA = 11.11 kN - P-System Releved Structure Q-System for Δ AD P & Q System for δ AA Bar Force and Reactions 9-40 Copyright © 2018 McGraw-Hill Education. L = 10' A = 1.2 in.2 120 kN P14.10. Δ AD + RA δ AA = 0 980 88.2 + RA = 0 AE AE RA = 11.11 kN - P-System Releved Structure Q-System for Δ AD P & Q System for δ AA Bar Force and Reactions 9-40 Copyright © 2018 McGraw-Hill Education. A B C I = 600 in.4 I = 600 in.4 I = 200 in.4 D 15' 20' P11.26 Analysis for temperature forces. M AB = M ED = -58.3 kip-ft M BA = M DE = -58.3 kip-ft M BA = M DE = -58.3 kip-ft M BA = M DE = -39.3 kip-ft M BC = M DC = 39.3 kip-ft M CB = -MCD = -50.8(0.711) = M CB = -MCD = -36.3 kip-ft M CF = -36.3 kip-ft M CF = -36.3 kip-ft M CF = -36.3 kip-ft M BA = M DE = -36.3 kip-ft M CF = -36.3 kip-ft M -101.3(0.711) = MCF = -72 kip-ft Ax = E x = -11.4(0.711) = Ax = E x = -8.1 kips $\neg Ay = -E y = -4.7$ kips Fx = -19.4(0.711) = Fx = -13.8 kips $\neg Horizontal$ deflection is given by the ratio of applied load to S from an imposed unit sidesway. P7.1. Derive the equations for slope and deflection for the beam in Figure P7.1. Compare the deflection at B with the deflection at midspan. Columns transmit only axial load from the roadway beams to the arch. w = 2 kN/m P5.24. RB = 54.15k + $\Sigma M P$. G P4.26. C 6 kips 12' B G F D 8' E A 8' 10' 10' 8' P4.33 Freebody Right Side of Centerline : Final Results: + $\Sigma M A = 0$; 30 k (10 ¢) - 6 k (20 ¢) - E y (20 ¢) = 0 E y = 9k + $\Sigma Fy = 0$ 0; Ay - $30 + 9 = 0 \ k + k \ \Sigma MC = 0$; E x ($20 \ c$) - 9k (10) = 0 E x = $4.5k \neg Ay = 21k$ Entire Structure: + $\Sigma Fx = 0$; - 6k - 4.5k + Ax = 0 Ax = $10.5k \ 4-34$ Copyright © 2018 McGraw-Hill Education. A uniform live load of 2 kips/ft is applied along the top deck girder DE, assume that the load acts uniformly on the arch through the vertical struts. (a) Considering all joints as rigid, compute the axial forces and moments in all bars and the deflection at midspan when the three 24-kip design loads act at joints 7, 8, and 9. P2.20. D 1' C w = 3 kips/ft 11' 20 kips E F G 11' B 22' 4' 4' 12' 2' 2' P3.35 Freebody Timber Member + $\Sigma Fx = 0$; $20 \ k - BFx = 0$; $BFx = 20 \ k \neg BFy / 11 = BFx / 10 \ c BFy = 22 \ k$ = BFx 2 + BFy 2 = (20)2 + (22 k)2 BF = 29.73k Compression in the link + $\Sigma M e = 0$; - BFy (4 ¢) - CG (8 ¢) = 0 - 22 k (4 ¢) - CG (8 ¢) = 0 CG = 11k Compression in the link $\Sigma Fy = 0$; + DE + CG + BFy - 72 k = 0 DE - 11k + 22 k - 72 k = 0 DE = 61k Tension in the link Entire Structure Compute Reactions + $\Sigma M A = 0$; 20 k (22 ¢) - 3 k/ft (24 ¢) $(16 \text{ }) + \text{MA} = 0 \text{ MA} = 7121 \cdot \text{k} + \dots \Sigma Fx = 0$; $20 \text{ }\text{k} - \text{Ax} = 0 \text{ }\text{Ax} = 20 \text{ }\text{k} \neg \Sigma Fy = 0$; + 3 k/ft (24) + Ay = 0 Ay = 72 k $3-36 \text{ Copyright} \otimes 2018 \text{ }\text{McGraw-Hill} \text{ }\text{Education}.$ Continued Deflection at midspan (Joint 8) = 0.987 in. In member ABC consider only the strain energy associated with bending. Gilbert SOLUTIONS MANUAL CHAPTER 13: APPROXIMATE ANALYSIS OF INDETERMINATE STRUCTURES 13-1 Copyright }\otimes 2018 \text{ }\text{McGraw-Hill} \text{ }\text{Education}. A B D C 5m 5m 5m P10.14 Since the structure and the applied load are symmetrical with respect to a vertical axis thru support C, $\theta C = 0$ and the joint at C can be treated as a fixed EUD. , $\alpha = 6.5 \times 10$ (in./in.)/°F, 2 and E = 30,000 kips/in Establish the wind pressures on the building in Problem P2.13 when the windward roof is subjected to an uplift wind force. + $\Sigma MC = 0$; FDBX (6 ¢) - 2.25(12 ¢) = 0 FDBX = 4.5k FDBX F = DBY ¢ 12 9¢ FDBY = 3.375k (tension) Freebody Joint B + $\Sigma Fy = 0$; 2.25k + 3.375k - FCBY = 0 FCBY = 5.625k FCBY F = CBX 15¢ 12 ¢ FCBX = 4.5k (compression) 4-32 Copyright © 2018 McGraw-Hill Education. Continued Compute Reactions and Member end Moments + 9 Σ M B = 0; 54 kN ' - RD 9 - 5.5 = 0 2 RD = 26.39 kN + Σ Fy = 0; VBD = 54 - 26.39 = 27.61 kN + Member BD: Shear & Moment Diagrams Σ M B = 0 110.5 + 55.25 - RAX 6 = 0 RAX = 27.625 kN 10-25 Copyright © 2018 McGraw-Hill Education. Carry out an approximate analysis of the frame by estimating the location of the points of inflection in the girder. 2 in 2 A 24 kips P14.1 AE Δ 1(30,000)1¢¢ = = 156.25k L 16(12) 2(30,000)0.8¢¢ FAB = FAD = = 200 k 20(12) (a) FAC = Joint Equilibrium \Sigma Fy = 0; 160(2) + 156.25 K = 0 K = 476.25k / ¢¢ (b) Displacement due to 24k F = ΔK 24 k = $\Delta (476.25) \Delta$ = 0.0504 ¢¢ (c) Bar Forces due to 24k @ A (156.25) = 7.88k 1¢¢ (200) = 0.0504 ¢¢ FAB = FAD 14-2 Copyright © 2018 McGraw-Hill Education. P11.5. Analyze each structure by moment distribution. Given: 2 4 E = 29,000 kips/in. Using the method of sections, determine the forces in the bars listed below each figure. 14' P = 80 kN P8.41.

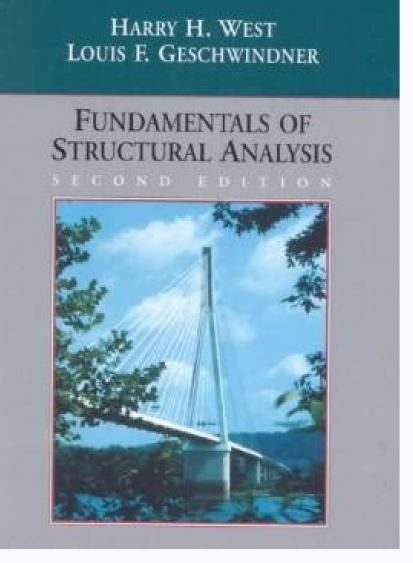
? A P4.45. Compare the results of Problem 13.21 (a) and (c) with Problem 13.22 (a) and (c). 5 6 7 4 8 3 9 2 10 1 11 20 19 18 17 16 15 14 13 48 kips P6.34 Uniformly distributed dead load + 48-kip live load Reactions (kips), Deflections (in), and Deflected Shape 6-41 Copyright © 2018 McGraw-Hill Education. 10 kN/m 2 kN/m D A C 20 kN 5m 3m B 5m 5m P5.28 + $\Sigma M A = 0$; + $\Sigma Fy = 0$; $\approx 20 \ \ddot{o} 20(5m) + 40 \ cç \div - 5 \ RB + 20(2m) = 0 \ ce^{3} \div \sigma \ RB = 81.33 \ kN - 60 \ kN + 81.33 \ RAY = 0 \ RAY = 21.33 \ kN - 60 \ kN + 81.33 \ RAY = 21.33 \ kN - 60 \ kN + 81.33 \ RAY = 21.33 \ kN - 60 \ kN + 81.33 \ RAY = 21.33 \ kN - 60 \ kN + 81.33 \ RAY = 21.33 \ kN - 60 \ kN + 81.33 \ RAY = 21.33 \ kN - 60 \ RAY = 21.33 \ kN - 60 \ RAY = 21.33 \ kN - 60 \ RAY = 21.33 \ RAY = 21$





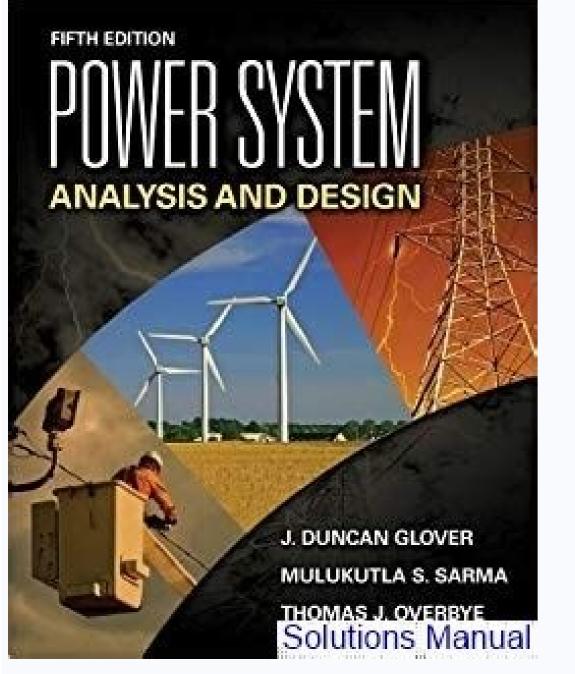
P6.34. Using the influence lines, determine the maximum live load force (consider both tension and compression) produced by the 54-kip truck as it transverses the bridge, which consists of two trusses.

x 2 æç x \ddot{o} ÷ c ÷- M = 0 4 çè 3 ÷ø æxö M = 12 + x - R çç ÷÷ èç 3 ø 1(x) + 12 - M = 12 + x - Check@ x = 6: V and M Diagrams 1 1kN - ['] V ¢ - V = 0 2 1 æxö V = 1 - ['] çç ÷÷ 2 çè 2 ø ÷ x3 12 Ans. 2 2 n = 3 AE AE éê æ 3 \ddot{o} ÷ \dot{u} 43 AE K11 = Σ cos2 fx = 1 + çç ÷÷ + çç ÷÷ K11 = ê ú 1 è ø è ø L L ë 5 5 û 25 L n = 3 AE é ù æ 3 \ddot{o} æ 4 \ddot{o} æ 4 \ddot{o} æ 3 \ddot{o} æ 4 \ddot{o} æ 3 \ddot{o} æ 4 \ddot{o} a 4 \ddot{o} æ 3 \ddot{o} æ 4 \ddot{o} æ 4 \ddot{o} æ 3 \ddot{o} è 5 ø û 1 L l e e a 3 \ddot{o} æ 4 \ddot{o} æ 4 \ddot{o} æ 4 $\ddot{o$

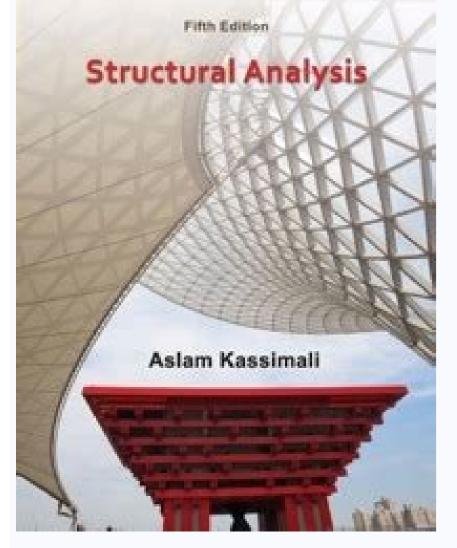


A P12.1. Draw the influence lines for the reaction at A and for the shear and moment at points B and C. Continued 3x (e) B-C + $\Sigma Fy = 0$; $M(x) V - 3x + 22 = 0 V = -22 + 3 x z x V(x) + \Sigma M z = 0$; $A-B + \Sigma Fy = 0$; Cy = 22 kips x M + 3 x - 22 x = 0 2 1 M = 22 x - x 2 2 8 kips $-8 k - V = 0 V(x) M(x) V = -8 k A 20 - x + \Sigma M Z = 0$; z - 8(20 - x) - M = 0 M = 8 x - 160 (f) M at section (1) RA + x = 112 3 M = 22(11) - (11)2 = 60.5 kip \cdot ft 2 (g) Mmax, Set V = 0; $-22 + 3 x = 0.22 = 7.332 3 3 = 22^{-7} 7.33 - (7.33)2 2 = 161.26 - 80.59 V = Mmax Mmax = 80.67$ kip \cdot ft + (h) $\Sigma Fy = 0$; -8 + 34 - 3 x - V = 0 V = 26 - 3 x 3x 8 kips $3x \cdot x \Sigma M z = 0$; -8(4 + K) + 34 x - M = 0 2 3 M = -32 + 26 x - x 2 2 (i) Moment at section

(1) Let $x = 5 \notin + M(x) A 4' x By = 34$ kips $x \approx 36$ M = -32 + 26(5) - $cc \div 52$ c $2 \div 0$ M = 60.5 kip \cdot ft 5-14 Copyright © 2018 McGraw-Hill Education. P11.29. The footing has been proportioned so that the resultant of the column loads passes through the centroid of the footing, producing a uniformly distributed soil pressure on the base of the footing. 22' 12' H B P12.7. Load moves along girder BCDE. A 60 kN·m B 5m 5m C 5m D 5m 10 m P3.8 FBD "AB" Ay = Fby = 20 kN By Symmetry.



Alternate Computation by Virtual Work: 1 P-System 100 dx 1 1 kN $(\delta = \Sigma \circ MQ M P M 2 dx = EI 100 EI \circ P 1.5 \acute{e} 4 u 1 \acute{e} 4 M 2 dx + 2 M dx \acute{u} \delta = P P \circ \circ \acute{e} \acute{u} 100 EI \acute{e} 0 - 1.5 \acute{u} ú$ Use Q-System = 2 EI (20 - 3Ψ) L 2 EI M EF = -44.44 = Δ = Member AB Member AF 1.5 $\acute{e} 4 u \acute{e} 4 (-44.5 + 25 x) 2 dx + 2 (29.67 x 2) dx \acute{u} \circ \circ \acute{e} 4 u \acute{e} 4.44 = Δ = Member AB$ Member AF 1.5 $\acute{e} 4 u \acute{e} 4 (-44.5 + 25 x) 2 dx + 2 (29.67 x 2) dx \acute{u} \circ \circ \acute{e} 4 u \acute{e} 4.44 = Δ = Member AB$ Member AF 1.5 $\acute{e} 4 u \acute{e} 4 (-44.5 + 25 x) 2 dx + 2 (29.67 x 2) dx \acute{u} \circ \circ \acute{e} 4 u \acute{e} 4.44 = Δ = Member AB$ Member AF 1.5 $\acute{e} 4 u \acute{e} 4 (-44.5 + 25 x) 2 dx + 2 (29.67 x 2) dx \acute{u} \circ \circ \acute{e} 4 u \acute{e} 4.44 = Δ = Member AB$ Member AF 1.5 $\acute{e} 4 u \acute{e} 4 (-44.5 + 25 x) 2 dx + 2 (29.67 x 2) dx \acute{u} \circ \acute{e} 6 u \acute{u} - 1.5 u \acute{u} 4.3454.3) + 2(1380.7) u \acute{u} = 100 EI \acute{e} 0 - 1.5 u \acute{u} 4.44 = Δ = Member AB$ Member AF 1.5 $\acute{e} 4 u \acute{e} 4 (-44.5 + 25 x) 2 dx + 2 (29.67 x 2) dx \acute{u} \circ \acute{e} 6 u \acute{u} - 1.5 u \acute{u} 4.44 = Δ = Member AB$ Member AF 1.5 $\acute{e} 4 u \acute{e} 4 (-44.5 + 25 x) 2 dx + 2 (29.67 x 2) dx \acute{u} \circ \acute{e} 6 u \acute{u} - 1.5 u \acute{u} 4.44 = Δ = Member AB$ Member AF 1.5 $\acute{e} 4 u \acute{e} 4 (-44.5 + 25 x) 2 dx + 2 (29.67 x 2) dx \acute{u} \circ \acute{e} 4 u \acute{e} 4.44 = Δ = Member AB$ Member AF 1.5 $\acute{e} 4 u \acute{e} 4 (-44.5 + 25 x) 2 dx + 2 (29.67 x 2) dx \acute{u} \circ \acute{e} 4 u \acute{e} 4.44 = Δ = Member AB$ Member AF 1.5 $\acute{e} 4 u \acute{e} 4 (-44.5 + 25 x) 2 dx + 2 (29.67 x 2) dx \acute{u} \circ \acute{e} 4 u \acute{e} 4.44 = Δ = Member AB$ Member AF 1.5 $\acute{e} 4 u \acute{e} 4 (-44.5 + 25 x) 2 dx + 2 (29.67 x 2) dx \acute{u} \circ \acute{e} 4 u \acute{e} 4.44 = Δ = Member AB$ Member AF 1.5 $\acute{e} 4 u \acute{e} 4 (-44.5 + 25 x) 2 dx + 2 (20.67 x 2) dx \acute{e} 4 u \acute{e} 4.44 = Δ = Member AB$ Member AF 1.5 $\acute{e} 4 u \acute{e} 4.44 = Δ = Member AB$ Member AF 1.5 $\acute{e} 4 u \acute{e} 4.44 = Δ = Member AB$ Member AF 1.5 $\acute{e} 4 u \acute{e} 4.44 = Δ = Member AB$ Member AF 1.5 $\acute{e} 4 u \acute{e} 4.45 = 4 (-40.45 + 4.44 = Δ = Member AB$ Member AF 1.5 $\acute{e} 4 u \acute{e} 4.45 = 4 (-40.45 + 4.44 = Δ = Member AB$ Member ant intro



Analyze the structure in Figure P11.28 by moment distribution. D A B 12' F C 30' E 8' 22' G 8' P12.6 2k/ft Uniform Load Applied over Entire Length of Beam Rxns & M = W(Areas) é1 ù 1 1 1 By = 2 k / ft ê (-0.27)(30 ¢) + (0.1(30 ¢) - (0.053)(12)ú êë 2 úû 2 2 2 k By = 53.06 é1 ù 1 1 1 C y = 2 k / ft ê (-0.4)(12 ¢) + (1.27)(60) - (0.46)(30 ¢) + (0.25)(12 ¢)ú êë 2 úû 2 2 2 k C y = 60.6 é1 ù 1 E y = 2 k / ft ê (1.364)(52 ¢) - (0.744)12ú êë 2 úû 2 k E y = 62 é1 ù Gy = 2 k / ft ê (-8)(30 ¢) + (2.91)(30 ¢) - (1.59)(12 ¢)ú ê2 úû 2 2 ë MC = 171.8 ft · k é1 ù 1 MC = 2 k / ft ê (-8)(30 ¢) + (4.36)(12)ú êë 2 úû 2 MG = -187.7 ft · k 12-7 Copyright © 2018 McGraw-Hill Education. 30 kips P10.28. E P-System (kips) Therefore, (Δ AS + Δ AO) + δ AA X = Δ A D -75 (C) = 0 150 (T) (0 + 2.578) + 0.0538 X A C 3 in.2 1 150(15)(12) 5 -125(25)(12) + 1(29,000) 3 3(29,000) 1 in.2 FQ2 L 2 (T) \deltaAA = Σ Q-System (kips) X A = Ay = -47.9 kips A B 0 C 0 100 150 150 E 100 Final Results (kips) D 54.1 (T) -75 (C) A Ay = 47.9 -64 (C) By = 54.1 -64 (C) Cy = 102 9-38 Copyright © 2018 McGraw-Hill Education. If the truss is to be designed for a uniform live load of 0.32 kip/fit that can be placed anywhere on the span in addition to a concentrated live load of 24 kips that can be positioned where it will produce the largest force in bar CG, determine the maximum value of live load force (tension, compression, or both) created in bar CG. 194.4 kips 2.3' 453.6 kips 16' 12" 18" w = 24 kips/ft 27' P5.51 5-53 Copyright © 2018 McGraw-Hill Education.

Given: EI is constant for the 6 4 beam. reinforced concrete slab, supported on four steel stringers. Hint: Maximum deflection occurs at point of zero slope. 2 The facility, with an average weight of 90 lb/ft for both the floor and roof, is to be designed for the following seismic factors: SDS 0.27g and SD1 0.06g; reinforced concrete frames with an R value of 8 are to be used.

Frame is clamped in deflected position. P = 10 kips P13.8. The frame in Figure P13.8 is to be constructed with a deep girder to limit deflections. A N M L K I J 6 @ 10' = 60' P13.14 6 kips 15 9 + 9 0 - 27 (C) 3 E + 3 24 (T) 15 (T) N F - 27 (C) + 3 3 24 (T) M G - 24 (C) + H - 15 (C) 6 6 + 15 15 0 15 (T) L 3 kips -18 (C) + D - 9 (C) -24 (C) 6 kips -15 (C) 15 C -15 (C) -15 (C) 6 kips -9 (C) -18 (C) B 6 kips 6 kips -6 (C) 3 kips K J Ay = 18 kips Iy = 18 kips (a) Only tension diagonals effective. C 20 kips 15' A F D E 10 kips 15' 15' P4.11 Solve by Method of Joints Start at Joint F B -10 C 20 kips 30 10 + 10 10 20 kips A + 30 + 30 + 30 F 10 kips 10 kips + 30 E 30 kips 15 ft 30 D 30 kips 3@15 ft = 45 ft 4-12 Copyright © 2018 McGraw-Hill Education. A B I C I 2I 6m D 4m 4m P11.14 - wL2 -8(4)2 = = -10.67 N-m 12 12 I 1 3 æ 2I ö 1 = = , K BD = cc ÷÷ = 4 4 4 èc 6 ø 4 FEM BC = -FEMCB = K AB = K BC DFBA = DFBC = DFBD = 1 4 1 4 1 4 + + 1 4 = 1 3 1.78 1.78 0 MAB C FEM 3.56 3.56 0 -7.11 MBC 3.56 D -10.67 FEM MBC D FEM A 12.45 MCB 1.78 C 10.67 FEM B 1/3 1/3 C 0 3.56 3.56 M AB = 1.78 kN-m M BD = 3.56 kN-m M CB = 12.45 kN-m M BD = 3.56 kN-m M CB = 12.45 kN-m M C = 12.45 kN-m M C

(c) Determine the axial forces in all bars. 5 5 5 5 ki ps ki ps 5 5 ki ps ki ps A 5 5 5 ki ps ki ps 2. If the structure is stable, indicate if determinate or indeterminate. 3 P = 36 kips P9.10.

All truss joints are pinned. The sag at midspan is 12 ft. 12 ft P6.16. Shear at Support (a) absolute Max. Support D Settles 0.25" P-System Q1-System for δ EV Area of Bars AB, BD, CD, 5 in 2 Area all other Bars = 3 in 2 E = 30,000 ksi Q2-System for δ EH 8-16 Copyright © 2018 McGraw-Hill Education. T=? -wL2 Pab 2 -2(60)2 20(20)(40)2 - 2 = 12 12 L (60)2 = -777.78 ft · k FEM AC = wL2 Pba 2 2(60)2 20(40)(20)2 + 2 = + 12 12 L (60)2 = 688.89 ft · k FEMCA = + Σ MC = 0; RA 60 - 20(40) -120(30) + 538.89 RA = 64.35(20 ¢) - 40(10) M B = 887 ft · k Maximum Moment @ B due to Wheel Loads. 4-60 Copyright © 2018 McGraw-Hill Education. 1 P2.5. Refer to Figure P2.4 for the floor plan. The baseplates at the bottoms of the columns are connected to the foundations at points A and D by bolts and may be assumed to act as pin supports.

Joint B and C are rigid. $\Phi -125'74.33'167.38'0.954P 0.025P -0.03P 0.043P \downarrow -150'87.5'175'1P 0 0 0 0$ Influence Lines: If A point load of 3 kips is applied @ B, Then the forces @ D and RXNS @ C are: C x = 0.292 (3k) = 1.5k N D = -0.579(3k) = 1.74 k Compr. If a point load P = 3 kips is applied at B, compute shear, axial load and moment at D. Analyze the frame in Figure P11.11 by moment distribution. The hollow structural section beam ABCD in Figure P5.50 is supported by a roller at point D and two links BE and CE. "A": 35 (2 m) = 14 kN C 5 FG = (35)2 + (14)2 = 37.7 kN FGY = JT. Draw the shear and moment curves and sketch the 2 deflected shape. $\Sigma Fy = 12.5 -10 -10 + Dy = 0$ Ax Ay Dy = 7.5 kips $\Sigma M A = -7.5(40) + 10(20) + 10(40) + Dx (30) = 0$ Dx = -10 kips $\neg Ax = 7.5$ kips 6-29 Copyright © 2018 McGraw-Hill Education. Comparison of pinned base frames for problems P13.20 and P13.21 generate the same moment at joints B of 75 ft-k in both approximate and exact methods. Case 1: The columns and girders of the rigid building frame in Figure P5.58 have been designed initially for vertical load as specified by the building code.

30 kips P8.19. FI = VCOLUMN MP 12.86 = 2.14 k (Exact 2.2) \approx COL \ddot{o} \div \acute{e} 6 cc L \div cè 2 \div \acute{e} 7 Second Floor Column Forces 2 F1 = F4 = 3(2.67) = 8k (Exact 9.8k) F2 = F3 = 1.5(2.67) = 4 k (Exact 0.96 k) M EXT = 15k (32 c) + 25k (20 c) + 25k (8c) = 1180 \text{ ft} \cdot \text{k} \text{ M INT} = 157.5 \text{ V3 A} = 1180 \text{ First Floor Column Forces V3 A} = 7.5 \text{ k} \text{ F1} = F4 = 3(7.5) = 22.5 \text{ k} (Exact 24.9 \text{ k}) F2 = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ M EXT} = 15k (32 c) + 25k (20 c) + 25k (20 c) + 25k (8c) = 1180 \text{ ft} \cdot \text{k} \text{ M INT} = 157.5 \text{ V3 A} = 1180 \text{ First Floor Column Forces V3 A} = 7.5 \text{ k} \text{ F1} = F4 = 3(7.5) = 22.5 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5(7.5) = 11.25 \text{ k} (Exact 0.96 \text{ k}) \text{ F2} = F3 = 1.5

2 EI æç165.75 ÷ö M AB = ÷ = 55.25 kN · m ç 6 çè EI ÷ø M BD (2) 10-24 Copyright © 2018 McGraw-Hill Education.

The estimated uniform dead load for structural steel framing, fireproofing, architectural features, floor finish, and ceiling tiles equals 24 psf, and for mechanical ducting, piping, and electrical systems equals 6 psf. If IBC = 8IAB, how would you adjust your assumptions of member end moments.

P6.28. Determine the reactions for the structure. E is constant and equals 30,000 kips/in. , IAC = 160 in. 15 kips 8 kips w = 0.5 kip/ft 25 kip • ft A B 10' C 10' D 10' P5.1 Σ M A = -10(8) - 25(0.5 \cdot 10) - 25 + 20C y = 0 C y = 34 kips Σ Fy = Ay - 8 - 0.5 \cdot 10 - 15 + 34 = 0 Ay = -6 kips a) Cut beam with origin at A Σ Fy = -6 - 8 - V = 0 V = -14 kips Σ M A = -M + (10) \cdot 8 + x (-14) = 0 M = 80 - 14 x kip \cdot ft b) Cut beam with origin at D Σ Fy = -0.5(10) -15 + V + 34 = 0 V = -14 kips Σ M D = M + (x2) \cdot (-14) + 10(34) - 0.5(10)(5) + 25 = 0 M = 14 x - 340 kip \cdot ft 5-2 Copyright © 2018 McGraw-Hill Education. Δ CO + δ CC RC = 0 0.00849 + .0000824 RC = 0 RC = 103.03 kN Δ *AY = FL 103.03 kN(4) = 0.00412 m or 4.12 mm 500 \cdot 200 AE *Since bar AC is vertical and the force is known, there is no need to consider other bars. 4 P9.8. Determine the reactions for the beam in Figure P9.8. When the uniform load is applied, the fixed support rotates clockwise 0.003 rad and support B settles 0.3 in. 40 kN P3.8. Determine the reactions at all supports and the force transmitted through the hinge at B. Knee brace force is in tension on column "KJI" 13-48 Copyright © 2018 McGraw-Hill Education. P13.11. P5.51. Continued B 2.4 kips/ft 10 kips 10 kips E 2EI -50 D C B -480 D C 2EI -120 6 kips E 2EI 2EI 6 kips -430 -58 EI EI MP -466 A B 1 kip E C D B C FP (kip-ft) 1 kip E (kips) A 1 D B E C D C 1 kip -10 E B D 1 kip -1 A -10 \delta EV = M Q = M P dx EI MQ, EV FQ, EV MQ, EH (kip-ft) (kips) (kip-ft) + A A FL 1 CM1 M3 L + FQ P EI AE (kips) A -12 1 1 1 58(12) 2 10 430 + 466 6 + (-10 + x)(-1.2 x + 48 x - 480) dx + 10(430) 6 + 2 EI EI EI EI 2 AE 10 0 = FQ, EH 17,000 25,800 26,880 696 61,180 696 (1728) + (12) = \delta EV = 20.30 in. Reaction Magnitude (kips) Ay = C y = 9.1 kips 14 13.2 12 By 11.8 10.4 10 8.4 9.1 9.2 8 Ay = Cy 6 4 0 1 2 3 Settlement of Support B (in.) 9-8 Copyright © 2018 McGraw-Hill Education. 0.4 kip/ft P3.9. Determine the reactions for the structure.

 $\Delta AO = \Sigma FQ^{2} \Delta LP = 2 \hat{A} \frac{4}{3} (T) 1 \text{ in}. 2 \text{ AE 5} = 2 3 \text{ D B 3 in}. 2 () \frac{4}{3} (T) 2 1 \Delta AO = 2.578 \text{ in}. EQ \text{ M AB} = 2 \text{ K} (\theta B - 3\Psi) - 2 \text{ K} \theta B + 1.2 \text{ K} \theta C - 6 \text{ K} \Psi \text{ M BC} = 2(.6)\text{K} (2\theta C + \theta B) + 19.2 = 2.4 \text{ K} \theta C + 1.2 \text{ K} \theta C + 1.2 \text{ K} \theta C + 1.2 \text{ K} \theta C - 3\Psi) = 4 \text{ K} \theta C - 6 \text{ K} \Psi \text{ M BC} = 2 \text{ K} (2\theta C - 3\Psi) = 4 \text{ K} \theta C - 6 \text{ K} \Psi \text{ M B} = 2 \text{ K} (2\theta C - 3\Psi) = 4 \text{ K} \theta C - 6 \text{ K} \Psi \text{ M BC} = 2 \text{ K} (2\theta C - 3\Psi) = 2 \text{ K} (2\theta C - 3\Psi) = 4 \text{ K} \theta C - 6 \text{ K} \Psi \text{ M BC} = 2(.6)\text{ K} (2\theta C + \theta B) + 19.2 \text{ E} (2.6)\text{ K} (2\theta C + \theta B) + 10.2 \text{ E} (2.6)\text{ K} (2\theta C + \theta B) + 10.2 \text{ E} (2.6)\text{ K} (2\theta C + \theta B) + 10.2 \text{ E} (2.6)\text{ K} (2\theta C + \theta B) + 10.2 \text{ E} (2.6)\text{ K} (2\theta C + \theta B) + 10.2 \text{ E} (2.6)\text{ K} (2\theta C + \theta B) + 10.2 \text{ E} (2.6)\text{ K} (2\theta C + \theta B) + 10.2 \text{ E} (2.6)\text{ K} (2\theta C + \theta B) + 10.2 \text{ E} (2.6)\text{ K} (2\theta C + \theta B)$

5 4 6 5 6 1 7 7 4 8 3 8 3 1 120' 9 115.2' 2 1 115.2' 100.8' 76.8' 43.2' 43.2' 20 19 18 17 16 15 14 13 12 20" 120' 10 100.8' 76.8' 43.2' 43.2' 41.01 = 20.51 kips Each Truss = 2 FAL = 24 k · 2 + 10 k · G 20' Influence Lines Max. 1 36 kips 36 kips

Draw the shear and moment curves for each member of the frame in Figure P5.44. B 40 kips 20' C 20' D E 15' P4.30 Entire Truss + $\Sigma Fx = 0$; + $\Sigma M E = 0$; + $\Sigma Fy = 0$; E x - 30 = 0 E x = 30 k 30 (40) + Dy (15) = 0 Dy = 80 k 80 - 40 - E y = 0 E y = 40 k Isolate Top Truss + $\Sigma M C = 0$; 40 (15) - Ax (20) = 0 Ax = 30 k 4-31 Copyright © 2018 McGraw-Hill Education. Continued 30 kips C 30 kips D (b) Cantilever Method FED at Section 1 M1 6 ft $\Sigma M Z = 0$ 1 POI POI Z V1 D F1 A1 Column Areas B E Axial Stresses Through Section 1 F2 A1 Z A 2 F V2 Centroidal Axis of Columns 0 = 30(6) - F1 (12.5) - F2 (12.5) F1 = A1\sigma1 = F2 = 2(12.5) F1 = F1 = F2 = 2(12.5) F1 = F1 = F2 = 2(12.5) F1 = F2 = 2(12.5) F1 = F3 = 33.6 kips 12.5 ft Column Areas B E Axial Stresses Through Section 2 $\Sigma M Z = 0 = 30(28) - F1 (12.5) - F2 (12.5) 840 F1 = F1 = F2 = 2(12.5) F1 = F3 = 33.6 kips 12.5 ft Column Areas B E Axial Stresses Through Section 2 <math>\Sigma M Z = 0 = 30(28) - F1 (12.5) - F2 (12.5) 840 F1 = F1 = F2 = 2(12.5) F1 = F3 = 33.6 kips 12.5 ft Column Areas B E Axial Stresses Through Section 2 <math>\Sigma M Z = 0 = 30(28) - F1 (12.5) - F2 (12.5) \Sigma M D V D V D = 7.2 kips S D (P0 = 7.2 kips S D P0) = 0 = M D = -7.2 VD C <math>\Sigma M D = 0 = M D = -7.2 VD =$

Case B: Both the 90-kN and 60-kN forces at joints D and M act. (a) Use approximate analysis to compute the reactions and draw moment diagram for the column AB and draw the approximate deflected shape of the frame. (Each cable transmits a dead load from the beams of 36 kips to the arch.) Determine the reactions, the axial force, shear, and moment at each joint of the arch, and the joint displacements. 8-45 Copyright © 2018 McGraw-Hill Education. 50 kN P11.21.

B C RA D RC 8m 6m P12.3 12-4 Copyright © 2018 McGraw-Hill Education. (c) Compute the maximum moment at point D. Compression in FFE Max. 3 kips/ft 1.6 kips/ft C A B 3 4 D 8' 4' P3.24 1 R1 = 12 '1.4 = 8.4 k 2 R2 = 1.6 '12 = 19.2 kips Consider Beam ABC: + $\Sigma M A = 0$; 19.2 '6 ¢ + 8.4 '8 - 8Fby = 0 Fby = 22.8 kips Fbx (Fby = 22.8) = 4 3 4 Fbx = (22.8) = 30.4 kips 3 + $\Sigma Fx = 0 = 30.4 - Ax \setminus Ax = 30.4 kips + \Sigma Fy = 0$; Ay + 22.8 -19.2 - 8.4 = 0 Ay = 4.8 kips 3-25 Copyright © 2018 McGraw-Hill Education. 60 kN P10.30. F E 12 kips 9' 30 kips H I D 9' 18 kips J C 16' A B 3 @ 12' = 36' EF, EI, ED, FH, and IJ P4.37 Freebody Joint "F ": $\Sigma Fy = 0$; FFH = 0 + Freebody Left of Section (2) to Compute Bar Forces in Bars EI&ED: + Freebody Left of Section (1) to Compute Force in $\Sigma M D = 0$; -30 k (36 ¢) -18k (24 ¢) -12 k (9 ¢) + FEI (12 ¢) = 0 FEI = 135k compression Bar "EF": + $\Sigma M I = 0$; 7.30 k (24 ¢) -12 k (9 ¢) + FEDY (12 ¢) = 0 FEI = 135k compression 4.38 Copyright © 2018 McGraw-Hill Education. C 0 -0.25 in. Using the Müller-Breslau principle, draw the influence lines for the reactions and internal forces noted. P6.32. The tension ring lies 12 ft below the compression ring. 30 kips -73.3 kip-ft 20.5 kips B 15 kips 15 kips 24 ft C 15 kips 73.3 kip-ft 20.5 kips 12 ft A 8.3 kips = Ax 15 kips = Ay 11-47 Copyright © 2018 McGraw-Hill Education. FCD = 18 kN comp. Reactions are given. 1 P12.57. w = 3 kips/ft P9.23.

But support E settles by 4 1 in. P9.27. Discuss the results of your study of P6.32 and P6.33 with particular emphasis on the magnitude of the forces and displacements produced by the 48-kip load. ú 2 EI ú M BC = $(2 \theta B + \theta C)$ ú 4 ú 2 EI ú MCB = $(2 \theta C + \theta B)$ ú 4 û (3) (4) EI (-2 (0.011767 + 0.005884) 2 200 $(25 (-0.019456) = -44.125 \text{ kN} \cdot \text{m} = 2 \text{ M BA}$

= $44.125 \text{ kN} \cdot \text{m} \text{ M} \text{ BC} = \text{M} \text{ AB} = \text{M} \text{ AB} = -77.94 \text{ kN}.\text{m} \text{ Analysis of members} + \Sigma \text{ M} \text{ B} = 0 = \text{C} \text{ y} 4 - 44.125 \text{ C} \text{ y} = 11.031 \text{ kN} + \Sigma \text{ M} \text{ A} = 0 44.125 + 100 '3 - 77.94 \text{ VBA} 6 = 0 \text{ VBA} = 44.364 \text{ kN} \neg \text{ Joint Equilibrium, EQs. Joint C 2 EI (2 <math>\theta \text{C} + \theta \text{B}$) 4 $\theta \theta \text{C} = -\text{ B 2 MCB} = 0 = + (2) \Sigma \text{Fx} = 0 \text{ -VBA} + 100 \text{ - VAB} = 0 \text{ VAB} = 100 \text{ - } 44.364 \text{ 55.636}$ kN \neg Reactions and Member end Moments 10-19 Copyright © 2018 McGraw-Hill Education. (f) hinge P3.33. P5.56. $\delta \text{Dx} = 0.172 \text{ in}$. 5-18 Copyright © 2018 McGraw-Hill Education. For the steel rigid frame in Figure P8.31, compute the rotation of joint B and the horizontal displacement of support C. Take the origin at joint B. Cx = 100 \text{ E P9.35}. Express your answer in terms of E, I, L, and M. 4 ft 1 P2.6. The uniformly distributed live load on the 2 floor plan in Figure P2.4 is 60 lb/ft . P14.8 is composed of a beam supported by two struts at the cantilever end.

 $C 5^{12} L 55 (a) Draw a qualitative influence line for the moment at the top of column AB in Figure P12.55. Analyze the frame in Figure P11.19 by moment distribution. 15 kN 30 kN 15 kN 1 f kN 1$

Compute the reactions at supports A and C produced by the wind load. 18 kips P5.6. Write the equations required to express the moment along the entire length of beam in Figure P5.6. Use an origin at point A, and then repeat computations using an origin at point D. A P12.56. + $\Sigma Fx = 0$; Ax - 6.25 = 0 Ax = 6.25 kips + $\Sigma Fy = 0$; Ay - 40 + 20 = 0 Ay = 20 kips + $\Sigma MA = 0$; 40 '10 - 20 '20 - MA = 0 kip + f3-26 Copyright © 2018 McGraw-Hill Education. 2.3 6 @ 6.67' = 40' C4 A B2 (a G2 B4 C3 G2 B1 C 5 @ 8' = 40' C3 40' C1 20' P2.4 5 ft (a) Method 1: AT 10 20 AT 200 ft B3 Method 2: AT 200 ft 2 B3 6.67 ft 6.67 ft 2 B4 (b) Method 1: AT 10 20 AT 201 ft B3 det 30 ft 2 ft 11 133.4 4 3.33 2 4 ft 2 AT 1112. ft 6.67 ft 2 B4 (b) Method 1: AT 10 20 ; 3.33 ft (c) Method 1: AT 40.33(10) AT 493.4 ft 33.33 ft AT, C3 2-6 Copyright © 2018 McGraw-Hill Education. A B L P9.19 Compute reactions using MB and RO as redundants. Draw the shear and moment curves for each member of the frame in Figure P5.29. For each beam, with given moment diagram: (a) sketch the deflected shape; and (b) determine, qualitatively, the applied loading. 4 -150 P14.11. 20 kips A 6' 10 kips B C 8' D 6' 6' P16.4 - PL -20(12) = -30 kip-ft 8 8 & 6A ù & 610 ù $\hat{e} + \hat{e} + \hat{e} - 30\hat{u} & \hat{e} + 2\hat{u} + 33 = 0.07899$ EI K 21 = 2 = 0.16667 EI K 12 = 2 = 0.16667 EI K 32 = 2 = 0.16667 1 0.16667 1 0.16667 1 0.16667 1 0.16667 1 0.16667 1 0.16667 1 0.16667 4 0.28333\hat{u} \hat{u} \hat{u} & \hat{e} + \hat{e} + \hat{e} + 334 + 2 = 0.16667 EI K 28 + 2 = 0.16667 EI K 28 + 2 = 0.16667 1 0.16667 4 0.28333\hat{u} \hat{u} & \hat{e} + \hat{e} + 3.333\hat{u} & \hat{e} + 3.333\hat{u} + 3.333\hat{u} + 3.333\hat{u} + 3.333\hat{u} + 3.333\hat{u} + 2.333\hat{u} + 3.333\hat{u} + 3.

(a) For the indeterminate beam shown in Figure P12.48, construct the influence lines for MA, RA, and RB by applying a unit load to the beam at 4-fit intervals to compute the corresponding magnitudes of the reactions. B C D E H G 5 @ 24' = 120' P12.16 Moving Load 1 kip B A Ay C 24 ft 24 ft E D Cy By Dy 24 ft 24 ft F Ey Fy 24 ft 0.6 0.4 0.2 VAB (kips) -0.2 28.8 3 F 19.2 14.4 9.6 MC (kip-ft) 19.2 14.4 9.6 MC (kip-ft) 12.17 Copyright © 2018 McGraw-Hill Education. A = 2 in 2, E = 30,000 ksi Arrows Directed from near to far ends of Members. A B 6' C 30' D 24' P13.7 V & M at D Since structure indeterminate to 2° , We must guess location of two P.I.'s RD = 15k + 11.68 = 26.68 M D = 11.68' 5 + 15' 2.5 = 95.9 kip · ft Comments: 1. 4-28 Copyright © 2018 McGraw-Hill Education. Case 3: Repeat the analysis in Case 1, 2 increasing the area of member 5 to 20 in. The 2 area of all bars is 5 in. 20 kN 20 kN A C B 3m 3m D 3m P9.18 $\theta = \theta E + \Delta\theta E - D 60 60 (3) (1.5) + EI EI 2 180 \theta = EI Use Minus Sign. Ay = 45.15 kips Deflection of B: By = 10 kips Reactions: Ay = 0.272(45.15) = Ay = 12.3 kips P = \Delta KTOT 15 = \Delta(55.15) By = 0.272(10) = By = 2.7 kips \Delta = 0.272 in.$

Investigate Joint I. C B 4m A 3m 4m P8.31 P-System E = 200 GPC I = 80 '106 mm 4 Compute DCx 1kN · DCx = $\hat{\Sigma}$ ò MQ M P dx EI 4 DCx = \hat{O} (0.571x) 0 4 dy EI 5 dx dy + \hat{O} 9.404 x 2 EI EI 0 0 313.408 391.833 = + EI EI 705.241 kN 2 · m 3 = kN m4 200 '106 2 '80 '106 mm 4 2 m 10 mm 4 DCx = 0.0441 mm = \hat{O} 14.601 x 2 1kN · m θ B = $\hat{\Sigma}$ ò MQ M P Compute DB dx EI 4 θ B = \hat{O} (0.129 x) (25.71x) 0 dx EI 5 + \hat{O} (0.4571 x) (20.571 x) 0 d x EI 5 + \hat{O} (0.057 x) (20.571 x) 0 d x EI 5 + \hat{O} (0.057 x) (20.571 x) 0 d x EI 5 + \hat{O} (0.0571 x) 0 d x EI 5 + \hat{O} (0.057 x) (20.571 x) 0 d x EI 5 + \hat{O} (0.0571 x) 0 d x EI 5 + \hat{O} (0.057 x) (20.571 x) 0 d x EI 5 + \hat{O} (0.057 x) (20.571 x) 0 d x EI 5 + \hat{O} (0.057 x) (20.571 x) 0 d x EI 5 + \hat{O} (0.057 x) (20.571 x) 0 d x EI 5 + \hat{O} (0.0571 x) 0 d x EI 5 + \hat{O} (0.057 x) (20.571 x) 0 d x EI 5 + \hat{O} (0.0571 x)

P12.49. 0.002 rad P10.10. 20 kips P5.47.

For all 2 2 bars the area = 2 in. The bent frame BCDE in figure P3.18 is laterally braced by member AC, which acts like a link. Gilbert SOLUTIONS MANUAL CHAPTER 12: INFLUENCE LINES FOR MOVING LOADS 12-1 Copyright © 2018 McGraw-Hill Education. Recompute the reactions for the frame in Figure P9.43 if support C settles 0.36 in. P8.38. The designer could also investigate the shape of the arched truss. The three bay, one-story frame consists of beams pin connected to columns and column bases pinned to the foundation in Figure P3.36. Evaluate the deflections. Determine all reactions and draw the shear and moment curves. F w = 10 kN/m P12.22. The diagonal cables have two effects.

As part of the design, the building frame must be checked for lateral deflection under the 0.8 kip/ft wind load to ensure that lateral displacement will not damage the exterior walls attached to the structural frame. + $\Sigma M D = 0$; - 20 k (15¢) + FABY (30 ¢) = 0 FABY = 10 compression $\approx 15 \text{ ö FABX} = \varsigma c \div FABY = 15 \text{ k c} c^{10} \div \emptyset \text{ k Freebody Left of Section}$ Section (2): $\Sigma Fy = 0$; + FDJY - 20 k tension $\approx 15 \text{ ö FDJX} = \varsigma c \div FDJY = 15 \text{ k c} c^{20} \emptyset FAB = (10)^2 + (15)^2 = 18 \text{ k compr. P6.31}$.

E P4.15.

4m 3m B C 3m F 4m 60 kN E 3m A D P4.19 Joint C Reactions: $+\Sigma M A = 0$; 60'3 - Dy 10 = 0 FBC C Dy = 18 kN FFC + 18 $\Sigma Fy = 0$; Ay - 60 + 18 = 0 FCD 42 Ay = 42 kN + $\Sigma Fx = 0$; FBC = 42 kN comp. B P6.7. The cables in Figure P6.7 have been dimensioned so that a 3-kip tension force develops in each vertical strand when the main cables are tensioned. to the left of column BD. 5 in 2 5 in 2 20' 4 in 2 D 2 in 2 A C P = 40 kips 15' 15' P8.14 Q-System P-System L A æ ö cc \Sigma F F L ÷ 12 in /pt. 80 kN P15.1. Using the stiffness method, write and solve the equations of equilibrium required to determine the horizontal and vertical components of deflection at joint 1 in Figure P15.1. For all bars E = 200 GPa and 2 A = 800 mm.

Inversely Proportional to L FL AE AE (Δ L) F= L FAB 12 = FBC 18 12 \ FAB = FBC 18 12 \ FAB = FBC 18 Δ L = Girder ABC FAB + FBC = 1.71 2 F + FBC = 1.71 3 BC FBC = 1.02 kips FAB = .69 Column BD Reactions 11-33 Copyright © 2018 McGraw-Hill Education. 50 kN 70 kN 20 kN 90 kN 30 kN y1 A y3 y2 y4 y5 hz max B 6 @ 10 m = 60 m P6.18 Σ M A = 30(10) + 20(20) + 50(30) + 70(40) + 90(50) - E y (60) = 0 E y = 158.33 kips Ay = 101.67 kips 20 kN 50 kN 70 kN 30 kN Assuming Funicular shaped arch supporting a set of point loads, use the General Cable Theorem and maximum height of 20 m: 90 kN y2 y1 Hhmax = M max y3 y4 y5 Ay H (20) = 2266.8 By 30 kN 20 kN 50 kN 70 kN 90 kN H = 113.3 kN y4 = 20 m y2 = 6 @ 10 m = 60 m 1733.4 113.3 y2 = 15.30 m y3 = 2250.1 y5 = 1583.5 113.3 y1 = 19.86 m 101.67 71.67 51.67 1.67 Shear (kN) -68.33 -158.33 113.3 y5 = 13.98 m 1733.4 10167 2250.1 2266.8 1583.5 Moment (kN-m) 6-20 Copyright © 2018 McGraw-Hill Education.

P5.58. D 12 kips 12 kips 6' E C 12 kips 12 kips 6' E C 12 kips 12 kips 16' F B J H 6' G A 4 @ 8' = 32' P4.47 All Forces in kips Joint A + ΣFx = 0: X AJ = 0 \ YAJ = FAJ = 0 30 - FAB = 0 ΣFy = 0: \ FAB = 30 (C) Joint B 4 X BC = YBC = 24 k (C) 3 5 FBC = YBC = 24 k (C) 3 5 FBC = YBC = 24 k (C) 3 5 FBC = YBC = 24 k (C) Joint J 5 FCD = (YCD) = 40 k 3 5 FJI = YJI = 30 k (T) 3 XCD = 4 (Y) = 32 k 3 CD + ΣFx = 0 = 24 - 32 + FCI FCI = 8k (T) Joint I 4-48 Copyright © 2018 McGraw-Hill Education. D 3m 2m A 2m B 45° C 2 kN/m P F 4m 4m 4m P6.12 Use General Cable Theory: H · h = M = 16 kN · m at "C" H = 16 = 8 k. M A = MO = M Redundant. B P4.11. 33.6 kips P13.17. w = 12 kips/ft

P5.17. Clamp Joint 2 wL2 $6(12)2 = -72 \text{ kN} \cdot \text{m} 12 12 \text{ FEM23} = 72 \text{ kN} \cdot \text{m} \text{ FEM23} = -30 \text{ int} 2 \text{ Moments Produced by A 1 Radian Rotation of Joint 2, Multiplied by <math>\theta 2 \text{ M12JD} = 2 \text{ EI } (2[-1]) = 12 3 \text{ M32JD} = 2 \text{ E} (2 \text{ I})(-1) \text{ EI} = 12 3 \text{ D } 2 \text{ K } 2 = \text{M21JD} + \text{M 23JD} = -\text{EI} \cdot \text{EI} 3 5 \text{ K } 2 = -\text{EI} 3 14-9$ Copyright © 2018 McGraw-Hill Education. 12' P8.13. 4' P12.6. For the beam in Figure P12.6, draw the influence lines for reactions at B, C, E, and G, and moments at C and E. 10' 10' 40 kips B C 15' 20' D A P10.32 Unknowns: θB , θC , θD , $\Delta \Delta 20 \Delta \Psi CD = \Psi DC = 15 40(20) \text{ PL} = -100 \text{ k-ft FEM BC} = 8 8 \text{ FEMCB} = 100 \text{ k-ft } \Psi AB = \Psi BA = \text{Member end}$ Moments 2 EI æç $\Delta \ddot{o} \text{ gc}_2 \theta B - 3 \div \div 20 \text{ e} 20 \text{ ø} 2 \text{ EI M BC} = (2\theta B + \theta C) - 100 20 2 \text{ EI MCB} = (2\theta C + \theta B) + 100 20 2 \text{ EI æç } \Delta \ddot{o} \text{ MCD} = \text{gc}_2 \theta C + \theta D - 3 \div \div 15 \text{ e} 15 \text{ ø} M BA = \text{Equil. w1} = 2 \text{ kips/ft B } 13' \text{ w3} = 8 \text{ kips/ft D C } 13' 13' \text{ P5.19} + \Sigma M B = 0; -26(6.5 \text{ c}) + 65(6.5 \text{ c}) + 104(19.5 \text{ c}) - Dy (26 \text{ c}) = 0 \text{ Dy} = 87.5 \text{ k} \Sigma \text{ Fy} = 0; + -26 \text{ k} + \text{By} - 65 \text{ k} - 104 \text{ k} - \text{Dy} = 0 \text{ By} = 107.25 \text{ k} 5-21 \text{ Copyright © 2018 McGraw-Hill Education. P = 6 kips P8.23. P = 16 kips P11.22.}$

Draw the shear and moment curves for each member of the rigid frame in Figure P3.4. (NG) (c) Case 2 Unbraced Frame with Shear Joints 7-55 Copyright © 2018 MGCraw-Hill Education. E = 3000 kips/in. 42 Given: IAC = 340 in. (b) To restore joint B to its initial position in the horizontal direction, how much must par AB be shortened? (b) Repeat the analysis of the archi if a single 48-kip vertical load at s downward at joint 18. Also E is constant. Determine all reactions and draw the shear and moment curves for the beam in Figure P9.16. 13-3 Copyright © 2018 MGCraw-Hill Education. (a) Analyze the structure for full liver of curves for the girders, she forces in the cables, and the maximum deflection of the girders. See Figure P10.2. M A Y LA M A 1 La M ô 2 L 1 L M eç L 1 L $\delta + = '$ (c + + ' + ' (e + + 2 + 2 kig 2 d = 2 3 $2 + \phi$ ML 2 M L 2 L d = 2 4 M L 2 M L 2 4 = 2 (AM X) A - 2 (AM X) B L L 2 4 ML 2 $\delta + 2$ 2 (d = 2 3 $2 + \phi$ the d = 2 (d = 2 d = 2 (d = 2 d = 2 (d = 2 d = 2 d = 2 (d = 2 d =

Continued Segment "CD": + Σ MC = 0; 3(30)2 - VDC (30) = 0 2 = 45k - 236.8 + 236.8 + VDC + Σ Fy = 0; VCD + 45 - 3k/ft (30 ¢) = 0 VCD = 45k Segment "CH": + Σ M H = 0; - 23.7 -11.8 + VCH (20 ¢) = 0 VCH = 1.775k VHC = 1.775 ¬ Hx Joint "B": + Σ Fy = 0; FBA = 40.26 Ay = 40.26 Ay = 40.26 Joint "C": + Σ Fy = 0; FCH = 94.7k H y = 94.7k 11-27 Copyright © 2018 McGraw-Hill Education. P = 38 kN P13.9. The cross sections of the columns and girder of the frame in Figure P13.9 are identical. A P12.50.

 $C3^{40}$ 20' Building Section C1 P2.8 & 40 20 \ddot{o} + 2 \dot{c} + 2 + \dot{s} + 4 20 = 600 ft, K LL = 4, AT K LL = 2400 > 400 \approx 15 \ddot{o} + 60, \ddot{o} k L = 60 c c0.25 + = 33.4 psf = + + \dot{c} è 2 2400 g (a) AT = cc P3rd = 600(33.4) = 20040 lbs = 20.1 kips AT, C4 B4 AT, C3 PLAN P3rd P1st C3 ELEVATION 2-10 Copyright © 2018 McGraw-Hill Education. The truss, bolted to the left abutment at point A, may be treated as pin supported. Case 3 Braced Frame with Shear Joints (a) Moment Diagram (khjs) 7-57 Copyright © 2018 McGraw-Hill Education. So consider clusters 1; \dot{c} + 578.70 kip; \dot{n} FEM AB Case B: Sidesuiga 2 E1 1 (e) (-347) where $\Psi = L L 2$ E120 \dot{c} (1 \dot{s} + 578.70 kip; \dot{n} FEM AB Case J 31 in L 2 450 C P-System V = 0218 McGraw-Hill Education. 1 A 45° A B B1 1 45° 10° slab A B 2 14' B2 45° C B1 18' 10° slab Section 1-1 P5.55 BEAM 1 BEAM 2 5-59 Copyright © 2018 McGraw-Hill Education. So (2 \dot{n} P = 1 \dot{n} CC 6C P-System Σ FQ = 2 KP (4 \dot{n} (3) 2002) (5/3) 25/2) = + A E A E 12.11 138.89 210 = + a E A E A E 312.5 210 P + A E A E A E 312.5 210 P + A E A E A E 312.5 210 P + A E A E A E 312.5 20 P + A E

 $0.4 \text{ kip/f B t C 10' 10 kips 16' 10' D A 20' P3.21 Take Member AB as a free-body and sum moment about B: + \Sigma M B = 0; -10 '10 ¢ + Ax (20 ¢) = 0 Ax = 5 kips ¬ + \Sigma Fx = 0 of entire structure: 10 + 8.16(sin <math>\theta$) - 5 - Dx = 0 Dx = 10 + 1.6 - 5 = 6.6 kips ¬ Take BCD as a free-body and sum moment about B: + $\Sigma M B = 0; -10 '10 ¢ + Ax (20 ¢) = 0 Ax = 5 kips ¬ + \Sigma Fx = 0 of entire structure: 10 + 8.16(sin <math>\theta$) - 5 - Dx = 0 Dx = 10 + 1.6 - 5 = 6.6 kips ¬ Take BCD as a free-body and sum moment about B: + $\Sigma M B = 0; -10 '10 ¢ + Ax (20 ¢) = 0 Ax = 5 kips ¬ + \Sigma Fx = 0 of entire structure: 10 + 8.16(sin <math>\theta$) - 5 - Dx = 0 Dx = 10 + 1.6 - 5 = 6.6 kips ¬ Take BCD as a free-body and sum moment about B: + $\Sigma M B = 0; -8.16 k '10.2 ¢ + 6.6 k '20 ¢ - Dy$ (20) = 0 Dy = 2.44 kips $\Sigma Fy = 0$ of entire structure: 8.16 (cos θ) + 2.44 - Ay = 0 Ay = 8.00 + 2.44 = 10.44 kips 3-22 Copyright © 2018 McGraw-Hill Education. 80 kips P8.15. Continued (b) Cantilever Method P.I. occur @ midspan in beams & columns.

Assuming the container is water-tight, will the tsunami wave be capable of carrying away the container as debris? P = 6 kips w = 1 kip/ft A B 6' C 6' P8.20 Compute SC Px + $vx \cdot x = MP 2 x = MA L Q \cdot 6C = 0$ $Q \cdot e^{-1} EI = 2 Wx 3 \ddot{o} + \dot{c} \dot{c} \dot{c} \dot{c} \dot{c} + 2 + \phi + dx L 0 \acute{e} Px 3 Wx 4 \dot{u} \dot{u} + \delta C = \hat{e}$ $\hat{e}^{2} \tilde{a} EI 8EI \dot{u} \tilde{u} 0 L 6C = PL3 WL4 + 3EI 8EI (1) Compute I MIN. 1F SC £ 0.4 in 0.4 = 6(12)31728 1(12)4 1728 + 3(4000) I 8(4000) I MIN. - 1 0 A E 0 0.75 D 0.75 D n.75 D n.75 D n.75 D 0.75 D 0.75 D n.75 D 0.75 D n.75 D 0.75 D n.75 D 0.75 D$

Consider any live load reduction if permitted by the ASCE standard. Round the area to the nearest whole number. Line for Moment at Mid-Span Σ M about "O" 10 k (10 ¢ + 16 ¢ 6 k = 8.17 ft 24 M £ = 8k (25) + 10 k (7.5) + 6 k (4.5) = 122 kip · ft æ15.917¢ ö ÷ k Mmax = çç ÷ - 8 ´10 ¢ çè 12.73k ÷ø = 122.6 kip · ft 12-47 Copyright © 2018 McGraw-Hill Education. Analyze the frame in Figure P10.18. P8.8. When the truss in Figure P8.8 is loaded, the support at E displaces 0.6 in. EI = 36,000 kip · ft for columns. E A P4.29. Compute Axial Forces in Bars Bar AB MQ does not have to be consideral, MP = 0 Q-System FQ = 0.6 cos 52 + 0.5(sin 52) = 0.6(.62) + 0.5(.79) = 0.370 + 0.39 = -0.763 kips Bar BC FQ = 0.6 cos 21 + 0.5sin 21 FQ = 0.6(.933) + 0.5(.358) = 0.56 + 0.179 = 0.739 K 1k · \delta Cy = \Sigma FQ FP L AE é (-0.767)(-63.41)48.78'12 ù ú \delta Cy = 2 ê êë úû 20 ´ 39,000 + 2 [0.739 x - 4(.79)32.14 ´12] 20 ´30,000 \delta Cy = 0.134 in 8-18 Copyright © 2018 McGraw-Hill Education. E P4.48.

Assume rocker at support D acts as a roller. F 4m A P12.16. Members 2 AB, BC, and CD have area = 1 in. Establish the loading for members (a) floor beam B3, (b) floor beam B4, (c) girder G3, and girder G4. (a) (b) hinge (c) hinge (d) hinge (e) (f) P5.54 5-57 Copyright © 2018 McGraw-Hill Education. w = 3 kips/ft P11.16. Draw the shear and moment curves for each member of the frame in Figure P5.23. Compute the vertical displacement of joints B and C for the frame shown in Figure 4 2 P8.30. horizontally and connected to the pin support 4. Compute the horizontal displacement of joint B. = 0.444 9I 1.000 Σ K S¢ = 80 FEM AB = Case A: Sidesway Prevented Case B: Sidesway Correction Compute Holding Force S @ Joint B. For example, building codes typically specify maximum permitted displacements to ensure that excessive cracking of attached construction, such as exterior walls and windows, does not occur (Photo 1.1 in Section 1.3). Stone concrete has a unit 3 weight of 150 lb/ft . for all bars. 10' 20 kips B C 12' A D P11.19 PL 20(20) == -50 k \cdot ft 8 8 = 50 k \cdot ft 1 1æ I ö = , K BC = K AD = cc ÷ + 12 2 cè 20 ÷ ø FEM BC = FEMCB K AB DFBA = 10 3 , DFBC = 13 13 11-24 Copyright © 2018 McGraw-Hill Education. (c) P10.28. The forces in members of determinate trusses are not affected by member stiffness. (b) Draw the qualitative influence line for the moment at B. Consider column bases are fixed. I 35 kips 30 kips 35 kips hinge H G D h=? , and members 5 2 2 and 6, 2 in. Determine the reactions at A and C in Figure P9.39.

Curvature is also indicated. $\hat{e}\Delta 3$ ú \hat{e} 0.3451ú \ddot{e} û \hat{e} 0.2451ú \ddot{e} û \hat{e} 0.2451) = Q2 = -7.72 kips Q4 = -144(0.2915) - 48(0.3451) = Q4 = -58.54 kips Q5 = -200(0.2915) - 144(0.3451) = Q5 = -108 kips \neg Q6 = 0(0.2915) + 192(0.3451) = Q6 = 66.26 kips \Delta 3 = 0.3451 in.

MACBL/2L/2P7.5 + Σ MA = 0; - M + Cy(L) = 0 M LMAy = LCy = FBD 0£x 3/8 in. 6 kN P4.50. Continued Compute Slope at B: Slope at B: Slope at B: $\theta = dy 3x = ; x = 10 \notin dx 40 dy 3 = dx 4$ Since force at B has the same slope as the Arch: It is an Axial Force = 25 kips = Fb = (20 k) 2 + (15 x) 2 Conclusion: Arch is stressed by an Axial Force. Gilbert SOLUTIONS MANUAL CHAPTER 14: INTRODUCTION TO THE GENERAL STIFFNESS METHOD 14-1 Copyright © 2018 McGraw-Hill Education. 2 1 1 60 kips 30° 2 3 3 4 12' 12' P15.2 Compute Δx and Δy for joint 1, all bars: $\Delta = 3$ in 2, L = 20 ft., E = 30,000 ksi Bar 1 cos fx = 1; sin fx = 0 Unit horizontal displacement of joint 1 3 4 Bar 2 cos fx = , sin fx = 5 5 3 4 Bar 3 cos fx = -; sin fx = 5 5 Equilibrium Equations $\Sigma Fx = 0; K11 D x + K12 D y = 51.96 (1)$ $\Sigma Fy = 0; K21 D x + K22 D y = 30 (2)$ Unit vertical displacement of joint 1 Evaluate Stiffness Cofficients; Using Figure 15.2a and Figure 15.2b in Text.

(a) Calculate the reactions at support A. w = $3.5 \text{ kips/ft} \text{ A B } 12' 12' \text{ P11.1 Joint B Distribution Factors I 4I} = 20 80 \text{ I 5I} = = 16 80 9I \Sigma \text{K S} = 80 495 = 9 \text{ K AB} = \text{DFAB} = \text{K BC DFBC FEMS PL } 32' 20 = = 80 \text{ kip} \cdot \text{ft} 882 \text{ wL2 } 3(16) = = 64 \text{ kip} \cdot \text{ft} 12 12 \text{ FEM AB} = \text{FEM BC 11-2 Copyright } \text{Copyright } \text$ foot: æ lb ö 10 in 10" slab: $c_150 3 \div (16 \text{ ft})$ ce ft $\div 0 12$ in/ft 60' CL curb 3' = 480 lb/ft 2 rails: 120(2) = 240 lbft 10' 3' section A-A P3.34 R Each Girder 1' 60' 1' = 2.72 kip/ft 2.72 kips/ft 2 Girder self-weight: 400 curb w 240(2) Dead weight: CL = 2,000 lb/ft 2 curbs: Total: A girder wL 2 R = 1.36 kip/ft = 0.40 kip/ft Total Dead Weight: = 1.76 kip/ft Live Load 700 lb/ft (1.33) 2 Total Girder Load (D + L) per foot: Live x Impact Factor: = 0.47 kip/ft = 1.76 + 0.47 = 2.23 kip/ft = 1.76 + 0.47 = 2.2 $20 \text{ wL2 } 6(6)2 \text{ FEM AC} = = -7.2 \text{ kN} \cdot \text{m} 30 30 (8) \text{ PL FEMCD} = -50 = -50 \text{ kN} \cdot \text{m} 88 \text{ PL FEM DC} = -50 \text{ kN} \cdot \text{m} 88 \text{ PL FEM DC} = -37.5 \text{ kN} \cdot \text{m} 88 \text{ PL FEM DC} = -37.5 \text{ kN} \cdot \text{m} 88 \text{ PL FEM DC} = -37.5 \text{ kN} \cdot \text{m} 88 \text{ PL FEM DC} = -30 \text{ kN} \cdot \text{m} 88 \text{ PL FEM DC} = -37.5 \text{ kN} \cdot \text{m} 88 \text{ PL FEM DC} = -37.5 \text{ kN} \cdot \text{m} 88 \text{ PL FEM DC} = -37.5 \text{ kN} \cdot \text{m} 88 \text{ PL FEM DC} = -37.5 \text{ kN} \cdot \text{m} 88 \text{ PL FEM DC} = -37.5 \text{ kN} \cdot \text{m} 88 \text{ PL FEM DC} = -37.5 \text{ kN} \cdot \text{m} 88 \text{ PL FEM DC} = -37.5 \text{ kN} \cdot \text{m} 88 \text{ PL FEM DC} = -37.5 \text{ kN} \cdot \text{m} 88 \text{ PL FEM DC} = -37.5 \text{ kN} \cdot \text{m} 88 \text{ PL FEM DC} = -37.5 \text{ kN} \cdot \text{m} 88 \text{ PL FEM DC} = -30 \text{ kN} \cdot \text{m} 88 \text{ PL FEM DC} = -37.5 \text{ kN} \cdot \text{m} 88 \text{ PL FEM DC} = -30 \text{ kN} \cdot \text{m} 88 \text{ PL FEM DC} = -37.5 \text{ kN} \cdot \text{m} 8$ $+ 50 + (2 \theta D) 8 6 2 EI (2 \theta D) - 37.5 = 0 + 6 36.116$ Solving EQ's (1) and (2) $\theta c = EI (1) (2) \theta D = 11.74 EI$ Substitue values of θC and θD into EQ's for member and moments M AC = 4.84 kN · m MCA = 34.88 kN · m MCA = 34.88 kN · m $\theta A = \theta B = \theta E = 0$ M DB = -7.83 kN · m 2 EI M BD = -3.92 kN · m M AC = (θC) - 7.2 6 Analysis of Members 2 EI MCA = $(2 \theta C) + 10.8 6 2 EI M DE = (2 \theta D) - 50 8 2 EI M DE = (2 \theta D) - 50 8 2 EI M DE = (2 \theta D) - 37.5 6$ A P5.45. (b) Compute the change in length of member DE required to displace point B upward 0.75 in.) L (ft.) AL (in. (a) A two-story hospital facility shown in Figure P2.18 is being designed in New York with a basic wind speed of 90 mi/h and wind exposure D. A B 9' 4 kips/ft C 6' 6' P11.7 PL 8 = 45 FEM BC = FEMCD = EI 4 EI = 9 36 EI 3EI = K BC = 12 36 7 EI $\Sigma K S = 36 EI K BC = 12 3 EI K CD = 492 EI \Sigma K S = 12 K AB = wL2 = 27 12 473 = 7 DFBA = DFBC 121 = 2 DFBC = DFCD 11-8 Copyright © 2018 McGraw-Hill Education., E = 30,000 kips/in. P9.32. (b) Using RISA-2D, compare the 4 result if I = 640 in. (b) Compute the deflection under the load. If the maximum stress is not to$ exceed 25 kips/in.2, can the lower chord support the 4-kip load safely in addition to the three-hinged, parabolic arch ABC, shown in Figure P12.24. Determine the governing load combination for both negative and positive moments at the ends and mid-span of the beam. 2m 3m B 3m 40 kN x 20 m D 2m 4m C 10 m 10 m P6.14 Using the General Cable Theorem: Ay $\Sigma M A = PB(20) + PC(30) - Dy = 0 Dy = PC PB 1 3 PB + PC 2 4 20 m 1 3 \Sigma Fy = Ay - PB - PC + PB + PC 2 4 1 1 HhB = MB; H(3) = PB + PC 2 0 2 4 1 3 HhC = MC; H(2) = PB + PC 10 2 4 ((10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m (12 P + 14 P) 203 = (10 m MB 20 m)) B C 10 m PC PB Ay MC PC Dy 10 m$ 12 P + 43 P)102 B Dy Cable (Beam representation) C PB = 2.5PC Equilibrium of the beam suspended from points B and C: 40 kN $\Sigma Fy = 2.5PC + PC - 40 = 0$ Beam PC = 11.43 kN x = 2.86 m T = Dy2 + H 2) $\Sigma M A = 40(20 + 2.86) - 4 H - 40$ Dy 1 1 20 H = (2.5)11.43 + 11.43 2 4 3 H = 114.3 kN Dy = 11.43 kN Ay = 28.57 kN 2m T = 117.9 kN 3m H 4m 3m Ay T = 11.432 + 114.32 10-x x ΣMB = 40 x -11.43(10) = 0 (PC PB PB H 2m PC 6-15 Copyright © 2018 McGraw-Hill Education. P16.5. Continued Solve for Rotations $\theta 1$ and $\theta 2$; $F = K \cdot \theta + 120\dot{u} + 0.733 0.2\dot{u} + 0.120\dot{u} + 0.20.4 \dot{u} + 0.20.4 \dot{u} + 0.20.2 0.4 \dot{u} + 0.20.2 0.4 \dot{u} + 0.20.2 \dot{u} +$ $56.9 \text{ ft} \cdot \text{k} \notin + \text{MCB} \notin e = 0 + (0.2 \ \theta 1 + 0.4 \ \theta 2)$ EI MCB = MCB = 0 Shear and Moment Diagrams Beam AB: + $\Sigma \text{M} \text{ B} = 0$; -80 k + 47.9k + $\nabla \text{Fy} = 0$; -80 k + 47.9k + 28.9k + Education. Member AB BC CD BF CF CE AF FE ED +25 -50 -25 +40 A 20' 0 +30 +15 F +15 D E 60 kips 40 kips 20 kips 15' Original Truss 2 C -30 15' 15' (a) 3 2 (b) 3 2 A (in. 4-29 Copyright © 2018 McGraw-Hill Education.

The analysis is to include evaluating the support reactions and drawing the moment curves for column AB and girder BC. Gilbert SOLUTIONS MANUAL CHAPTER 7: DEFLECTIONS OF BEAMS AND FRAMES 7-1 Copyright © 2018 McGraw-Hill Education. P8.7. Determine the value of the force P that must be applied to joint C of the truss in Figure P8.7 if the vertical deflection at C is to be zero.

E C P4.18.

A B C 16' 4' P10.9 Ψ AB = 0.5 1 + Δ rad.

(c) Compare the results with an exact analysis using a computer software. K2y P14.3. The pin-connected bar system in Figure P14.3 is stretched 1 in. 60 kips w = 4 kips/ft P8.37. See Figure P10.1. P A FEMAB B L 4 L 2 L 4 P10.1 Compute FEM's 1 PL L PL L (2) + 2 4 4 4 2 3 = PL2 16 AM = FEM AB = 2(AM X) A FEM AB = 3PL 3PL 3PL = 16 8 16 2 - 4(AM X) B L L 2 2 & ö 2 3PL L ÷ 4 & c 3PL 2 ö ÷ L ÷ = 2 cc 2 ø ÷ L è c 16 Similarly from Symmetry FEM BA = + 3PL 16 10-2 Copyright © 2018 McGraw-Hill Education. (a) Draw the influence lines for the reactions at B and E, the shear between CD, the moment at B and D for the girder HG in Figure P12.18. A B 3m 8m 4m 4m 8m P6.10 Due to symmetry: w 8(w) Ay = By = = 4w 2 8' Max tension is equal to horizontal thrust at 1/2 span. Joint E acts as a hinge. (a) Use approximate deflected shape of the frame. Determine the uniform dead load in kips per linear values of the reactions 40 kips P5.50. hinge hinge (a) (b) (c) hinge hinge (d) (e) P3.33 (a) Indeterminate 3° (c) Unstable (R < 3 + C) (d) Unstable (e) Indeterminate 3° (f) Indeterminate 3° (f) Indeterminate 3° (f) Indeterminate 3° (c) Unstable (R < 3 + C) (d) Unstable using Moment Distribution $3(6)^2 = 54 \text{ ft} \cdot \text{k} 2 \text{ wL2}$ FEM BC = -FEMCB = $225 \text{ ft} \cdot \text{k} 12 3(24)^2$ FEMCD = -FEM DC = -FEM DC = -FEM DC = $-144 \text{ ft} \cdot \text{k} 12 3 \approx 1 \text{ o}$ KCB = $c_c \div = 0.025 \text{ DFCB} = 0.375 4 \text{ c}$ $30 \div 9 \text{ FEM AB} = KCB = + \Sigma M B = 0$; I = $0.0417 24 0.0667 \text{ DFCD} = 0.626 1 - 54 + 3(30 \circ)(15 \circ) + 248.1 - VCB (30) = 0 \text{ VCB} = 51.5 \text{ k} 13-11 \text{ Copyright} \otimes 2018 \text{ McGraw-line}$ M P8.18., are used to support the deck at the third points of each 120 ft span. Load Case 2: 2 3 IN Flow 30' A E I B F J G K H L 16' OUT Flow 16' 3' C 16' OPEN D P2.22 Load Case 3: hmax = hdes = 20 ft/sec 3 ft K 6 ft Hydrodynamic, Load Case 2: Hydrodynamic, Load Case 3: hmax = hdes = 20 ft/sec 3 ft K 6 ft Hydrodynamic, Load Case 3: hmax = hdes = 20 ft/sec 3 ft Hydrodynamic, Load Case 3: hmax = hdes = 20 ft/sec 3 ft Hydrodynamic, Load Case 3: hmax = hdes = 20 ft/sec 3 ft Hydrodynamic, Load Case 3: hmax = hdes = 20 ft/sec 3 ft Hydrodynamic, Load Case 3: hmax = hdes = 20 ft/sec 3 ft Hydrodynam ft hdes, K = Trib height = 8 + 6 = 14 ft hdes, J = Trib height = 1 + 8 = 9 ft 8 ft FdK = FdK = 1 2 1 2 FdJ = y s (I tsu)(C d)(C cx)(B)(hdes, K)(20) 2 1 2 70.4(1.0)(1.25)(1.0)(35)(9)(6.67) 2 FdJ = 616 kips 70.4(1.0)(1.25)(1.0)(35)(14)(20) 2 hdes, K = Trib height = 8 + 8 = 16 ft FdK = 8624 kips 1 2 FdK = 70.4(1.0)(1.25)(1.0)(35)(16)(6.67) 2 FdJ = 616 kips 70.4(1.0)(1.25)(1.0)(35)(9)(6.67) 2 FdJ = 616 kips 70.4(1.0)(1.25)(1.0)(35)(14)(20) 2 hdes, K = Trib height = 8 + 8 = 16 ft FdK = 8624 kips 1 2 FdK = 70.4(1.0)(1.25)(1.0)(35)(16)(6.67) 2 FdJ = 616 kips 70.4(1.0)(1.25)(1.0)(35)(14)(20) 2 hdes, K = Trib height = 8 + 8 = 16 ft FdK = 8624 kips 1 2 FdK = 70.4(1.0)(1.25)(1.0)(35)(16)(6.67) 2 FdJ = 616 kips 70.4(1.0)(1.25)(1.0)(35)(14)(20) 2 hdes, K = Trib height = 8 + 8 = 16 ft FdK = 8624 kips 1 2 FdK = 70.4(1.0)(1.25)(1.0)(35)(16)(6.67) 2 FdJ = 616 kips 70.4(1.0)(1.25)(1.0)(35)(14)(20) 2 hdes, K = Trib height = 8 + 8 = 16 ft FdK = 8624 kips 1 2 FdK = 70.4(1.0)(1.25)(1.0)(35)(16)(6.67) 2 FdJ = 616 kips 70.4(1.0)(1.25)(1.0)(35)(14)(20) 2 hdes, K = Trib height = 8 + 8 = 16 ft FdK = 8624 kips 1 2 FdK = 70.4(1.0)(1.25)(1.0)(35)(16)(6.67) 2 FdJ = 616 kips 70.4(1.0)(1.25)(1.0)(35)(14)(20) 2 hdes, K = Trib height = 8 + 8 = 16 ft FdK = 8624 kips 1 2 FdK = 70.4(1.0)(1.25)(1.0)(35)(16)(6.67) 2 FdJ = 616 kips 70.4(1.0)(1.25)(1.0)(35)(14)(20) 2 hdes, K = Trib height = 8 + 8 = 16 ft FdK = 8624 kips 1 2 FdK = 70.4(1.0)(1.25)(1.0)(35)(16)(1.25)(1.0)(35)(1.0) 1096.2 kips Hydrostatic on interior walls Fh = 1 y s bhdes = 2 1 70.4(35)3 Load Case 2 J 1 ft 8 ft 3 ft K FdK 8 ft 8 ft 2 2 Fh = 11.1 kips Load Case 3 J 3 ft 33 ft Fh FdJ 8 ft 2 2 Fh = 11.1 kips Load Case 3 J 3 ft 33 ft Fh FdJ 8 ft 2 2 Fh = 11.1 kips Load Case 3 J 3 ft 33 ft Fh FdJ 8 ft 2 2 Fh = 11.1 kips Load Case 3 J 3 ft 33 ft Fh FdJ 8 ft 2 Debris Impact on CD FdK K Fi = 330(0.65)(1.0) Fh = 214.5 kips Hydrostatic on inside walls 2-27 Copyright © 2018 McGraw-Hill Education. C A B 7' 7' P9.4 (Assume E = 29,000 ksi and I = 100 in.4) Selecting B as the redundant, the compatibility equation is: $X \Delta BO + \delta BB X B = \Delta B$, spring = - B K From P9.4: $\Delta BO + \delta BB X B = \Delta B$, spring = - C XB 3 kips/ft A 14.292 EI K = 235 kip/in $\Delta B = -B 235$ EI EI X B = By = 14 kips MC' A C Δ BO δ BBXB = By From symmetry: Ay = Cy Σ Fy = 0 = -3(14) + 14 + 2 Ay Ay = Cy = 14 kips Ay = Cy = 14 kips Ay = Cy = 14 kips Ay = Cy Σ Fy = 0 = -3(14) + 14 + 2 Ay Ay = Cy Σ Fy = 0 = -3(14) + 14 Analyze the structure in Figure P10.17. Determine the forces in all bars of the trusses. (d) Calculate the total weight of each truss and determine which truss has a more efficient configuration. Continued 16 17 18 19 20 21 22 23 1 - 1.989 0 0 2 - 1.989 0 0 2 - 1.989 0 0 2 - 1.989 0 0 2 - 1.989 0 0 2 - 1.989 0 0 3 - 1.989 0 0 2 - 8.656 0 0 3 - 8.656 0 0 4 - 8.656 0 0 5 - 1.989 0 0 5 -
1.989 0 0 5 - 1.989 0 5 - 1.98 8.656001 - 12.241002 - 12.241003 - 12.241003 - 12.241003 - 12.241003 - 12.241003 - 12.241005 - 12.24105 - 12.24 $3\ 0\ 0\ 4\ 0\ 0\ 5\ 0\ 0\ 0\ 5\ 0\ 0\ 0\ 5\ 0\ 0\ 0\ 13-52$ Copyright © 2018 McGraw-Hill Education. (frame is still not stiff enough) 5-62 Copyright © 2018 McGraw-Hill Education. Σ MC = 0 = 21⁶ -12³ - 4 FGF + Σ Fy = 0 = -FBCY + 9 = 0 FGF = 22.5 kN FBCY = 9 compr. The load P is equal to 55 kips, and the load is eccentric from the column centerline with an eccentricity of 10 in. 8-49 Copyright © 2018 McGraw-Hill Education. Write the equations for moment as a function of distance along the longitudinal axes for members AB and BC of the frame in Figure P5.10. 4-54 Copyright © 2018 McGraw-Hill Education. "I": 4-44 Copyright © 2018 McGraw-Hill Education. The average 2 2 area of the interior column is 42 in. D 90 kN E 6m 60 kN C F 6m 30 kN B G 6m H A 8m P4.26 4-27 Copyright © 2018 McGraw-Hill Education. Compute the vertical reaction at F. 16' P P14.2. The cantilever beam in Figure P14.2 is supported on a spring at joint B. Max. Modify stiffnesses as discussed in Section 11.5. Draw the shear and moment curves for the top slab AB. Continued Segment "BC": + $\Sigma M B = 0$; $(30 c)^2 - VCB (30 c) + 260.5 = 0.2 VCB = 49.74 k - 118.4 + 3k/ft + <math>\Sigma Fy = 0$; -3k/ft (30 c) + 49.74 + VBC = 0 VBC = 40.26 k Segment "AB": $+\Sigma M A = 0$; -VBA (20 c) + 118.4 + 59.2 = 0 VBA = 8.88 k - VAB = 8.88Member FP AB +30 BC +50 5 3 0 CD -30 DA -30 BD -80 FQ + - Σ = 14100 (a) δBH WQ=UQ L 14100 14100 = AE E 30,000 = 0.47 ¢¢ 1k · δBH = ΣFP FQ δBH (b) ΔLAB To restore B to orginal horizontal postion. P4.2. Classify the trusses in Figure P4.2 as stable or unstable. For the frame in Figure P8.33, compute the horizontal and vertical displacements at joint 4 2 B. In addition to the applied load, the temperature of beam BC 4 increases by 60°F. I In4 10 kips P8.38. P2.10. Determine the forces or components of force in all bars of the trusses in Figure P4.50. No reproduction or distribution without the prior written consent of McGraw-Hill Education. 3 4 FMCX = 5.33 kN (T) FMCY = 4 kN 2 2 FMC = FMCX + FMCY = 6.666 kN 4-40 Copyright © 2018 McGraw-Hill Education. Compute I, Note: IE Joint C is rigid, I can be reduced Approx 7 of D ΣM A = 0 1 '14 - Dy 30 = 0 14 7 Dy = = 30 15 P-System Q-System 8-47 Copyright © 2018 McGraw-Hill Education. Analysis of ½ of structure due to symmetry. Q-System 8-48 Copyright © 2018 McGraw-Hill Education. The weight of light fixtures and utilities suspended from the bottom of the slab is 2 estimated to be 5 lb/ft . 2-16 Copyright © 2018 McGraw-Hill Education. The weight of light fixtures and utilities suspended from the bottom of the slab is 2 estimated to be 5 lb/ft . 2-16 Copyright © 2018 McGraw-Hill Education. The weight of light fixtures and utilities suspended from the bottom of the slab is 2 estimated to be 5 lb/ft . 2-16 Copyright © 2018 McGraw-Hill Education. Freebody Left of Section (1) Joint A, Compute FAD: Σ Fy = 0; + - FAD + 40 k - 10 k = 0 FAD = 30 k compr. (b) Sketch the influence line for axial load in column AB and indicate the spans that should be loaded to maximum the axial load. ASCE Approximate method produces a larger value of base shear. If the truss is to be designed for a uniform live load of 0.32 kip/ft that can be placed anywhere on the span in addition to a concentrated live load of 13 kips that can be positioned where it will produce the largest force, compute the maximum tension and compression forces in bar HD. Consider only bending deformations. 10 kN 2m B D E 20 kN 4m A 2m 4m P3.29 Compute Reactions + $\Sigma M A = 0$; -RB 2 m + 10 (6 m + 20) Am = 0 RB = 70 kN + $\Sigma Fx = 0$; -Ax + 10 = 0; Ax = 10 kN - + $\Sigma Fy = 0$; Ax = 10 kN - + $\Sigma Fy = 0$; Ay = 90 kN Freebody Diagrams Note: Bar CD is a two-force member. hinge (a) (b) (c) hinge hinge (d) link (e) P3.32 (a) Indeterminate, 3° (b) Determinate, 3° (c) Determin (c) Determinate (d) Indeterminate, 2° (e) Indeterminate, 2° (f) Determinate 3-33 Copyright © 2018 McGraw-Hill Education. The arch carries most of the forces produced by the vehicles passing over the bridge. Draw the influence lines for the reactions at A and F and for the shear and moment at Section 1. under the applied loads, compute the 2 reactions. Compute cross sectional area: æ1 ö Area = $(0.5 ¢ \ ^2 6 ¢) + 2 cc \ ^2 0.5 ¢ \ ^2 2.5 ¢) + (1.5 ¢ \ ^2 120 lb/ft \ ^2 = 900 lb/ft$. (b) What is the axial force in member AD and in member AB? S B $3/7 \ 1/4 \ 4/7$ C MBC ... D C D C FEM 1/4 1/2 (Symmetric) A = 50.8 ... 1.6 -6.2 29.1 46.5 0 F -162.8 FEM 58.2 D 3.1 D 0.2 D -101.3 MBA -162.8 FEM 29.1 C 1.6 C 0.1 C -132.0 MBA x2 x2 55.3 ... 0.8 -6.2 14.5 46.5 0 MBC ... C D C D FEM 3/7 12' D 4/7 E -108.5 FEM 62 D -8.3 D -0.5 D 73.3 MBA -108.5 FEM 31 C -4.2 C -0.25 C -82.0 MBA 11-35 Copyright © 2018 McGraw-Hill Education. Use E = 29,000 2 kips/in. 15° compression ring 1 1 compression ring 60' 60' Section 1-1 P6.15 Each cable carries a tributary width equal to the arc length at one end and zero at the other. Draw the influence lines for the forces in members BC, AC, CD, and CG. Analyze the frame in Figure P11.30 by moment distribution. E hB B hC hD C D 20 kips 30 kips 20' 50 kips 30' 50' 50' F6.11 A Σ M A = 20(20) + 30(50) + 50(100) - E y (150) = 0 E hB E y = 46 kips Ay hC hD 30 kips 50 kips 20' 50 kips 30' 50' F6.11 A Σ M A = 20(20) + 30(50) + 50(100) - E y (150) = 0 E hB E y = 46 kips maximum in cable? Compute all reactions and draw the moment diagrams for all members. P6.27. Computer study. The pin joint at B can be treated as a hinge. Draw the shear and moment curves for the beam in Figure P5.32. Indeterminate, stable Geometrically unstable: concurrent reactions 4-2 Copyright © 2018 McGraw-Hill Education. 12 kN 6 kN 12 kN B C D 3m E A G F 4@4m P4.45 + \Sigma MC = 0, FBD to right of C RAK = REX: 12 (4) -15(8) + REX 3 = 0 REX = 24 kN Joint A + Σ Fy = 0: YAB = 15 kN (C) 4 X AB = 15 = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C)
Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 - FAG = 0 FAG = 4 (C) Joint B FBC = 20 kN 3 + Σ Fx = 0: 24 - 20 + Σ Fx = 0: 24 + Σ Fx = 0: 24 + Σ Fx = 0: 24 + masonry wall constructed of lightweight, hollow concrete 2 block that weighs 38 lb/ft. Consider both axial and bending deformations. Beam IJ: M IJ @ 0.50 FEM = 0.50 wL2 = 60 ft k 12 VIJ @ 0.45(20 ¢ 3.6) = 32 k wL2 3.6(20)2 = 160ft k VJI @ 0.55(20 ¢ 3.6) = 40 k 9 9 Beam JK Since interior beam and spans are about the same then: M JI @ wL2 3.6(24)2 = 172.8 ft k 12 12 V @ each end = 0.50 wL = 0.50(3.6)24 = 43.2 k M JK = M KJ = 13-21 Copyright © 2018 McGraw-Hill Education. It consists of a 10-in.thick reinforced concrete slab supported on steel beams. C P = 18 kN P8.24. In Figure P8.27 support D is constructed 1.5 in. 2 3 6 @ 6.67' = 40' C C 3 C1 40' 20' P2.4 (a) AT = 10(20) = 200 ft , K LL = 2, AT $K LL = 400 > 400 2 w \approx 15 \ddot{o} \div L = 60 cc_{0.25} + \div = 60 psf ce^{400} = 800 lb/ft = 0.60 kips/ft (b) AT = 6.67(20) = 133.4 ft$, $K LL = 2, AT K LL = 1440 > 400 2 \approx ce^{2} L = 60 cc_{0.25} + P = P P P P \ddot{o} \div 60 \div 500 lb/ft = 0.60 kips/ft (c) AT = 36(20) = 720 ft$, $K LL = 2, AT K LL = 1440 > 400 2 \approx ce^{2} L = 60 cc_{0.25} + P = P P P P \ddot{o} \div 60 \div 500 lb/ft = 0.60 kips/ft (c) AT = 36(20) = 720 ft$, $K LL = 2, AT K LL = 1440 > 400 2 \approx ce^{2} L = 60 cc_{0.25} + P = P P P P \ddot{o} \div 60 \div 500 lb/ft = 0.60 kips/ft (c) AT = 36(20) = 720 ft$, $K LL = 2, AT K LL = 1440 > 400 2 \approx ce^{2} L = 60 cc_{0.25} + P = P P P P \ddot{o} \div 60 \div 500 lb/ft = 0.60 kips/ft (c) AT = 36(20) = 720 ft$, $K LL = 2, AT K LL = 1440 > 400 2 \approx ce^{2} L = 60 cc_{0.25} + P = P P P P \ddot{o} \div 60 \div 500 lb/ft = 0.60 kips/ft (c) AT = 36(20) = 720 ft$, $K LL = 2, AT K LL = 1440 > 400 2 \approx ce^{2} L = 60 cc_{0.25} + P = P P P P \ddot{o} \div 60 \div 500 lb/ft = 0.60 kips/ft (c) AT = 36(20) = 720 ft$, $K LL = 2, AT K LL = 1440 > 400 2 \approx ce^{2} L = 60 cc_{0.25} + P = P P P P \ddot{o} \div 60 \div 500 lb/ft = 0.60 kips/ft (c) AT = 36(20) = 720 ft$, $K LL = 2, AT K LL = 1440 > 400 2 \approx ce^{2} L = 60 cc_{0.25} + P = P P P P \ddot{o} \div 60 \div 500 lb/ft = 0.60 kips/ft (c) AT = 36(20) = 720 ft$, K LL = 2, AT K L $38.7 \text{ psf} > , \text{ ok } 2\ 1440 \ \emptyset \ 15 \text{ q} (Wtrib)(Lbeam) = 2\ 38.7(8)(40)\ 5 \text{ spaces} @ 8' \text{ each} = 6192 \ \text{lbs} = 6.19 \ \text{kips} \ \text{G3}\ 2 \ \# 80 \ 2 \ (d) \ \text{AT} = cc \div 40 + 33.33(10) = 493.3 \ \text{ft}, \ \text{KLL} = 2, \ \text{AT}\ \text{KLL} = 986.6 > 400 \ \text{ce}\ 2 \ \div \emptyset \ \# \ 15 \ 0^{\circ} + 60 \ \text{L} = 60 \ cc \ 0.25 \ + \div = 43.7 > , \ \text{ok}\ ce^{2}\ 2\ 986.6 \ \emptyset \ w \ P \ P \ P \ w = 43.7(4) = 174.8 \ \text{lb/ft} = 0.17 \ \text{kips/ft}\ P = 43.7(6.67(20)\ 6 \ \text{spaces}\ @ 6.67'$ each = 2914.8 lbs = 2.91 kips G4 2 2-8 Copyright © 2018 McGraw-Hill Education. Given: E is constant. E 6m B C D G H 6m 40 kN A I 40 kN A [40 kN A [40 kN A] 40 kN A] 40 kN A [40 kN A] 40 kN A [40 kN A] 40 kN A] 40 kN A [40 kN A] 40 kN A] 40 kN A [40 kN A] 40 kN A] 40 kN A [40 kN A] 40 kN A] 40 kN A] 40 kN A [40 kN A] 40 kN Unknowns: θB , θD FEMS wL2 5(15)2 375 375 ==k · ft, FEM BA = k · ft 12 12 4 4 wL2 5(8)2 80 80 == - k · ft, FEM CB = k · ft FEM BC = 12 12 3 3 Member end Moments FEM AB = - 2 EI 375 2 EI 375 (θ), M BA = ($2\theta B$ + θD), M DB = ($2\theta D$ + θB), M BC = -(5k/ft)(8ϕ) $c_{c} \div = -160 \text{ k}\cdot\text{ft} = ce^{2} 2 \div 0 12 12 \text{ M AB} = -2 EI 375 (<math>\theta$), M BA = ($2\theta B$ + θD), M DB = ($2\theta D$ + θB), M BC = -(5k/ft)(8ϕ) $c_{c} \div = -160 \text{ k}\cdot\text{ft} = ce^{2} 2 \div 0 12 12 \text{ M AB} = -2 EI 375 (<math>\theta$) + 15 B 4 15 4 $\approx 8\phi \ddot{o} 2 EI 2 EI (2 \theta B + \theta D)$, M DB = ($2\theta D$ + θB), M BC = -(5k/ft)(8ϕ) $c_{c} \div = -160 \text{ k}\cdot\text{ft} = ce^{2} 2 \div 0 12 12 \text{ M AB} = -2 EI 375 (<math>\theta$) + 15 B 4 15 4 $\approx 8\phi \ddot{o} 2 EI 2 EI (2 \theta B + \theta D)$, M DB = ($2\theta D$ + θB), M BC = -(5k/ft)(8ϕ) $c_{c} \div = -160 \text{ k}\cdot\text{ft} = ce^{2} 2 \div 0 12 12 \text{ M AB} = -2 EI 375 (<math>\theta$) + 15 B 4 15 4 $\approx 8\phi \ddot{o} 2 EI 2 EI (2 \theta B + \theta D)$, M DB = ($2\theta D$ + θB), M BC = -(5k/ft)(8ϕ) $c_{c} \div = -160 \text{ k}\cdot\text{ft} = ce^{2} 2 \div 0 12 12 \text{ M AB} = -2 EI 375 (<math>\theta$) + 15 B 4 15 4 $\approx 8\phi \ddot{o} 2 EI 2 EI (2 \theta B + \theta D)$, M DB = ($2\theta D$ + θB), M BC = -(5k/ft)(8ϕ) $c_{c} \div = -160 \text{ k}\cdot\text{ft} = ce^{2} 2 \div 0 12 12 \text{ M AB} = -2 EI 375 (2 \theta B + \theta D)$), M DB = ($2\theta D$ + θB), M BC = -(5k/ft)(8ϕ) $c_{c} \div = -160 \text{ k}\cdot\text{ft} = ce^{2} 2 \div 0 12 12 \text{ M AB} = -2 EI 375 (2 \theta B + \theta D)$), M DB = ($2\theta D$ + θB), M BC = -(5k/ft)(8ϕ) $c_{c} \div = -160 \text{ k}\cdot\text{ft} = ce^{2} 2 \div 0 12 12 \text{ M AB} = -2 EI 375 (2 \theta B + \theta D)$), M DB = ($2\theta D$ + θB) $c_{c} \div 0 = -2 EI 375 (2 \theta B + \theta D)$), M DB = -($2\theta D$ + θB) $c_{c} \div 0 = -2 EI 375 (2 \theta B + \theta D)$ M BD Equilibrium at Joint B Equilibrium at Joint D M BA + M BD + M BC = 0 (1) M DB = 0 (2) from Eqns. 12-23 Copyright © 2018 McGraw-Hill Education. (b) Assuming the wind pressure on the windward side varies linearly between the 35-ft intervals, determine the total wind force on the building in the direction of the wind. P9.45. Which loading is more critical? = 11.68k 13-10 Copyright © 2018 McGraw-Hill Education. C 10' 6 kips B D 20' A E 40' P3.9 Entire Structure: + $\Sigma M A = 0$; 16 k (20 ¢) + 6 k $(20 \ c) - E \ y$ (40 c) = 0 E y = 11 kips + $\Sigma Fy = 0$; $Ay + 11k - 16 \ k = 0 \ Ay = 5 \ kips \ \neg$ Entire Structure: + $\Sigma Fx = 0$; $6 \ k - 4.67 \ k - Ax = 0 \ Ax = 1.33 \ kips \ \neg 3-10 \ Copyright$ (20 c) + $E \ x (30 \ c) = 0 \ E \ x = 4.67 \ kips \ \neg$ Entire Structure: + $\Sigma Fx = 0$; $6 \ k - 4.67 \ k - Ax = 0 \ Ax = 1.33 \ kips \ \neg 3-10 \ Copyright$ (20 c) + $E \ x (30 \ c) = 0 \ E \ x = 4.67 \ kips \ \neg$ Entire Structure: + $\Sigma Fx = 0$; $6 \ k - 4.67 \ k - Ax = 0 \ Ax = 1.33 \ kips \ \neg 3-10 \ Copyright$ (20 c) + $E \ x (30 \ c) = 0 \ E \ x = 4.67 \ kips \ \neg 3-10 \ Copyright$ (20 c) + $E \ x (30 \ c) = 0 \ E \ x = 4.67 \ kips \ \neg 3-10 \ Copyright$ (20 c) + $E \ x (30 \ c) = 0 \ E \ x = 4.67 \ kips \ \neg 3-10 \ Copyright$ (20 c) + $E \ x (30 \ c) = 0 \ E \ x = 4.67 \ kips \ \neg 3-10 \ Copyright$ (20 c) + $E \ x (30 \ c) = 0 \ E \ x = 4.67 \ kips \ \neg 3-10 \ Copyright$ (20 c) + $E \ x (30 \ c) = 0 \ E \ x = 4.67 \ kips \ \neg 3-10 \ Copyright$ (20 c) + $E \ x (30 \ c) = 0 \ E \ x = 4.67 \ kips \ \neg 3-10 \ Copyright$ (20 c) + $E \ x (30 \ c) = 0 \ E \ x = 4.67 \ kips \ \neg 3-10 \ Copyright$ (20 c) + $E \ x (30 \ c) = 0 \ E \ x = 4.67 \ kips \ \neg 3-10 \ Copyright$ (20 c) + $E \ x (30 \ c) = 0 \ E \ x = 4.67 \ kips \ \neg 3-10 \ Copyright$ (20 c) + $E \ x (30 \ c) = 0 \ E \ x = 4.67 \ kips \ \neg 3-10 \ Copyright$ (20 c) + $E \ x (30 \ c) = 0 \ E \ x = 4.67 \ kips \ \neg 3-10 \ Copyright$ (20 c) + $E \ x (30 \ c) = 0 \ E \ x = 4.67 \ kips \ \neg 3-10 \ Copyright$ (20 c) + $E \ x (30 \ c) = 0 \ E \ x = 4.67 \ kips \ \neg 3-10 \ Copyright$ (20 c) + $E \ x (30 \ c) = 0 \ E \ x = 4.67 \ kips \ \neg 3-10 \ Copyright$ (20 c) + $E \ x (30 \ c) = 0 \ E \ x = 4.67 \ kips \ \neg 3-10 \ Copyright$ (20 c) + $E \ x (30 \ c) = 0 \ E \ x = 4.67 \ kips \ \neg 3-10 \ Copyright$ (20 c) + $E \ x (30 \ c) = 0 \ E \ x = 4.67 \ kips \ x =$ Continued Member Numbers & Joint Letters Exact Analysis - Risa Software Results, (C) Reactions X Force Y Force Moment (k) (k) (k-ft) -4.893~5 k -6.667 0 K -5.107~5 k 6.667 0 K -5.107~5 k 6.667 0 Totals -10 Joint Label X Translation Y Translation Y Translation Y Translation Y Translation (in) (in) (radians) A 0 0 -5.531e-3 B .524 002 -2.04e-3 C .577 0 0 L .576 -.028 0 M .579 -.020 N .579 -.005 0 0 .579 .008 0 P .575 .021 0 I .576 0 0 D .59 -.024 0 E .58 -.023 0 F .575 -.005 0 G .576 .012 0 H .58 .016 0 J .53 -002 -1.983e-3 k 0 0 -5.627e-3 Reactions & Deflected Shape Moment Diagram Axial Force (See Label Diagram w/Table of MBR Forces) 13-49 Copyright © 2018 McGraw-Hill Education. A P3.36. MCD = (2 θ C), M DC = (θ) 10 10 15 15 C Equilibrium at Joint B Equilibrium at Joint C M AB = M BA + M BE + M BC = 0 from Eqns. C P8.2. For the truss in Figure P8.1, compute the vertical displacement of joint A and the horizontal displacement of joint C. Determine (a) the absolute maximum value of live load moment and shear produced in the 50-ft girder and (b) the maximum value of moment at midspan (Figure P12.44). Continued FBD between Sections 1 and 2: V2 = 15 kips POI 90 kip-ft 1 1 6 ft $\Sigma Fx = 0 = 2(15) - 2V2$ 7.2 kips POI M BA = M EF = M 2 M 2 = V2 (16) = 240 kip-ft 90 kip-ft E B MBE MBA = 240 M BA = M EF = 240 kip-ft MEF MDB 16 ft A 2 F V2 2 V2 Equilibrium of Joint B: $\Sigma FB = 0 = 90 + 240 - M BD F2 F2 M BD = 330 \text{ kip-ft FBC} = 7.2 \text{ kips MBC} = 90 \text{ kip-ft V1} = 15 \text{ kips MBC} =
26.4 \text{ kips } \Sigma M POI = 0 = 330 - VBE (12.5) VEB = 26.4 \text{ kips VEB} = 26.4 \text{ kips VEB}$ FBA = 33.6 kips FBA = 33.6 kips MBA = 240 kip-ft B Summary of Portal Method Member End Forces 90 kip-ft 9 FBE + VBC - VBA FBE = 0 Equilibrium of Column AB: VBA = 15 kips 15 kips 330 kip-ft 240 kip-ft 33.6 kips (C) FBA = 33.6 kip 330 kip-ft 240 kip-ft 33.6 kips (C) FBA = 33.6 kip 330 kip-ft 240 kip-ft 33.6 kips (C) FBA = 33.6 kip 330 kip-ft 240 kip-ft 33.6 kips (C) FBA = 33.6 kip 330 kip-ft 240 kip-ft 33.6 kips (C) FBA = 33.6 kip 330 kip-ft 240 kip-ft 33.6 kips (C) FBA = 33.6 kip 330 kip-ft 240 kip-ft 33.6 kips (C) FBA = 33.6 kip 330 kip-ft 240 kip-ft 33.6 kips (C) FBA = 33.6 kip 330 kip-ft 240 kip-ft 33.6 kip 330 kip-ft 33.6 kip 33.6 kip 33.6 kip 33.6 kip 33.6 kip 33.6 kip 33.6 ki applied to the roadway have two paths to the end supports. The purpose of this study is to show that the magnitude of the joint displacements as well as the magnitude of the forces in members may control the proportions of structural members. 180 kN 300 mm A 600 mm B C varies 300 mm 6m 320 mm 3m 2@1m P8.43 E = 3500 k/in.2 = 24,100 MP 2 Δx = 1m Table 1 M P2 I (10-6) Segment No. MP kN m h mm 1 51.43 325 bh 3 12 mm '106 915.41 2 154.29 375 1406.25 16.928 3 257.15 425 2047.08 32.303 4 360.01 475 2857.92 45.350 5 462.87 525 3858.75 55.523 6 565.73 575 5069.58 63.132 7 668.59 600 5760.00 6760.00 6760.00 68.906 9 450.00 5760.00 450 2430.00 30.00 11 90.00 350 1143.33 7.085 I= 2.889 Σ = 445.4 '10-6 Compute ΔC: Base Properties of Segment on Depth @ Center Use P-System as Q-System as Q-System as Q-System as Q-System (MQ = MP) Δx æç Δx ö ÷ ΣMQ2 = ç ÷ EI çè E ÷ø 0.5 I æ 1 ÷ ö é (445.4 '10-6) ù çæ10-2 ÷ ö ú ÷ ΔC = çç ÷ ê ú ççè ç 180 ÷ ø ÷ çè 24100 ÷ ø ê 0.5 ë û 180 kN · ΔC = ΣMQ M P = 205 mm 8-50 Copyright © 2018 McGraw-Hill Education., all other bars = 2 in. Given: AE = constant, 2 A = 1000 mm, and E = 200 GPa. B 6m C D 48 kN 8m 8m P9.28 Selecting Cx as the redundant, the compatibility equation is: $(\Delta CS + \Delta CO) + \delta CC XC = \Delta C \delta CC = \Sigma FQ2 L AE \delta CC = 2 8(1000) 5 10(1000) 3 6(1000) + 2() + () (12) 1000(200) 8 1000(20$ $1000(200) 2 2 2 = 0.0633 \text{mm} \ A \ 6(1000) (83) 24 \ 1000(200) \ \Delta \text{CO} = \Sigma \text{FQ} \ \Delta \text{LP} = -1/2 \ (T) \ B \ 0.375 \ \Delta \text{CO} = -0.27 \ \text{mm} \ \neg \text{Therefore}, \ (\Delta \text{CS} + \Delta \text{CO}) + \delta \text{CC} \ \text{XC} = \Delta \text{C} \ -3/8 \ (C) \ C \ Q \ -5ystem \ (kN) \ 1 \ 1/2 \ (T) \ 0.375 \ (O \ -0.27) + 0.0633 \ \text{X} = 0 \ \text{C} \ \text{X} = C \ \text{x} = 4.27 \ \text{kN} \ \text{C} \ \text{A} \ -32 \ (C) \ \text{B} \ 24 \ 24 \ (T) \ \text{C} \ \text{P-System} \ (kN) \ 32 \ (T) \ 48 \ 24 \ \text{Ax} = 4.27 \ \text{A} \ -29.9 \ (C) \ \text{B} \ \text{Ay} = 25.60 \ 22.4 \ (T) \ \text{Final}$ Results (kN) C 48 34.1 (T) Cy = 22.40 9-30 Copyright © 2018 McGraw-Hill Education. 8' w = 8 kN/m P11.14. Computer Analysis of Truss with Rigid Joints. P5.53. If the code requires that the maximum lateral deflection at the top of the roof not exceed 0.48 in. 20' B C 100 kip ft 20' A 15' 20' P10.29 Unknowns: θA , θB , θC , Δ : Chord rotations in terms of Δ : $\Delta \Psi AB = -\Psi 25 \Delta 3\Delta 3 \Psi BC = V = = \Psi 25 5(20) 4 \Delta 4\Delta \Delta \Psi CD = H = = = \Psi 20 5(20) 25$ Relative stiffness setting 10E=K: Member end moments: $3MAB = (2) K (2\theta A + \theta B + 3\Psi) 5 3 M BA = (2) K (2\theta B + \theta A + 3\Psi) 5 3 M BA = (2) K (2\theta C - 3\Psi) 4 3 MCB =$ = K 25 5 (200) E = = K 20 (150) E 3 = = K 20 4 K AB = K BC KCD ((Equilibrium equations: M AB = 0, M BA + M BC = 0, M CD + M DC = 0 M + M DC + CD = 0 25 20 Therefore: 12 6 18 $\theta + \theta + \Psi = 0 5 A 5 B 5 6 32 9 100$ (Eq. 2) $\theta A + \theta B + 2qC - \Psi = 5 5 10 K$ (Eq. 3) $2 \theta B + 7 \theta C - 9\Psi = 0$ ()) ()) (Eq. 1) (Eq. 4) areas for each bar, given an allowable tensile stress of 45 ksi, and an allowable compressive stress of 24 ksi. B C 15' A 30' P9.41 Bar BC Rabricated 1.2" too Long. 0.06 (1800 k) = 0.033W = 59.2 k Controls 0.27 (1800 k) = 0.044 SDS IW = 0.044 (0.27)(1.5)(1800 k) = 0.0178W = 32.1k Force @
Each Level FX = WX hxk ΣWi hik Vbase, Where Vbase = 59.2 k T < 0.5 Sec. Tension in bar CG. Check results by moment distribution. (a) The arch in Figure P6.32 supports a roadway consisting of simply supported beams connected to the arch by high-strength cables with area 2 A = 2 in. Compute the horizontal and vertical components of deflection at point D in 4 Figure P8.28. 39 kips 40 kips P8.17. 15 kN F E w = 6 kN/m 30 kN G 3m D 4m w = 6 kN/m 30 kN H C 6m A B 12 m (not to scale) P3.17 + $\Sigma Fx = 0$; 15 + 30 + 30 - RAx = 0 RAx = 75 kN - + $\Sigma M A = 0$; 30 $^{\prime} 6 m + 30 ^{\prime} 10 m + 15 ^{\prime} 13m + 72 ^{\prime} 6 + 72 ^{\prime} 6 + 72 ^{\prime} 6 + 72 ^{\prime} 6 + 78 ^{\prime} 8 + 20 ^$ RAy = 0 RAy = 39.75 kN 3-18 Copyright © 2018 McGraw-Hill Education. P13.24. wL 2 P3.35. Locate all points of zero shear and moment. See Figure P5.57b. In addition to the load at joint C, the temperature of member BD is subject to a temperature increase of $60^{\circ}F$. 10 kips P12.42. The building, located on level flat ground, is classified as stiff because its natural period is less than 1 s. Use 4 E = 30,000 ksi and I = 600 in. P3.30. Q4 = 58.54 kips Q3 = 60 kips $\Delta 1 = 0.2915$ in. 60 kips P4.14. (a) Determine the reactions and all bar forces of the three-hinged, trussed arch in Figure P6.27 for the following cases: Case A: Only the 90-kN force at joint D acts., ABD = 5 in. D B C hinge 6m E A 15 m 15 m P8.34 MP Moments MP = $-78.75x \times 2$ M P = $-2.1x \times 2$ M P = -2.BC + CD) P-System 15 (-1.25 x)(-78.75 x) (-0.5 x)(-2.1x 2) dx dx + 2 ò EI EI 0 éæ 98.44 ö 3 ù 2 êç ÷ x ú + êçèç 3 ÷ø ú EI ë û 0 6 é1.05 x 4 ù ê ú ê 4 ú ë û 0 15 é 7088 + 13290ù ë û 40, 756 kN.m 200 '106 '1800 '10-6 = 0.113 m Q-System 8-40 Copyright © 2018 McGraw-Hill Education. 10 kips B C 15' A D 30' P13.20 FBD "BC" Girder (a) Fixed Column Bases + $\Sigma MC = 0$; 2(301·k) - VBC (30 ¢) = 0 VBC = 2 k \ Axal Force in Column AB = 2 k T Column DC = 2 k C Assumptions: 1) Shear in Columns are Equal 2) P.I. in Columns @ 0.6h = 9¢ FBD Column "A-P.I." + $\Sigma M P$. Its roof consists of asphalt shingles. (Recall that two columns don't resist horizontal load because they are oriented in the vertical direction.) If the beam (Member 2) were stiff, i.e., with a large cross sectional area such that it cannot be shortened easily, joints 2 and 3 would deflect to the right by approximately the same amount, which would have resulted in about the same axial forces in Members 4 and 5, except that Member 4 is in compression and Member 5 is in tension. Sketch the deflected shape, and determine the deflection at midspan. 35 kN C w = 6 kN/m 3m D B 6m A 9m P10.22 wL $26(9)2 = -40.5 \text{ kN} \cdot \text{m}$ 12 2 EI M AB = $\theta 6 B 2 EI 2 \theta B M BA = 6 2 EI 2 \theta B + \theta D - 40.5 M BD = 9 2 EI 2 \theta D + \theta B + 40.5 M DB = 9 FEM BD = -4 EI 2 EI \theta + \theta = -40.5 9 D 9 B 4 EI 2 EI 2 EI 2 H AB = 0 6 B 2 EI 2 \theta B M BA = 6 2 EI 2 \theta B M BA = 6 2 EI 2 \theta B M BA = 6 2 EI 2 \theta B M BA = 6 2 EI 2 \theta B + \theta D - 40.5 M DB = 9 FEM BD = -4 EI 2 EI \theta + \theta = -40.5 9 D 9 B 4 EI 2 EI 2 EI 2 \theta B M BA = 6 2 EI 2 \theta B M BA = 6 2 EI 2 \theta B + \theta D - 40.5 M BD = 9 2 EI 2 \theta D + \theta B + 40.5 M DB = 9 FEM BD = -4 EI 2 EI \theta + \theta = -40.5 9 D 9 B 4 EI 2 EI 2 EI 2 \theta B M BA = 6 2 EI 2 \theta B M BA = 6 2 EI 2 \theta B + \theta D - 40.5 M DB = 9 2 EI 2 \theta D + \theta B + 40.5 M DB = 9 FEM BD = -4 EI 2 EI \theta + \theta = -40.5 9 D 9 B 4 EI 2 EI 2 EI 2 \theta B M BA = 6 2 EI$ $(2 a) \theta + \theta + EI \theta B = 145.59 B 9 D 3$ from (1a) and (2a) (1a) 165.75 EI -174 $\theta D = EI \theta B = () () ()$ Joint Equilibrium EQ's (1) M DB = 0.2 EI ($2 \theta D + \theta B$) + 40.5 = 0.9 -105 + M BD + M BA = 0.105 + 2 EI $\approx c 165.75 + c 174 \theta D = EI \theta B = () () () ()$ Joint Equilibrium EQ's (1) M DB = 0.2 EI ($2 \theta D + \theta B$) + 40.5 = 0.9 -105 + M BD + M BA = 0.105 + 2 EI $\approx c 165.75 + c 174 \theta D = EI \theta B = () () () ()$ Joint Equilibrium EQ's (1) M DB = 0.2 EI ($2 \theta D + \theta B$) + 40.5 = 0.9 -105 + M BD + M BA = 0.105 + 2 EI $\approx c 165.75 + c 174 \theta D = EI \theta B = () () () () = 0.2 EI (2 \theta D + \theta B) + 40.5 = 0.9 -105 + M BD + M BA = 0.105 + 2 EI \approx c 165.75 + c 174 \theta D = EI \theta B = () () () () = 0.2 EI (2 \theta D + \theta B) + 40.5 = 0.9 -105 + 2 EI \approx c 165.75 + c 174 \theta D = EI \theta B = () (0 + 0.5 + 0$ EI (2 θ B + θ D) - 40.5 9 2 EI (2 θ B) = 0 6 Substituting θ B = θ D into EQ's for member end moments gives. 4 2 For the columns, I = 640 in. The area of all bars is shown on the sketch of the 2 truss; E = 30,000 kips/in. 20 kN P4.51. (c) If the temperature of bars AB and BC increases 80°F, determine the vertical displacement of joint -6 C. Locate all inflection points, and sketch the deflected shape. + $\Sigma Fy = 0$; 9k - Fby = 0 5 5 Fb = Fby = (0) = 15 kips (T) 3 3 + $\Sigma Mc = 0$; -12 k ' 9 ¢ - Fhg 9 ¢ = 0 Fc = Fhg = -12 kips \ compr. The shear connection at B acts as a hinge. P13.23. P 5m D B 3m 1 8m 2 4m A 5m P12.26 - 1 C 0 3 0 5m 0 0 B 4m 0 4m A - 1 V1 (kN) - 4 1.6 M1 (kN-m) - 4.8 V2 (kN) M2 (kN-m) 12-29 Copyright © 2018 McGraw-Hill Education. A A' C hinge δ =?, 2 E = 29,000 kips/in. D 6' B P = 2 kips C 10' A 20' P8.32 Virtual Work (a) Compute θA : x QM · θA = \hat{o} (1 - 0.05 x)(0.6 x) 0 æ 0.6 x 2 0.03 x 3 \ddot{o} + = ccc + $c\dot{c}$ + $c\dot{c}$ 2 3 + ϕ dx EI 20 0 P-System 40 = EI θA = 0.00199 Rad Compute δBx : Q · δBx = $\Sigma \hat{o}$ MQ M P dx EI 20 1ft $k \cdot \theta A$ = \hat{o} (1 - 0.05 x)(0.6 x) 0 æ 0.6 x 2 0.03 x 3 \ddot{o} + = ccc + $c\dot{c}$ + $c\dot{c}$ 2 3 + ϕ dx EI 20 0 P-System 40 = EI θA = 0.00199 Rad Compute δBx : Q · δBx = $\Sigma \hat{o}$ MQ M P dx EI 20 6 1 · δ Bx = δ (0.3 x)(0.6 x) k 0 = (0.06 x 3) = 20 0 dx dx + x (2 x) EI δ 0 EI + 0.66 x 3 Q-System for θ A 624 EI δ BX = 0.372 in. Given: EI is constant throughout, 2 L = 12 ft, and E = 4000 kips/in. A bridge is composed of two trusses whose configuration is shown in Figure P12.38. Given: 2 4 E = 30,000 kips/in. Continued Observations Case 1: Cutting a horizontal section at the mid-height of the building, it can be shown that the applied horizontal load (100 kips), which can be either a wind or earthquake load in practical applications, has to be resisted (or be balanced) by the horizontal components of two diagonal member forces. E 10 kips 8' B D C 8' A 6' 6' 6' P8.33 Seg Or Range \deltaBX MO1 δBY MO2 AB A $0 \le x1 \le 10'$ 0.2x1 0.45 x1 MP 3x1 k BC B $0 \le x2 \le 6'$ 0.33(x2 + 6) -1 x2 0.75(x2 + 6 $\dot{0}(0.43 \text{ x})(3 \text{ x}) \text{ EI EI } 6 \ 10 = \dot{0} \ 0.6 \text{ x} \ 2 + 2 \dot{0} (1.665 \text{ x} \ 2 + 20 \text{ x} + 60) \text{ dx} + \dot{0} \ 1.89 \text{ x} \ 2 \text{ dx} \ 0 = 0.2 \text{ x} \ 3 = 10 + 1.11 \text{ x} \ 3 + 20 \text{ x} \ 2 + 120 \text{ x} \ 6 + 0.63 \text{ x} \ 3 \ 10 \ 2510 \ 2510(1728) = = 1
\text{ c} \text{ c} \ (150) + (29000) \text{ EI } 10 \ 6 \ 1 \text{ k} \cdot \delta \text{BY} = \dot{0} \ (0.45 \text{ x})(3 \text{ x}) \ 0 \ 6 \ \text{ dx} \ \text{ dx} + \dot{0} \ (0.75(\text{ x} + 6) - \text{ x})(5(\text{ x} + 6)) \text{ EI } \text{ EI } 10 + \dot{0} \ (-0.25)(\text{ x} + 6)(-5(\text{ x} + 6)) \ 10 \ 6 \ \text{ dx} \ \text{ dx} + (0.15 \text{ x})(-3 \text{ x}) \text{ EI } \dot{0} \ \text{EI } 6 \ 10 = \dot{0}$ $1.35 \text{ x } 2 + \delta (1.25 \text{ x } 2 + 30 \text{ x} + 135) \text{ dx} + \delta (1.25 \text{ x } 2 + 15 \text{ x} + 45) + \delta - 0.45 \text{ x } 3 \text{ 10} + 0.417 \text{ x } 3 + 15 \text{ x } 2 + 135 \text{ x } 6 + 0.417 \text{ x } 3 + 7.5 \text{ x } 2 + 45 \text{ x } 6 - 0.15 \text{ x } 3 \text{ 3024} = = 1.2 \text{ c} \text{ c} \text{ EI 10 Q1 System } \delta \text{Bx } \text{Q2 System for } \delta \text{By } 8-39 \text{ Copyright } \mathbb{C} 2018 \text{ McGraw-Hill Education.}$ tension in the cable in Figure P6.13. E is constant. EI δ BH 8-44 Copyright © 2018 McGraw-Hill Education. 1 2 3 4 5 A 3 @ 30' = 90' N B C D 4 @ 25' = 100' T = Ct hn4 / 4 [Ct = 0.035 for steel moment frames] (b) (c) = 0.0441 (1)(0.9)(3870) = 153.6 kips T = 0.75 sec. Gilbert SOLUTIONS MANUAL CHAPTER 3: STATICS OF STUCTURES - 100' T = Ct hn4 / 4 [Ct = 0.035 for steel moment frames] (b) (c) = 0.0441 (1)(0.9)(3870) = 153.6 kips T = 0.75 sec. Gilbert SOLUTIONS MANUAL CHAPTER 3: STATICS OF STUCTURES - 100' T = Ct hn4 / 4 [Ct = 0.035 for steel moment frames] (b) (c) = 0.0441 (1)(0.9)(3870) = 153.6 kips T = 0.75 sec. Gilbert SOLUTIONS MANUAL CHAPTER 3: STATICS OF STUCTURES - 100' T = Ct hn4 / 4 [Ct = 0.035 for steel moment frames] (b) (c) = 0.0441 (1)(0.9)(3870) = 153.6 kips T = 0.75 sec. Gilbert SOLUTIONS MANUAL CHAPTER 3: STATICS OF STUCTURES - 100' T = Ct hn4 / 4 [Ct = 0.035 for steel moment frames] (b) (c) = 0.0441 (1)(0.9)(3870) = 153.6 kips T = 0.75 sec. Gilbert SOLUTIONS MANUAL CHAPTER 3: STATICS OF STUCTURES - 100' T = Ct hn4 / 4 [Ct = 0.035 for steel moment frames] (b) (c) = 0.0441 (1)(0.9)(3870) = 153.6 kips T = 0.75 sec. Gilbert SOLUTIONS MANUAL CHAPTER 3: STATICS OF STUCTURES - 100' T = Ct hn4 / 4 [Ct = 0.035 for steel moment frames] (b) (c) = 0.0441 (1)(0.9)(3870) = 153.6 kips T = 0.75 sec. Gilbert SOLUTIONS MANUAL CHAPTER 3: STATICS OF STUCTURES - 100' T = 0.0441 (1)(0.9)(3870) = 153.6 kips T = 0.75 sec. Gilbert SOLUTIONS MANUAL CHAPTER 3: STATICS OF STUCTURES - 100' T = 0.0441 (1)(0.9)(3870) = 153.6 kips T = 0.75 sec. Gilbert SOLUTIONS MANUAL CHAPTER 3: STATICS OF STUCTURES - 100' T = 0.0441 (1)(0.9)(3870) = 153.6 kips T = 0.75 sec. Gilbert SOLUTIONS MANUAL CHAPTER 3: STATICS OF STUCTURES - 100' T = 0.0441 (1)(0.9)(3870) = 153.6 kips T = 0.75 sec. Gilbert SOLUTIONS MANUAL CHAPTER 3: STATICS OF STUCTURES - 100' T = 0.0441 (1)(0.9)(3870) = 153.6 kips T = 0.075 sec. Gilbert SOLUTIONS MANUAL CHAPTER 3: STATICS OF STUCTURES - 100' T = 0.0441 (1)(0.9)(3870) = 153.6 kips T = 0.075 sec. Gilbert SOLUTIONS MANUAL REACTIONS 3-1 Copyright © 2018 McGraw-Hill Education. Draw the envelopes for maximum shear and moment in a 24-ft-long simply supported beam produced by a live load of 0.4 kip/ft of variable length and a concentrated load of 10 kips (Figure P12.42). Influence Line for positive moment midspan AC. D 60 kips 15' B E G F 30 kips 15' 60 kips A 20' 20' 20' P4.18 + $\Sigma M A = 0$; - By (20 ¢) + 60 k (30 ¢) + 60 k (20 ¢) + 30 k (40 ¢) = 0 By = 210 k $\Sigma Fy = 0$; - 210 k - 60 k - 30 k + Ay = 0 + Ay = 300 k + $\Sigma Fx = 0$; - Ax + 60 k = 0 Ax = 60 k \neg 4-19 Copyright © 2018 McGraw-Hill Education. B w = 6 kips/ft 80' 20' cable A 6 @ 40' = 240' P6.5 Use

General Cable Theorem Compute Loads to Main Cable P = 6 k/ft ($40 \ c$) = 240 k Hhz = M z H (20) = 43200 H = 2160 k Compute "By" (Neglect End Cable Forces) $\Sigma M A = 0$; 240(40 + 80 + 120 + 160 + 280) - By (240) + 260(80) = 0 By = 1320 k Max T = (2160)2 + (1320)2 TMAX = 2531.4 k Reactions @ "B" By total = 1320 + 120 By = 1440 k Bx = H Bx = 2160 k Reactions @ "A" $\Sigma Fy = 0.240(6) - 1440 - Ay = 0$ Ax = H $\neg Ax = 2160 k \neg 6-6$ Copyright © 2018 McGraw-Hill Education. Δ Influence Line for Axial load in AB and loading for maximum axial load. 20 kips 24 kips 10 kips 20' 10' A B C D E L K J I 6 @ 20' = 120' P12.39 Max.

w = 0.50 kip/ft 3 diverse B 6' 1' D C w = 0.50 kip/ft 12'' 12' 14' P11.13 Base analysis on f dimensions. Determine all bar forces and reactions for the truss in Figure P9.38. 2 4 Given: E = 30,000 kip/in. 5 If A settles by -0.5 in. The roof weighs 2 25 lb/ft based on the horizontal projection of the roof area 20' P = 12 kips P5.21. *Ponding refers to the pool of water that can collect on a roof when the roof drains are not adequate to carry away rain water or become clogged. 60 kN 6m 45 kN E C 6m 30 kN F B 6m G A 8m 10 m P4.23 + Σ M A = 0; 60 ' 18 + 45' 12 + 30' 6-18RGY = 0 RGY = 100 kN Σ Fx = 0; 60 + 45 + 30 - RAK = 0 RAK =

 $6 \text{ kN C } 3m \ 10 \text{ kN B D } 3m \text{ A H G F } 4@4m \ P4.22 + \Sigma M \text{ A} = 0 = 10'3 + 6'6 - \text{RE } 16; \text{RE } = 4.125 \text{ kN } \text{ Joint E 5 FDE} = (4.125) = 5.5 \text{ kN } 3 \text{ Check } \Sigma Fx = 0 \text{ JT } \& \text{ G } \Sigma Fy = 0 \ 4-23 \text{ Copyright } @ 2018 \text{ McGraw-Hill Education}. What is the percent reduction in deflection? and I = 150 in. As a consequence, the diagonal Member 4 is stretched less with a lower axial force than Member 5.$

(b) Determine the axial forces in all links.

2 E = 29,000 kips/in.

Briefly describe the difference in behavior. Moment (ft k) Shear (k) 3 Reactions Joint Label X Force (k) Y Force (y) Moment (k-ft) A -12.601 -24.888 114.541 B -20.13 .049 177.591 C -19.966 -.082 176.218 D -12.303 24.92 111.952 Totals -65 0 13-31 Copyright © 2018 McGraw-Hill Education. Unless otherwise noted, EI is constant. E = 200 GPa for all members. P10.33. B A K = 40 kN/m C 10 m P9.21 Select RB as the Redundant. Use E = 30,000 kips/in.

 $w = 5 \text{ kips/ft A hinge 20' B C 6' P5.21 FBD "AB" + 5k/1 (20 ¢) - VB2 (20 ¢) = 0 2 VB2 = 62 k \Sigma Fy = 0; Ay - F k/1 (20 ¢) + 62 k - 12 k = 0 \Sigma M A = 0; + Ay = 50 k FBD "BCD" + \Sigma M D = 0; \Sigma Fy = 0; + 5k/1 (30 ¢) - 62 k - 5k (30 ¢) - 62 k - 5k (30 ¢) + C y + 171.25k = 0 Dy = 40.75k 5-23 Copyright © 2018 McGraw-Hill Education. Compute the deflection at midspan and the slope at A in Figure P8.19. G P12.30. Determine the forces or components of force in all bars of the trusses in Figure P4.49. Concrete Frame X = 0.9 Reinf.$

Compute all reactions. B A C w = 9 kN/m 6m 6m P5.32 Reactions Resultant of distni loads each side of ... £ 1 R = 6 ' 9 = 27 kN 2 + Σ M A = 0; 27 ' 4 - 27(8) + 12 RC = 0 RC = 9 kN , RA = 9 kN 5-34 Copyright © 2018 McGraw-Hill Education. at Joint A Equil. w = 6 kN/m P6.8. Compute the support reactions and the maximum tension in the cable in Figure P6.8. B 4m A 2m 12 m P6.8 Entire Structure: + ≈ 6 kN \ddot{o} + (24) + H (4) = 0 Σ M A = 0; cc \dot{c} \dot{c} m $\div \dot{\sigma}$ 2 H = -432 + 6 By 2 (1) FBD "CB": + 6(12)2 = 0 2 H = -108 + 3By Σ MC = 0; 4 H - By (12) + (2) Subst. (c) Locate the point of zero shear between B and C. 47 Ay , Max.

1 P16.3. In Problem P16.2, find the force in the spring located at B if beam ABCD supports a downward uniform load w along its entire length. P5.1. Write the equations for shear and moment between points B and C as a function of distance x along the longitudinal axis of the beam in Figure P5.1 for (a) origin of x at point A, and (b) origin of x at D. Consider only the bending deformations of members AB and BC and the axial deformation of CD. (a) In Figure P6.26 compute the horizontal reaction Ax at support A for a 10-kip load at joint B., and E = 9000 2 kips/in. Express answer in terms of E, I, and L.

above its intended position. ED = 48 kips (T) Node $E: \Sigma Fx = FE + 48 = 0$ FE = 48 kips (T) Node $G: \Sigma Fy = AG = 0$ AG = 0 kips $\Sigma Fx = -64 + GF = 0$ GF = 64 kips (T) Node $G: \Sigma Fy = AG = 0$ AG = 0 kips $\Sigma Fx = -64 + GF = 0$ GF = 64 kips (T) Node $G: \Sigma Fy = -53.67$ sin (26.57) = 0 BC = -53.67 kips (C) 4-11 Copyright © 2018 McGraw-Hill Education. Determine the vertical and horizontal displacements at A of the pin-connected structure in Figure P9.31. A B 8m C 2m P9.2 $\Sigma Fy = 0$: RA + RB - 4(10) = 0 ≈ 10 $\circ \Sigma MA = 0$: MA + 8RB - 4(10) $\varsigma \div = 0$ $\varsigma \div = 0$ $\varsigma \div 2 \div \emptyset$ AA = 18.5 kN() A = 28 kN.m() (+) Deflection at C Select RB as the Redundant M (x) = -2 x 2 + 18.5 x - 28 By integration method with boundary conditions ($\delta(0) = 0$, $\theta(0) = 0$) $\circ 1 \approx 2$ 18.5 2 $\theta(x) = c_{\tau} \times 3 + x - 28 \times \div \div \div EI$ es $3 2 \ \emptyset$ At support B, $\theta B = 0.00178$ rad. 8 kips w = 3 kips/ft G' C A E D x 5' 3' 10' P5.2 Σ M A = -6(8) + 13(3 \cdot 10) -18E y = 0 8 kips E y = 19 kips B 3 kips/ft $\Sigma Fy = Ay - 3 \cdot 10 + E y = 0$ Ay = 11 kips $\Sigma Fx = E x - 8 = 0$ C E x = 8 kips D Ay Ey $\Sigma Fy = Vx - 3(10 - x) + 19 3$ kips/ft Vx Vx = 3 x + 11 kips $\approx 0 \times \Sigma M A = M x + \varsigma \varsigma \div 3 \times -19(x) = 0$ $\varsigma \div 2 \div \emptyset Mx D 3 M x = 19 x - x 2$ kip $\cdot ft 2 x 10$ -x Ey 5-3 Copyright © 2018 McGraw-Hill Education.

B A I = 240 in. 2 (a) Compute the stiffness coefficient associated with a 1-in. 10'' P5.46.

Assume I = 0.5IG. BA () C ö 1 240 æç 2 8 ç 8 + 12÷÷ ø 2 EI çè 3 16,640 = (1728) = $\Delta CO = -7.188$ in. 4' 200 kip • ft 170 kip • ft 170

Sketch the deflected shape hinges at B and C. H 20' 10' A B C D E F 6 @ 20' = 120' P12.33 0.8 0.667 0.559 0.6 Member Axial Force (kips) 0.373 0.4 0.186 0.2 0.083 8 ft 0 -0.2 -0.167 -0.186 -0.373 -0.4 -0.083 -0.25 -0.6 Member HD -0.8 0 20 40 60 80 100 Location of Unit Force (ft) Maximum tension force in HD: æ1 ö HDT, max = 0.32 çç 0.559(60 + 12) ÷ ÷ + 13(0.559) = 13.71 kips èç 2 ø Maximum compression force in HD: æ1 ö HDT, max = 0.32 çç 0.373(40 + 8) ÷ + 13(0.373) = 7.71 kips èç 2 ø Maximum compression force in HD: æ1 ö HDT, max = 0.32 çç 0.373(40 + 8) ÷ + 13(0.373) = 7.71 kips èç 2 ø I2-36 Copyright © 2018 McGraw-Hill Education. 10 kips 10 kips C D B I E F H hinge 30' Ax 20' A G 40' 40' P6.26 Vertical reactions can be found. Determine the horizontal and vertical deflection of the hinge at point C of the arch in Figure P8.16 for a single concentrated load of 60 kips applied at joint B in the vertical direction. P8.32. Compute all reaction, draw the shear and moment curves for the beam, and sketch the deflected shape of the structure. Draw the shear and moment curves for the column in Figure P5.46. C D E B F 5' 15' - 00' P12 20 12 22 Converted to 2018 McGraw Hill Education. P8.32 Compute all reaction, draw the shear and moment curves for the beam, and sketch the deflected shape of the structure. Draw the shear and moment curves for the column in Figure P5.46. C D E B F 5' 15' - 00' P12 20 12 22 Converted to 2 2 k + C y = 0.6 k c

, and E = 29,000 kips/in. The deck and tower making up the two-span, cable-stayed bridge in Figure P6.16 are constructed of reinforced concrete. B P10.17. Given: E = 30,000 kips/in. 1728 Select RDX as Redundant Use Moment-Area To Compute $\theta = \theta \pm + \Delta \theta \pm$ Released Structure P-System B 2 & L wL2 $\ddot{o} \div wL3 \theta = 0 + ccc$ $\dot{\tau} \div = 3 ce^2 2 8EI \phi \div 24$ EI -16 k (48 ft)3 (144) in 2 $\theta = 61 \pm 30,000$ kips/in. 1728 Select RDX as Redundant Use Moment-Area To Compute $\theta = \theta \pm + \Delta \theta \pm$ Released Structure P-System B 2 & L wL2 $\ddot{o} \div wL3 \theta = 0 + ccc$ $\dot{\tau} \div = 3 ce^2 2 8EI \phi \div 24$ EI -16 k (48 ft)3 (144) in 2 $\theta = 61 \pm 30,000$ in 4 in 2 $\theta = 0.00983$ Radians 24 $\dot{\tau} 30,000$ in 2 ft 2 Q-System $\Delta = \theta \pm 9.5662$ in. To reduce the vertical displacement of the roadway floor system of the arch (shown in P6.32, part b) produced by the 48-kip load at joint 18, diagonal cables of 2 in. = Vmin. $\Delta A = 0 = -4.571 + 60 + 5.571 + 60 + 5.571 + 60 + 5.571 + 60 + 5.571 + 60 + 5.571 + 60 + 5.571 + 60 + 5.571 + 60 + 5.571 + 60 + 5.571 + 60 + 5.571 + 60 + 5.571 + 60 + 5.571 + 60 + 5.571 + 60 + 5.571 + 6.571$

P B 30' rod A C turnbuckle 80' 80' P6.20 Equilibrium Joint C: $\Sigma Fx = 0 = -T + 750 \cos(20.56)$ 750 kips T = 702.25 kips 20.56 o $\Sigma Fx = 0 = -C y - 750 \sin(20.56)$ T C y = 263.39 kips Cy = 0.5P By symmetry: Ay = C y = 263.39 kips The rod size is found as: Areq , rod = T = 21.95 in 2 32 ksi 4 Areq , rod = T = 21.95 in 2 32 ksi 4 Areq , rod = T = 21.95 in 2 32 ksi 4 Areq , rod = 5.29 in. 18 kN w = 12 kN/m 3m A C B 8m 6m P14.4 Equibrium at Joint 2 xL2 12 '82 FEM12 = -64 + θ 2 M12JD = -64 Moment in Clamp = 64 - 13.5 = 50.5 kN · m 43.286 æç EI ÷ö ç- ÷ EI çè 4 ÷ø Moments due to Unit Rotation at 2 = -74.82 kN · m M21 = $64 + \theta 2$ M 21JD 2 EI 2 EI EI =(-1) = L 8 4 2 EI 4 EI EI JD = M23 = (2[-1]) = L 6 3 2 EI 2 EI EI JD = M32 = (-1) = L 6 3 EI æç 2 EI ÷ö 7 EI JD JD K 2 = M21 + M 23 = -74.82 kN · m M21 = $64 + \theta 2$ M 21JD 2 EI 2 EI EI =(-1) = L 8 4 2 EI 4 EI EI JD = M23 = (2[-1]) = L 8 2 2 EI 4 EI 2 EI JD = M23 = (2[-1]) = L 6 3 2 EI 2 EI EI =(-1) = L 6 3 2 EI 2 EI EI JD = M23 = (-1) = L 6 3 2 EI 2 EI EI =(-1) = L 6 3 2 EI 2 EI 2 EI EI =(-1) = L 6 3 2 EI 2 EI EI =(-1) = L 6 3 2 EI 2 EI EI =(-1) = L 6 3 2 EI 2 EI EI =(-1) = L 6 3 2 EI 2 EI EI =(-1) = L 6 3 2 EI 2 EI EI =(-1) = L 6 3 2 EI 2 EI EI =(-1) = L 6 3 2 EI 2 EI EI =(-1) = L 6 3 2 EI 2 EI EI =(-1) = L 6 3 2 EI 2 EI EI =(-1) = L 6 3 2 EI 2 EI EI =(-1) = L 6 3 2 EI 2 EI EI =(-1) = L 6 3 2 EI 2 EI 2 EI EI =(-1) = L 6 3 2 EI 2 EI 2 EI EI =(-1) = L 6 3 2 EI 2 EI 2 EI EI =($ce^3 + a_6 M12JD = 43.286 ec^2 EI + ce^2 + e^2 + e^2$ BK, BC, and LK if the live load is applied to the truss in Figure P12.30 through the lower chord. B B C 6m A 9m P13.3 If both ends of girder BC are fixed: FEM = $48.6 \text{ kN} \cdot \text{m}$ M BA = $-48.6 \text{ kN} \cdot \text{m}$ M BA = $M AB = + VCB : \Sigma M B = 0.9 \ 108' + 97 - 48.6 - VCB = 0.2 \ VCB = 59.98 \ kips + \Sigma Fy = 0 = VBC + 59.38 - 108 \ VBC = 48.62 \ kN \ Moment at base of column = 1 \ (48.6) \ 2 \ If \ I \ BC = 8I \ AB \ IC \ / \ DC = 0.1875 \ 8 \ IC \ / \ 9 \ Read in \ Fig \ 13.6 \ M \ BC = 0.15 \ FEM = .15(81) = 12.2 \ kN \ \cdot m + \Sigma M \ A = 0 = 48.62 + 24.31 - VB \ 6 \ VB = 12.16 \ kN \ 13-5 \ Copyright \ \odot \ 2018$ McGraw-Hill Education. (3) & (4) into EQ's 1 PL 100(6) == -75, FEM BA = 75 kN · m 8 8 Member end Moments FEM AB = - ù 2 EI (2 ′ 0.005 + 0B) - 75 kN/m ú ú 6 ú 2 EI ú MBA = (2 0B + 0.005) + 75 ú 6 ú EQ 1. (b) If the beam carries a uniformly distributed live load of 2 kips/ft that can act on all or part of each span as well as a concentrated live load of 20 kips that can act anywhere, compute the maximum moment at B. Practical Design Example The tall building in Figure P9.49 is constructed of structural steel. Continued y P@¢0 @A @B 35° a' x2 x1 FBD "BC" FBD "FC" 50 Sin 50 Sin 6 50 Cos 6 28.68' 40.96-X1 (0-90) Cx - Cy ↑ VF NF 40.96' 0 0 0 0.09P -0.127P 0 MF 0 45° 35.36' 3 $5.6\ 0.09P\ 60^{\circ}\ 43.3'\ 25'\ 15.96\ 0.257P\ 0.257P\ -0.362P\ +0.004P\ 75^{\circ}\ 48.3'\ 12.94'\ 28.02\ 0.45P\ +0.0063P\ -0.362P\ -0.362P\ +0.0063P\ -0.362P\ -0.362P\ +0.362P\ -0.362P\ +0.362P\ +0$ -0.913P -0.512P +6.464 @F 135.57° 35' @C 53.9 -6.44 150° 25' 43.3 84.26 0.092P 0.958P 0 (P is below PT.F) -0.08P +3.78 165° 12.94' 48.3 90.72 0.05P 0.998P 0 -0.022P Appr. Continued Forces at Each Floor Level Wx hxk Floor Roof Weight Wi, (kips) Floor Height Wi (the start floor Level Wx hxk Floor Roof Weight Wi, (kips) Floor Height Wi (the start floor Level Wx hxk Floor Roof Weight Wi, (kips) Floor Height Wi (the start floor Level Wx hxk Floor Roof Weight Wi, (kips) Floor Height Wi (the start floor Level Wx hxk Floor Roof Weight Wi, (kips) Floor Height Wi (the start floor Level Wx hxk Floor Roof Weight Wi, (kips) Floor Height Wi (the start floor Level Wx hxk Floor Roof Weight Wi, (kips) Floor Height Wi (the start floor Level Wx hxk Floor Roof Weight Wi, (kips) Floor Height Wi (the start floor Level Wx hxk Floor Roof Weight Wi, (kips) Floor Height Wi (the start floor Level Wx hxk Floor Roof Weight Wi, (kips) Floor Height Wi, (kips) Floor Height Wi (the start floor Level Wx hxk Floor Roof Weight Wi, (kips) Floor Height Wi, (k 76.1 th 810 36 45,638 0.213 56.0 5 4 rd 3 810 24 28,922 0.135 34.8 2nd 810 12 13,261 0.062 16.0 3,870 213,961 2-20 Copyright © 2018 McGraw-Hill Education. E B C D 12' A F 5' 10' 5' P3.25 + Σ Fy = 0 of free-body DEF: -10 -10 + FY = 0 FY = 20 kips + Σ M D = 0; 10 $\stackrel{?}{2.5}$ - 20 $\stackrel{?}{5}$ + Fx (12) = 0 Fx = 6.25 kips ¬ Consider the entire structure as a construction. free-body. 100 kN P11.31. A B 9' C 6' P9.1 E = 29,000 ksi, I = 150 m 4 δ CC = PL3 1(153) 1728 = 3EI 3' 29,000 '150 δ CC = 0 0.25 + XC 0.447 = 0 XC = 0.559 kips 9-13 Copyright © 2018 McGraw-Hill Education. P14.7. Continued Joint Equilibrium EQ: $\Sigma M 2 = 0.12 \text{ kN} + K 2\theta = 0.5 - 12 \text{ kN} + EI \theta = 0.3 - 36 \theta = 5EI \text{ Member}$ and Movements M12 = $0 + \theta 2$ M12JD = $-36 \approx c$ EI + $\ddot{o} 18 c$, + = + = $3.6 \text{ kN} \cdot \text{m} 5\text{EI} c$ 2 + $\phi 5 - 36 (-\text{EI}) = 7.2 \text{ kN} \cdot \text{m} 5\text{EI} c$ 2 + $\phi 2$ M23JD = $-72 + \phi 2$ M2 Continued (a) 6 Reactions –3 Equations of Equilibrium -1 Equation of Condition (MHINGE = 0) \ Indeterminate 3º (b) Reactions Form A Parallel Force System : Unstable Remove Restraints: 1@A; 1@B; 1@C; 3@ Cut@D \ Indeterminate 6º (d) 5 ft VC = -3 kips 5 ft By = -3 kips 5 ft By = -3 kips 5 ft By = 18 kips 15 15 Shear (kips) -3 75 Moment (kip-ft) -75 Deflected Shape POI POI 5-20 Copyright © 2018 McGraw-Hill Education. , 2 2 AAC = 4 in. If windows are inundated, calculate the expected hydrostatic loading on the adjacent outside walls due to water retained by the floor, or floors. B C D E F 15' A M L K J I 7 @ 15' = 105' P12.35 12-38 Copyright © 2018 McGraw-Hill Education. $\Sigma Fy = 0$; + - 20 k + FABY + FBDY = 0 FBDY = 15k tension By observation FBC = 20 k compr. How far should support B and the midspan of the beam. B 30 kN • m 4m w = 5 kN/m D C 3m 3m 6m P5.25 Link BC Carries Axial Load Only. Given: 2 6 4 E = 200 GPa, A = 500 mm , I = 200 × 10 mm . D 4m C E 4m B F I 4m A G H 4m 4m P4.20 Compute Reactions + $\Sigma M A = 0$; Consider a Freebody of Joint H 100(4) - Gy B = 0 \ Gy = 50 kN + $\Sigma Fy = 0$; -100 + Ay + Gy = 0 Ay = $100 - Gy = 50 kN \Sigma Fx = 0$; Ax = 0 Note: Since structure and load are symmetrical, the forces are equal in corresponding members on either side of the centerline Joint A Because of symmetry the forces F in bars BH and FH are equal in corresponding members on either side of the centerline Joint A Because of symmetry the forces F in bars BH and FH are equal in corresponding members on either side of the centerline Joint A Because of symmetry the forces F in bars BH and FH are equal in corresponding members on either side of the centerline Joint A Because of symmetry the forces F in bars BH and FH are equal in corresponding members on either side of the centerline Joint A Because of symmetry the forces F in bars BH and FH are equal in corresponding members on either side of the centerline Joint A Because of symmetry the forces F in bars BH and FH are equal in corresponding members on either side of the centerline Joint A Because of symmetry the forces F in bars BH and FH are equal in corresponding members on either side of the centerline Joint A Because of symmetry the forces F in bars BH and FH are equal in corresponding members on either side of the centerline Joint A Because of symmetry the forces F in bars BH and FH are equal in corresponding members on either side of the centerline Joint A Because of symmetry the forces F in bars BH and FH are equal in corresponding members on either side of the centerline Joint A Because of symmetry the forces F in bars BH and FH are equal in corresponding members on either side of the centerline Joint A Because of symmetry the forces F in bars BH and FH are equal in corresponding members on either side of the centerline Joint A Because of symmetry the forces F in bars BH and FH are equal in corresponding members on either side of the centerline Joint A Because of symmetry the forces F in bars BH and FH are equal in corresponding members on either sin the magnitude and sense assume tension. Construction of an influence line for an indeterminate beam. Computer analysis of a truss with rigid joints. P13.2. Guess the location of the points of influence line for an indeterminate beam. As a result, the arch undergoes less deformation and deflections of the floor system are significantly reduced. Draw free-body diagrams of each member and determine the forces applied by the pins to the members. This condition has resulted in the collapse of flat roofs.

(b) Compute the horizontal base shear and the overturning moment of the building. Use an approximate analysis of the continuous truss in Figure P13.11 to determine the reactions at A and B. If a uniform load of 2 kips/ft is applied over the entire length of the beam, compute the reactions at A and B. If a uniform load of 2 kips/ft is applied over the entire length of the beam, compute the reactions at A and B. If a uniform load of 2 kips/ft is applied over the entire length of the beam, compute the reactions at B, C, D and E, moments at C and D. What value of jacking force T must be applied at supports B and C to tension the system? E P9.34. 3 kips P5.18. 2(36,000 ft $2 \cdot k$) 20 k (18¢) 2 $\theta A + \theta B - 3(-0.01) = 8000 \theta A + 4000 \theta B + 75 18$ ¢ 8 2 2(36,000 ft $2 \cdot k$) (2 $\theta B + \theta C$) M BC = = 6000 $\theta B + 2000 \theta B + 2000 \theta B + 2000 \theta B + 2000 \theta B + 4000 \theta B + 75 18$ ¢ 8 2 2(36,000 ft $2 \cdot k$) (2 $\theta B + \theta C$) M BC = = 6000 $\theta B + 2000 \theta B + 20000 \theta B + 2000 \theta B + 2000 \theta B + 20000 \theta B + 2000 \theta B + 2000 \theta$ $2(72,000 \text{ ft } 2 \cdot \text{k}) (2 \theta \text{B}) 12 \text{ c} 2(72,000 \text{ ft } 2 \cdot \text{k}) \text{ M EB} = (\theta \text{B}) 12 \text{ c} \text{ Joint A M BE} = 24,000 \theta \text{B} + 2M \text{A} = 0; \text{ M AB} = 0.000375 - 0.5 \theta \text{B} \text{ Joint B} + 2M \text{B} = 0; \text{ M BA} + \text{M BC} + \text{M BE} = 0.38000 \theta \text{B} + 4000 \theta \text{A} + 165 = 0 \text{ Substitute } \theta \text{ A from Above, Gives: } 36,000 \theta \text{B} + 127.5 = 0 \theta \text{B} = -0.003542 \theta \text{A} = -0.003542 \theta \text{A} = -0.003542 \theta \text{A} = -0.00375 - 0.5 \theta \text{B} \text{ Joint B} + 2M \theta \text{B} = 0; \text{ M BA} + M \theta \text{C} + M \theta \text{B} = 0.38000 \theta \text{B} + 4000 \theta \text{A} + 165 = 0 \text{ Substitute } \theta \text{ A from Above, Gives: } 36,000 \theta \text{B} + 127.5 = 0 \theta \text{B} = -0.003542 \theta \text{A} = -0.003542 \theta \text{A} = -0.003542 \theta \text{A} = -0.00375 - 0.5 \theta \text{B} \text{ Joint B} + 2M \theta \text{B} = -0.00375 - 0.5 \theta \text{B} \text{ Joint B} + 2M \theta \text{B} = -0.003542 \theta \text{A} =$ $-0.0076\ 10-17\ Copyright$ © 2018 McGraw-Hill Education. $\theta B = 89.74\ 176.77\ 154.88\ \theta C = \Psi AB = EI\ EI\ EI\ Shears$ in Legs + $\Sigma M\ A = 0\ -71.29\ -93.72\ +VBA\ 8 = 0\ VBA = 20.68\ kN\ +\Sigma M\ D = 0\ -88.67\ -147.58\ +VCD\ 6 = 0\ VCD = 39.38\ kN\ VDC = 39.38\ kN\ +\Sigma M\ B = 0;\ 71.29\ +88.67\ -VCB\ 10 = 0\ VCB = 16\ kN\ VBC = -VCB = 16\ kN\ VBC = -VCB = 16\ kN\ VBC = -VCB$ kN Shear in Girder 10-37 Copyright © 2018 McGraw-Hill Education. (a) Use approximate analysis to compute the reactions and draw the moment diagrams for column AB and girder BC of the frame in Figure P13.20, and draw the deflected shape. Given: AE = constant, 2 A = 1000 mm , and E = 200 GPa. 60 kN 60 kN B C A 4m E D 100 kN 3m 3m 3m P9.29 AE = Constant A = 1000 mm2 E = 200 GPa Δ DH (1 kN) = Σ FQ FP L Select DX as the Redundant AE ii(-50)(-0.667)(6) + (+116.7)(-0.556)(3)i + (+116.7)(-0.556)(3)i + (+116.7)(-0.556)(3)i + (+116.7)(-0.556)(5) + (+116.7)(-0.56)(-0.5 Compatibility Equation Δ DH + δDD DX = 0 \ DX = -30.9 kN() Q-System for ΔDH P-System for Δ machine. Continued 1k · $\Delta Cy = \Sigma$ FQ FP L + $\Sigma \delta$ MQ M P dx AE EI 1 é 1 · $\Delta Cy = (-46.55)(-0.763)(48.78^{12}) + (-22.18)(-0.739)(32.14^{12}) AE e + (-22.9)(-0.763)(48.78^{12}) + (-22.9)(-0.763)(48.78^{12}) + (-22.18)(-0.739)(32.14^{12}) AE e + (-22.9)(-0.763)(48.78^{12}) + (-22.18)(-0.739)(32.14^{12}) AE e + (-22.9)(-0.763)(48.78^{12}) + (-22.$ 0.25 x) dx + ò dx ò EI EI 0 0 -90, 792(12)3 40,600 + = 0.068¢¢ - 8.71¢¢ $\Delta Cy = 20$ '30,000 30,000 '600 = -8.6 ¢¢ + Horizontal Displacement Q FAB = 0.42 sin 52 + 0.5cos 52 = 0.64 k; M AB = 0.1354 x Q FBC = 0.42 sin 21 + 0.5cos 21 = 0.62 k Q M BC = 6.61 - 0.206 x Q FDE = -0.42 sin 52 - 0.5cos 52 = 0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M BC = -0.42 sin 52 + 0.5cos 52 = 0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M BC = -0.42 sin 52 - 0.5cos 52 = 0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M BC = -0.42 sin 52 - 0.5cos 52 = 0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M BC = -0.42 sin 52 - 0.5cos 52 = 0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M BC = -0.42 sin 52 - 0.5cos 52 = 0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M BC = -0.42 sin 52 - 0.5cos 52 = 0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M BC = -0.42 sin 52 - 0.5cos 52 = 0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M BC = -0.42 sin 52 - 0.5cos 52 = 0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M BC = -0.42 sin 52 - 0.5cos 52 = 0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M BC = -0.42 sin 52 - 0.5cos 52 = 0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M BC = -0.42 sin 52 - 0.5cos 52 = 0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M BC = -0.42 sin 52 - 0.5cos 52 = 0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M BC = -0.42 sin 52 - 0.5cos 52 = -0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M BC = -0.42 sin 52 - 0.5cos 52 = -0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M BC = -0.42 sin 52 - 0.5cos 52 = -0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M BC = -0.42 sin 52 - 0.5cos 52 = -0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M BC = -0.42 sin 52 - 0.5cos 52 = -0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M BC = -0.42 sin 52 - 0.5cos 52 = -0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M BC = -0.42 sin 52 - 0.5cos 52 = -0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M BC = -0.42 sin 52 - 0.5cos 52 = -0.64 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M DE = -0.1354 x Q FCD = -0.62 k Q M DE = -0.1354 x Q FCD k MCD = -6.61 + 0.206 x Horizontal Displacement of Joint C FP L dx + Σ ò MQ M P AE EI Δ CH ·1k = Σ FQ Δ CH = 1 é -46.55(0.64)(48.78¹2) + (-11.23)(0.62)(32.14¹2) at + EI 32.14 ò 0 (-0.1354 x) dx + EI 32.14 ò 0 (-0.1354 x) (-4.95 x) dx + EI 0 -6240 160,512.5(12)3 = + 20'30,000'600 = -0.0104 ¢¢ + 15.41¢¢ (6.61 - 0.206 x)(659.5 - 20.52 x) dx EI 32.14 ò 0 (-6.61 - 0.206 x)(-241.46 + 7.51x) dx EI = 15.4 in *As you can see in computations of deflections, the effects of axial load are very small compared to the defections due to moment; Therefore, we typically neglect the effect of axial load when analyzing beams, frames, and arches that do not support a funicular loading. B P5.2. Write the equations for shear and moment between points D and E. Determine the horizontal and vertical components of force that the support must apply to the bars. Assume member BD acts as an axially loaded pin-end compression strut (a) In Figure P8.15 compute the vertical and horizontal components of displacement of joint E produced by the loads. Computer analysis of beam of varying depth. A B 3I 12' C I D 21' 12' P11.16 3 3 I 3I 9 I = = 4 21 28 84 I I 7I = = = 12 12 84 16 I \Sigma K S = 84 K AB = K BD D.F. 3m P4.19. i.e M = 0 & V = 0 Compute Forces on a Normal Section at "D' Freebody of "DE": Determine Moment at D Σ MO = 0; 20 k (11.25¢) - 30 k (10 ¢) + M D = 0 M D = 75 kip.ft Shear VD = 24 k - 12 k = 12 k Axial Force FD = 16 k + 18k = 34 kips compression 6-26 Copyright © 2018 McGraw-Hill Education. 9¢ = 4.5 = 54 ¢¢ 2 5wL4 5(0.417)(120)4 1728 = 1.15 in 384 EI 384 ′ 29,000 ′ 58320 Double Δ to account for contribution of web members. Ans. Evaluate influence line ordinates at 8-ft intervals in span AC and CD and at E. 6-42 Copyright © 2018 McGraw-Hill Education. ; 2 for the diagonal brace, A = 4 in. 12 P6.33. Assume max moment under Wheel C Mmax = 20.5k '14.889 = 305.3 kip · ft 4. B w = 2.4 kips/ft C D A 6' 10' 8' P5.6 Origin at "A" A-B B-C C-D Origin at "D" éx ù M1 = 39.1x1 - 2.4 x1 ê 1 ú ê2ú ë û = 39.1x1 - 1.2 x12 éx ù M2 = 39.1x2 - 2.4 x2 ê 2 ú - 18(x2 - 6) ê2ú ë û = 21.1 + x2 - 1.2 x22 + 108 C-D M 4 = 17.3 x 5 - (x5 - 8)2 M6 = -1.2(x5 - 8)2 + 0.7 x6 + 324 B-A M3 = 39.1x3 - 18(x3 - 6) - 2.4(x5 - 8)2 M6 = -1.2(x5 - 2.4 - 16(8 + x3 - 16) M3 = 415.2 -17.3 x3 Check Moment at "C" M 4 = 17.3 × kips C 12' A 18' P9.44 Refer to solution for P11.27 Compatibility Equation $\Delta CO = -0.36$ ¢¢ $\Delta CO + \Delta C$ Due to a + δCC RC = -0.36 ¢¢ 30000(60) 30000(40) RC = 0.78k 9-49 Copyright © 2018 McGraw-Hill Education. Evaluate the ordinates at 15-ft intervals. Determine the shape of cable subjected to the loading shown. in diameter are added as shown in Figure P6.34. 18 kN w = 4 kN/m A B 2I 3m C hinge 6m D I 6m 3m P9.12 Evaluating the integral: $\Delta CO = -549$ EI Indet to 1st Degree Compute δcc . B Final Results (kips) -18.6 (C) 0 A Ay = 9.3 -12.4 (C) By = 18.6 (C) members of the truss are 20 in. ; 4 2 for the girders, I = 800 in. B A C I 2I 6' 9' P9.5 Case I Variable EI Use Moment-Area to Compute deflections $\Delta CO = EI 1 \approx 9\ddot{o} 4.5$ $(6(9 + 3) \delta CC = EI 2 \approx 0.5)$ Compatibility equation $\Delta CO = EI 1 \approx 9\ddot{o} 4.5$ $(6(9 + 3) \delta CC = EI 2 \approx 0.5)$ $\Delta C = 0 = \Delta CC + \delta CC RC 4590 684 + O = R EI EI C RC = -6.71 kips$ MA = 60 - 6.71'15¢ = 40.65 ft · kips RA = 6.71 kips Case I Joint Constant EI This example shows that increasing the stiffness of an indeterminate structure in the region between points A & B produces a 35% increase in the moment at support A. A P9.39. Continued Case 2 Case 1 Joint Reactions Joint Reactions Joint Label X [k] Y [k] Joint Label X [k] Y [k] Joint Label X [k] Y [k] N1 -41.304 H2 49.755 H3 30.978 M3 37.316 M4 -51.83 M4 -62.193 M5 -75 N4 -58.696 75 N4 73.37 M5 62.807 Case 3 Joint Reactions Joint Label X [k] Y [k] N1 -28.296 -75 N4 -70.704 75 Totals: -100 0 Member Label Axial [k] M1 -53.028 M2 29.298 M3 21.872 M4 -36.62 M5 88.38 4-58 Copyright © 2018 McGraw-Hill Education

Use both the method of introducing unit displacements and the member stiffness matrix of Equation 16.36. Advanced embedding details, examples, and help! Something went wrong. and I = 3.16 in. 10' 17' 20' 100 kips 100 k

30 kips P13.10. Also compute the maximum deflection produced by the load. The effectiveness of this strengthening scheme is demonstrated in P6.32. B 20° C D 80 kips 15' 15' P8.11 T 6BD Due to DT = +60 F T 6BD = $\alpha \cdot \Delta T \cdot L = 6.5'10-6$ (60)(25'12) = 0.117 in Bar deformation due to 80 k load at "C" PL -60(15'12) = = -0.093¢¢ AE EA 80(20'12) $\Delta LBC = = 0.1655¢¢ EA - 100(25'12) \Delta LBD = = -0.2586¢¢ EA - 400(25'12) \Delta LBB = = -0.2586¢¢ EA - 400(25'12) = 0.117$ in Bar deformation due to 80 k load at "C" PL -60(15'12) = = -0.093¢¢ AE EA 80(20'12) $\Delta LBC = = 0.1655¢¢ EA - 100(25'12) \Delta LBD = = -0.2586¢¢ EA - 400(25'12) = 0.117$ in Bar deformation due to 80 k load at "C" PL -60(15'12) = = -0.093¢¢ AE EA 80(20'12) $\Delta LBC = -0.2586¢¢ EA - 100(25'12) \Delta LBD = = -0.2586¢¢ EA - 400(25'12) = 0.117$ in Bar deformation due to 80 k load at "C" PL -60(15'12) = = -0.093¢¢ AE EA 80(20'12) $\Delta LBC = -0.1655¢¢ EA - 100(25'12) \Delta LBD = -0.2586¢¢ EA - 400(25'12) = 0.117$ in Bar deformation due to 80 k load at "C" PL -60(15'12) = = -0.093¢¢ AE EA 80(20'12) $\Delta LBC = -0.2586¢¢ EA - 100(25'12) \Delta LBD = -0.2586¢¢ EA - 0.117)$ if $\dot{e} \delta \dot{u} \Delta LAB = -0.2586¢¢ EA - 0.117)$ if $\dot{e} \delta \dot{u} \Delta LAB = -0.2586¢¢ EA - 0.117)$ if $\dot{e} \delta \dot{u} \Delta LAB = 0.258¢¢ EA - 0.117)$ if $\dot{e} \delta \dot{u} \Delta LAB = 0.258¢¢ EA - 0.117)$ if $\dot{e} \delta \dot{u} \Delta LAB = 0.28¢¢$ is a LAB $\dot{e} \dot{v} \Delta LAB = 0.28¢¢$ Shorter (c) δCV use P-System he magnitude in corresponding spans is similar and one can have confidence the exact analysis by Moment Distribution. Since the top chord of the truss carries the largest forces, these members would be the most impactful towards reducing the overall weight. (With no load, move B' 0.47" to the left) æ + 5 & 0 WQ = UQ + 0.14 (0.47 ¢¢) = 0.25 ¢¢ \Delta LAB = 0.28¢¢ Shorter (c) \delta CV Use P-System as Q-System WQ = UQ 406Cv = $\Sigma PQ \cdot \Delta L = \Sigma PQ (\alpha t \Delta TL) k 1 e \delta CV = 0.29 ¢¢ 12 e 3 \phi c DY = 0.29 ¢¢ 12 e 3 \phi c DY = 0.29 ¢¢ 12 e 3 \phi c DY = 0.29 ¢¢ 12 e 3 \phi c DY = 0.29 ¢¢ 12 e 3 \phi c DY = 0.29 ¢¢ 12 e 3 \phi c DY = 0.29 ¢¢ 12 e 3 \phi c DY = 0.29 ¢¢ 1$

Where two values of Cp are listed, this indicates that the windward roof slope is subjected to either positive or negative pressure, and the roof structure should be designed for both loading conditions. 8 kips 15 kip • ft 5' B F 5' D E 4' 5' 6 kips A 10' P3.2 + Σ M A = 0; 15 ft · k + 8k (5¢) + 6 k (4¢) - C y (10¢) = 0 C y = 8.5k Σ Fy = 0; + -8k - 6 k + 8.5k + Ay = 0 Ay = 5.5k + Σ Fx = 0; Ax = 0 3-3 Copyright © 2018 McGraw-Hill Education. Continued Dy = 4.1 kips Dx = 1.3 kips 9.3 D D D 1.3 64.8 B C B C B - 35.2 C - 16.7 - 4.1 - 1.4 Shear (kips) A Ax = 1.3 kips Deflected Shape Moment (kip-ft) A A Ay = 4.1 kips 10-35 Copyright © 2018 McGraw-Hill Education. The cable in Figure P6.11 is capable of carrying a tensile load of 160 kips and the pin supports are capable of providing a horizontal reaction of 150 kips. Computer study—Influence of supports on frame behavior. The load can only be applied between points B and D of the girder shown in Figure P12.21. Use E = 29,000 kips/in. w = 4 kN/m P9.2. For the beam in Figure P9.2, compute the reactions, draw the shear and moment curves, and compute the deflection of the hinge at C.

If indeterminate, indicate the degree. Take the origin at point A. B 15 kips 10 kips D C 10' I J 10' A G H F 4 @ 10' = 40' P4.17 Compute Reactions: $+ \Sigma M A = 0$; 15' 20 + 20' 20 + 10' 40 - 40 E y = 0 E y = 27.5 kips $+ \Sigma F y = 0$; A y = 2.5 kips $+ \Sigma F x = 0$; A x = 15 kips $\neg Determine Zero Bars: AB, CD, FJ, HI, IG, GJ, CG Solve by Method of Joints 4-18 Copyright © 2018 McGraw-Hill Education. Shear At Support wL2 PL 0.4(24)2 10(24) <math>+ = + 8 4 8 4 M = 28.8 + 60 = 88.8$ kip \cdot ft M= At 1 4 RA = 10 + 0.4 (12) = 14.8 k At 1 4 Point Point At Midspan RA = 4.8 + 7.5 = 12.3 k M = 12.3'6 - 2.4 k '3 = 66.6 kip \cdot ft Moment Envelope (kip-ft) Shear Envelope 12-45 Copyright © 2018 McGraw-Hill Education. Draw the shear and moment curves produced by the total dead load for beams B1 and B2. 45 kips 2 2 3 3 1 4 45 kips 4 6 5 1 5 15' 15' P4.53 Case 1: Determine all bar forces, joint reactions, and joint displacements, assuming pin joints. Determine the reactions. Assume the area of the interior columns is twice the area of the maximum stress, using the equation $\sigma = 2 3 2 11 1 9 9 24$ kips 6 5 4 kips 6

Compute FEF $\Sigma M J = 0$; 30(16) + 30(48) - 33.75(64) - FEF (12) = 0 FEF = -20 k Compression 13-15 Copyright © 2018 McGraw-Hill Education. Draw the influence lines for the force in cable CE, the vertical reaction at support A, and the moment at B.

Computer investigation of wind load on a building frame. ; 2 E = 30,000 kips/in. 8' 12' P9.42 B Selecting Cy as the redundant, the compatibility equation is: X ($\Delta CS + \Delta CO$) + $\delta CC XC = \Delta C = -C K \delta CC = \Delta 1 + \Delta 2 + \Delta 3 \Delta 1 A 4/5\Delta' 3/5\Delta'$ () 8 (12 + 20) $\div \phi \in B = 3(12 + 20) \div \phi \in B = 3(12 +$

P6.32). w = 2 kips/ft P5.27. Determine the member end moment sand draw the shear and moment curves. If unstable, indicate the reason. P9.9. Continued 4 kips/ft MA = 31.5 kip-ft C A B 1.5EI Ay = 18.9 kips EI By = 21.1 kips 18.9 8 B Shear (kips) A C -13.1 13.3 2.166 ft Moment (kip-ft) -8 -31.5 Reaction Magnitude (kips and kip-ft) As I of segment AB increases the moment reaction at A increases while the vertical reaction, By, decreases. Continued Part (c) FIXED COLUMN BASE Three assumptions were required for the fixed base frame. Determine the forces in all bars of the truss. Indicate the spans that should be loaded with a uniformly distributed live load to maximize moment in the column. 5m w = 40 kN/m P10.15.

Determine the location of the 40-kN load such that sags at points B and C in Figure P6.14 are 3 m and 2 m, respectively. A C B P12.55 Influence Line for Moment at A and loading for maximum moment. Draw the shear and moment curves for the frame in Figure P10.6. Given: EI is constant. P = 30 kips D C 12' B E 12' A 9' P8.13 FP L AE \approx 3 \ddot{o} + 60(9'12) \approx 3 $\div\ddot{o}$ (-50)15'12 = cc; $\div + cc$, $\div ccc$, $\div cc$, $\div cc$, $\div cc$, $\div cc$, $\div ccc$, $\div cc$, $\div cc$

For the girder in Figure P12.15, draw the influence lines for the reaction at A, the moment at point B, and the shear between points A and B. 1.0 5° 0.8 roofs with obstructed slippery surfaces 0.6 unobstructed slippery surfaces with thermal resistance, $R \ge 30^{\circ}F \cdot h \cdot ft2/Btu (5.3^{\circ}C \cdot m2/W)$ for unventilated roofs or $R \ge 20^{\circ}F \cdot h \cdot ft2/Btu (3.5^{\circ}C \cdot m2/W)$ for ventilated roofs Cs 0.4 0.2 0 0 30° 60° Roof Slope 90° Roof slope factor Cs with warm roofs and Ct ≤ 1.0 P2.19 qhGCp qhGCp θ h qzGCp qhGCp θ h qzGCp qhGCp θ h qzGCp qhGCp Section (b) P2.13 Sloped Roof Snow Load PS CS pf From Fig. = 0; 5k (9 ¢) - M A = 0; M A = 45ft \cdot k FBD "AB" (b) Pinned Column Bases Assumptions: 1) Shear in Columns are Equal. P3.7. The support at A prevents rotation and horizontal displacement but permits vertical displacement. Continued + M BC + MCB - VBC (3¢) = 0 Σ MC = 0; VBC VBC Segment BC M + MCB (20B + 0.111\Delta x) = BC = EI 3¢ 3¢ = (0.6670B + 0.03703\Delta x) EI Where θ B = -0.011349 Δ x VBC = 0.0294 Δ x EI + Σ Fx = 0; 100 k - VBA - VBC = 0 JT. B 0.474 260 - 260 0 FEM D MDC 260 -130 -61.6 -16.2 -4.3 ... 46.4 FEM C D C D ... MCD 0.474 0.526 -260 130 61.6 16.2 4.3 ... -46.4 260 -260 0 C FEM C D D D ... MBA 3.87 kips S = 7.73 kips 12 ft 3.87 kips C 3.87 kips 46.4 kip-ft 46.4 kip-ft 3.87 kips 3.87 kips B 24 ft C -46.4 kip-ft -46.4 kip-ft 3.87 kips 12 ft A 3.87 ki

Calculate the axial forces produced by the live load in column C1 in the third and first stories. k k 2-22 Copyright © 2018 McGraw-Hill Education. D 4' A P12.11. Wait a moment and try again. B 4I C 12' I A 6' 24' P10.5 WL2 2.8(24)2 == -134.4 kip \cdot ft ü 2 EI BC ü (20B) + FEM BC ï (20B) + FEM BC ï i 4 EI BC 0B ï i = -134.4 ii L ï 2 EI AB 4 EI0B ï i (20B) = M BA = ï 12 ï L ï 2 EI AB 4 EI0B ï i (0 B) = M AB = ï 12 ï L ï 2 EI AB 2 EI0B ï i (0 B) = M AB = ï 12 L ï] FEM BC = - Compute Reactions at A and C + Compute VCB; 0 = 67.2(12) + 162.4 - 78.4 - VCB 24 VCB = 37.1 Equil. P7.16, the nine equally spaced loads of 36 kips approximate a uniformly distributed load that will produce a uniform compressive stress on all cross sections of the arch (see the plot of axial forces and moments on page 6.32B. 1 1 2 1'' L 1 = 12' K2x 4 2 L 2 = 15' 45° 3 P14.3 Δ L1 = 1¢¢; Δ L2 = 0.707 ¢¢ 2 Δ LAE 1¢¢ (2 in)30,000 = = 416.67 kips 12 '12 in L1 0.707(3in 2)30,000 Δ L AE F2 = = 353.5 kips 15 '12 L2 F2 x = 0.707(353.5) = 249.93 kips F1 = F2 y = 0.707(353.5) = 249.93 kips K 2 x = 416.67 + 249.93 = 666.6 kips K 2 y = 249.93 kips 14-4 Copyright © 2018 McGraw-Hill Education. P9.38. Note that allowable compressive stress is lower due to buckling.

Then compute the reactions at A and C. Calculate the hydrodynamic and hydrostatic resultant load and location on the walls ABC and IJKL for Load Cases 2 and 3, due to both inflow and outflow directions. Using Bernoulli's principle in Section 8.8, compute (a) the horizontal and vertical components of the displacement of joint B and (b) the change in slope of member BC. Compute the deflection at B and the slope at C in Figure P8.21. Assume the truck can move in either direction. At alternate joints, floor load is distributed to three points along the arch axis, thereby reducing joint deflections significantly in both the horizontal and vertical directions. Gilbert SOLUTIONS MANUAL CHAPTER 2: DESIGN LOADS AND STRUCTURAL FRAMING 2-1 Copyright © 2018 McGraw-Hill Education. 20 kips P3.26. -12 5 kN/m P5.31. 3 P16.8. Solve Problem P16.7 using the direct summation of global element stiffness matrices. Ignore the weight of the hangers. Given: E = 200 GPa, 2 2 AAB = 1000 mm , and AAC = AAD = 500 mm . Analyze the Vierendeel truss in Figure P11.31 by moment distribution. P = 20 kips hinge A B 10' C 5' D 15' P5.17 5-19 Copyright © 2018 McGraw-Hill Education. (b) Repeat the computations with pinned column bases at A and D.

B 100 k - 0.0107 Δ x EI = 0 Δ x = 0.124 ¢¢ θ B = -0.011349 Δ x = -0.001407 rad. EI is constant. Neglect weight of beam. A 24 kips B 12' F 6' E 12' C D 9' 6' 9' P4.42 + Σ M D = 0 = 24 k '3¢ - FAD 24 FAD = 3 kips tension + Σ M f = 0 = 24 k '12 ¢ - 3k '15¢ - FBC 9 ¢ FBC = 27 kips compr. Floor beams are connected to columns by rigid joints. 2) Mmax = -1.5(12.67)2 + 38'12.67 - 160 = -240.79 + 481.46 - 160 Mmax = 80.67 kip · ft = 60.5 kip · ft 5-13 Copyright © 2018 McGraw-Hill Education. Both structures have same boundary conditions of pinned bases, and both are symmetric in geometric and material properties, the same assumption applied to both: lateral load is divided equally to the columns. The remaining results are consistent with similar truss bar forces for the middle 2/3 of the truss. VPO = -VOP = F1 = 1.714 k æL ö Moments @ ends of beam = V çç BM ÷÷ çè 2 ÷ø æ15¢ ö M PO = MOP = 1.714 k çç ÷÷ = 12.86 ft ·k (Exact 18.8) èç 2 ø Axial force in the beam & shear @ top of column.

Neglect all the applied loads. 5' 10' 10' P5.22 Left Side: Displacement: $F = -kx 5 \Sigma M A = + (15 \cdot 5) - (5) By = 0 2 By = Ay = 37.5 kips 26.25 = -30 \cdot x x = 0.875 in Right Side: \Sigma MC = 150 - 5(37.5) + 10(30) - 10 Dy = 0 x = Dy = 26.25 kips 7 8 ¢¢ \Sigma Fy = -37.5 + C y - 30 + 26.25 = 0 C y = 41.25 kips 15 kips/ft A B C Dy 37.5 3.75 2.5 ft -37.5 2.5 ft -37.5 2.5 ft 46.875 0 Shear (kips) 0 0 Moment (kip-ft) -375 Deflected Shape A B C Dy 5-24 Copyright © 2018 McGraw-Hill Education. L.L. Compr in Bar KJ (2 Trusses) 3 3 (24) - (10) = -21.75 kips 4 8 Load Each Truss = 10.88 kips FKJ = - Max. D 16' P3.28. P2.16. A C B 12' 0.6" D 18' 12' P11.25 I = 240 in4, E = 29,000 kips/in2 wL2 2(2)2 = -24 kip \cdot ft 12 12 FEMCB = 24 kip \cdot ft 6 EIA 6(29,000)240(0.6) FEM BD = -2 = L (12 '12)2 = -1208 in \cdot kip = -101 ft \cdot k D.F. I 240 K AB = = 13.33 = .308 L 18 3 I 3 (240) K BC = = 15.00 = .396 4 L 4 12 43.33 = \Sigma K S¢ FEM BC = - Free Bodies + <math>\Sigma M B = 0 = 6.06 + 24 \cdot 6 - RCy 12 RCy = 150.06 kips VBC = 24 - 12.51 = 11.495 kips 11-32 Copyright © 2018 McGraw-Hill Education. 4 kN B C x 6m A 9m P5.11 Reactions <math>\Sigma Fx = 0$; RAX = 4 kN \neg + $\Sigma M A = 0$; 4 kN '6 + 18kN '3 - RC 9 = 0 RC = 8.67 kN + $\Sigma Fy = 0$; W x 4 = ; x 9 RAY - 18 + 8.67 = 0 RAY = 9.33 kN Wk = 4x 9 Shear + $\Sigma Fy = 0$; W x V = x + 8.67 = 0 2 4x x V = - 8.67 9 2 2 V = x 2 - 8.67 9 Moment + $\Sigma M Z = 0$; x æxö M + $Wx \cdot cç \div \div \cdot 8.67 x = 0 2 èc 3 @ æ 4 ö x 2 M = 8.67 x - cx 3 27 5-12 Copyright © 2018 McGraw-Hill Education. But interpolation should only be carried out between values of the same sign.$

Lateral bracing is located on all four edges of the mechanical floor framing for stability and transfer of lateral loads. Draw the influence lines for the reactions Ax and Ay at the left pin support and the bending moment on section 1 located at the face of column AB. Draw the influence lines for RA and the bar forces in members AD, EF, EM, and NM., I = 180 in. 8-9 Copyright © 2018 McGraw-Hill Education. For both the girder and the arch, determine all forces acting on the arch joints as well as the joint displacements. E P4.44. For all members, E = 4000 ksi 4 and I = 1000 in. Express answer in terms of EI.

C D 30 kips I I I I 1.5I A F E 16' 16' P11.27 -6 EI -6(3000)1500 æç 1 \ddot{o} (1) = \dot{c} = -108.5 kip-ft 2 L (12(12)) \dot{c} = \dot{c} = -162.8 kip-ft 2 L (12(12)) \dot{c} = -162.8 kip-ft 2 L (12(12)) \dot{c} = -162.8 kip-ft 2 L (12(12)) \dot{c} = -162.8 kip = -162.8

One efficient and practical way to increase the lateral stiffness to a frame with simple framing is to use diagonal braces, as in Case 3. 10 kips P3.18. Analyze the frame in Figure P11.32 by moment distribution.

Approximate method gave reasonable results for girder end moments and Column end moments, which were within 6.4% and 3.6%, respectively. 30' P2.22.

Continued 24 25 26 27 1 9.428 0 0 2 9.428 0 0 2 9.428 0 0 4 9.428 0 0 5 9.428 0 0 4 9.428 0 0 5 9.428 0 0 1 -6.667 0 0 2 -6.667 0 0 3 -6.667 0 0 5 -6.667 0 0 5 9.428 0 0 2 9.428 0 0 3 9.428 0 0 4 9.428 0 0 5 9.428 0 0 3 9.428 0 0 4 9.428 0 0 5 9.428 0 0 1 8.656 0 0 3 8.656 0 0 13-53 Copyright © 2018 McGraw-Hill Education. 9 kips P=? To counteract this sag, a cable and post are added beneath the beam. (kN) 9-29 Copyright © 2018 McGraw-Hill Education. (c) Using only Table 8.2, explain your result. and E = 30,000 kips/in. P3.21. Results are Inconsistent: (1) Magnitudes must be the same since bars have 45° slope. This analysis shows that forces take the most direct path to the supports. 8' B C 8' 12' 5k A 8' D 16' P16.6 Force Vector Joint "B" Joint "C" Freebody "BC" M1 + 20 = 0 M2 - 20 + 10 = 0 F3 - 2.5 = 0 M1 = 20 M2 = 10 F3 = 2.5k é 20ù ê ú [F] = êê-10úú ê ú êë-25úû Stiffmess Matrix Unit Rotation at Joint "C" Unit Rotation at Joint "B" 16-11 Copyright © 2018 McGraw-Hill Education. 9' P11.10. (f) Evaluate the moment at section 1. , and 2 E = 29,000 kips/in.

The objective is to establish the difference in response of a parabolic arch to (1) uniformly distributed loads and (2) a single concentrated load. Hhz = M 4 Shear (kips) Point D: 150(hC) = 2100 2300 2100 1080 hC = 14 ft Moment (kip-ft) Point D: 150(hD) = 2300 hC = 15.3 ft Ax = H A Ay hB B T 6-12 Copyright © 2018 McGraw-Hill Education. WQ = UQ $\Sigma Q \cdot \delta P = 0 k 3 \delta - 1k (3\phi) = 0 2 AH \delta AH = 2 c 8-31 Copyright © 2018 McGraw-Hill Education. 10 kips 10 kips P11.15. 1.5" P8.27. displacement, the designers will shorten the distance between supports by moving support A to the right. (b) If the horizontal displacement at joint B is not to exceed 83 in., what is the minimum required value of I? Load is transferred from the roadway to the upper panel points by a system of stringers and floor beams (not shown). P13.18. <math>\Delta C$, TEMP + $\delta CC RC = 0 \delta CC = 0.27 c c + 0.0076 c c RC = 0.27 c c + 0.0076 c c$

= +8.1k 2 1 + M A max. 3 kips P13.14. C p = 0.8 on windward side p = 49.05 k z (0.85)(0.8) = 33.354 K z Compute "p" for Various Elevations Elev. 9 kips A P12.54.

The 18-kip load is an additional live load that represents a heavy wheel load. CL 11" P = 24 kips 22" CL C D B 9' 16" 9' 22" varies 12" A 16" 18' CL P8.44 (a) Roller Support at D 8-51 Copyright © 2018 McGraw-Hill Education. (b) Diagonal bars do not buckle and may carry either tension or compression. Expansion, $\alpha = 6$ '10-6 in / in of Determine Bar Forces and Reactions $\Delta L AB = \alpha L\Delta T = 6$ '10-6 '60' 25'12 $\Delta LBC = \Delta L AB = 0.108$ Ins ΔC , TEMP 1k ' ΔC , TEMP = $\Sigma FQ \Delta LP$ Final Results 5 (0.108)2 = 0.27 in 4 $\delta CC 11.17 \Delta C$, TEMP = $\delta CC 1140 AE 1140 = 0.0076$ in 5'30,000 Previously Computed Compatibility EQ. The moment diagram deviates considerable. of cross section @ 120 lb/ft3. Determine the forces or components of force in all bars of the trusses in Figures P4.43.

FBD "BC" Girder P13.20. El 5 6CC = 1.214 in. P13.20. Computer Study—comparison of cantilever and portal methods with an exact analysis. P2.17 Cs is Approximately 0.9 (Non-Slippery Where pf Flat Roof Snow Load pf 0.7 CeCt I pg Surface) Ce = 0.7 Windy Area Ps = Cs Ps = 0.9 (19.6 psf) = 17.64 psf Ct = 1.0 Heated Building I = 1.0 Type II Occupancy Pg = 40 psf for Boston æ 16 ¢ õ Cs = Based on Roof Slope θ = Tan-1 cg + + = 33.7 cè 24 ¢ $+ \theta$ Pf = 0.7 (0.7)(1.0)(1.0)(1.0)(40 psf) = 19.6 psf Uniform Load Acting on Trusses Spaced @ 16o.c. Wsnow = 17.64 psf (16 ¢) = 24 Copyright © 2018 McGraw-Hill Education. 40 kips P13.16. Analyse by Moment Distribution. For the girder in Figure P12.19, draw the influence lines for the reaction at I, the shear to the right of support I, the moment at C, and the shear between CE. If stable, indicate if determinate or indeterminate. I = 348 in. Plot the graph of eactions due to 10 kN/m at 0 b D A C 15 m 15 m 12 m 30 m P12.22 Eq. of Parabola y = Kx 2 at x = 30, yB = 1:12 = K 75 Point D at x = 15, yD = Influence Lines 1 (15) = 375 kN ætl õ M k + M at D = cg. '24 ' 5.625+10 kN/m cè 2 + ø (LOAD 'AREA) = 675 kN · m M at D if Uniform Load Acts over Entire Span ætl õ ætl õ M D = WA = 10 cg. '1.25 '60++10 cg. '36 '3.75++ M * = 0 cè 2 cè 2 + ø + ø *Uniform load on a parabolic arch produces axial force on all sections but no moment or shear. Compute the vertical displacement of the hinge at C for the funcical roboting shown in Figure P8.16. 4. Given: E = 200 GPa and 6 4 I = 120 ' 10 mm . E = 29,000 kips/in. 4m 4m B C D 4m A 200 kN P9.31 Select RC as Redundant-Q-System PS-System ACC : 1.4 CO = 70712 5.66 (0.7

(b) If support B settles 3 in. of Condition (MHINGES = 0) Determinate (f) Remove Restraints: 3@B; 3 Cut @ C; 3 Cut @ D Indeterminate 9° 5-56 Copyright © 2018 McGraw-Hill Education. The top 2 chord members 1, 2, 3, and 4 are $4 \times 4 \times 1/4$ in. The pin joint at E acts as a hinge. Analysis of 2nd floor and attached columns. at Joint B MCB = M BC = M DC 2 EI (20C + θ B) + 16 8 2 EI (θ C) = 8 Equil.

M EXT = 15 (6 ¢) = 90 ft · k k M INT = F1 (22.5¢) + F2 (7.5¢) + F3 (7.5¢) + F4 (22.5) = 3V1 A(22.5)(2) + V1 (1.5 A)7.5(2) = 157.5 V1 A M EXT = M INT 90 = 157.58 V1 A V1 A = 0.5714 k (Exact 2.4 k) F2 = F2 = 0.5714 k (1.5) = 0.857 (Exact 0.54 k) Beam "PO" (M19 Risa) Assume shears are equal each end of beam. Use the shear at hinge B as the redundant.

B w = 6 kN/m C 5m A 3m P8.29 Virtual Work Compute $\delta Cx: x Q1 \cdot \delta Cx = \Sigma \delta MQ1 M P 0 dx EI 5 dx EI 0 I æ 3 \ddot{o} = cc30 x 2 + x 4 \div \div EI ec 4 ø 1 \cdot \delta Cx = \delta x (60 + 3 x 2) kN 5 0 = 1218.75 EI \delta Cx = 25.4 mm$ P-System Q1 System for δCx Compute $\delta Cy: x Q2 \cdot \delta Cy = \Sigma \delta MQ2 M P 0 dx EI 5 3 dx dx + \delta x (20 x) EI EI 0 0 1455 I 3 5 3 3 = (180 x + 3 x) 0 + 60.67 x 0 = EI EI \delta Cy = 30.3 mm 1 kN \cdot \delta Cy = \delta (3kN \cdot m)(60 + 3 x 2) Q2$ System for $\delta Cx 8-35$ Copyright © 2018 McGraw-Hill Education.

A B C (a) A B C D (b) A B C F D E (c) D I J G H E F A B (d) P10.35 (a) Indeterminate 3°: Rotation @ A, B, & C (b) Indeterminate 3°: Rotation @ A, B, C, D, E & F (d) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C (b) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C, D (c) Indeterminate 3°: Rotation @ A, B, C (c) determine the forces in the bars listed below. Their analysis results are differ due to the knee brace. Ignore the applied load. Analyze by moment distribution. (b) Draw freebody diagrams of each truss and evaluate the forces applied to the trusses at joints C, B, and D. C P4.9. Determine the forces in all bars of the trusses. Round the camber to 1¼ in. itudinal direction. Express answer in terms of E, I, L, and w. Analyze the frame in Figure P10.16. B 12' I = 240 in.4 I = 120 in.4 A 24' P10.25 I G = 2 IC ; E = 29,000 ksi; $\theta C = 0.75 = 0.0026$ rad 12(24) θ A and $\theta B =$ Unknowns 2 EIC $\acute{e} 2\theta + \theta B$ $\grave{u} \hat{u} 12(12)$ $\ddot{e} A 2$ EIC $\acute{e} 2\theta + \theta A \grave{u} M BA$ Draw the shear and moment curves for the footing in the long = $\hat{u} 12(12) = B 2 E 2 IC \neq 2\theta - 3(0.0026) M BC = \hat{u} 24(12) = B 2 E 2 IC \neq 0 - 3(0.0026) M CB = \hat{u} 24(12) = B M AB = Equilibrium Equations (1) M AB = 0 (2) M BA + M BC = 0$ Substitute Moment Equations into Equilibrium Equations into Equilibrium Equations (1) M AB = 0 (2) M BA + M BC = 0 Substitute Moment Equations (1) M AB = 0 (2 (1a) Substitute θA into 2 a $\theta B = 0.00233$ rad $\theta A = -0.001115$ rad Substitute Values for θA and θB into Moment Equations M AB = 0 M AB = 0 2(29000) 1 (120) éë 0.00223 - 0.0078 u = 72 i 2 M BA = M BA = 13.45 ft k M BC = -13.45 ft k M CB = -13.45 ft k M CB = -13.45 ft k M CB = -13.45 ft k M BC = -13.45 ft k M BC = -13.45 ft k M CB MCB = -22.434 ft·k 10-28 Copyright © 2018 McGraw-Hill Education. The external pressures for the wind load perpendicular to the ridge of the building are shown in Figure P2.13b. (a) Compute the reactions, plot the deflected shape, and draw the shear and moment curves for the columns and girder, using the computer program. A E 4 @ 30' = 120' (b + r = 12) > (2n = 12) (b + r = 10) > (2n = 12) (b + r = 10) > (2n = 10) Determinate, stable Unstable: load applied at joint connecting diagonals cannot be supported. 2-23 Copyright © 2018 McGraw-Hill Education. The truss in Figure P3.30 is composed of pin-jointed members that carry only axial load. Likewise, constant positive shear results in a linear moment diagram with positive slope. K J P12.32. P4.51. P6.4. (a) Determine the reactions at supports A and E and the maximum tension in the cable in Figure P6.4. (b) Establish the cable sag at points C and D. B 30 kips components are equal, the only way equilibrium can be satisfied i.e. Y+Y = 0 is for Y's to = 0 if Y's = zero, F's must equal zero Joint B + $\Sigma Fy = 0.50$ - YAB = 0 Since FBH = 0, joint consists of three bars, two of which are colinear: therefore AE 10, 791.33 800(106) = + 6 -6 200 '10 '600 '10 1500 ' 200 '106 5 11.18 = 0.0899 + 0.0026 = 0.0925 m = 92.5 mm P-System Note: The Contribution on Axial Deformations in Member BCD is Small and Not Included. -1 P8.22. 6' 6' P14.2 Stiffness of beam under mid-span point load, KP: P 3 1 K P L æc L öæ 2 L L ö÷ 5K P L + \div = cc \div \div \div ccc 2 2 EI è 2 øè 3 2 2 ÷ø 48EI 48EI = 23.15 kips/in KP = 5 L3 L/2 L/2 t BA = 1 = tBA = ΔB = 1 in. Determine the reactions at all supports and the force transmitted through the hinge at C. The reinforced concrete bridge girder, attached to the massive end wall as shown in Figure P12.59, may be treated as a fix-ended beam of varying depth. P16.4. Using the stiffness method, analyze the frame in Figure P16.4 and draw the shear and moment curves for the members. Case 2 and 3: As a general rule, the stiffer part of the structure attracts (i.e., resists) more forces. (a) Compute the vertical deflection and slope of the cantilever beam at points B and C in Figure P8.20. = 0.1 - MA max.(see 11.25, for evaluation of δAA) 4.375 E 4.375(12)3 = 29,000 in = 0.261 k Horiz 1 1.2 & RAX = 4.6 kips 0.261 δAA = Select RAX as Redundant + ΣMC : RAY 30 - RAX 15 = 0 RAY = 1 R = 2.3 kips 2 Ax Released Structure Deflected Shape Moment Created by Fabrication Error Units (ft.kips) 9-46 Copyright © 2018 McGraw-Hill Education. Published on Jul 30, 2020AboutDownload Solution Manual Fundamentals of Structural Analysis 5th edition 0073398004 Fulllink donwload Want more? Draw the influence lines for the moment at D, and the moment at D, and the shear to the right of support A. Determine the reactions at supports A, C, and E. 258 P2.17. Consider the truss in Figure P9.36 without the applied loads. 8 kips 10 kips 6 kips A 10' B 6' 10' 30' 10' P12.44 Max. Given: I = 144 in. , carry out an approximate analysis of the second floor by analyzing the second floor beams and the attached columns (above and below) as a rigid frame. L.L. Compr. w = 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kips B 2.4 kips/ft B C 2I 6 kips D 2I I 12' 6' A 5' 20' P8.38 10 kip C B FP (kip-ft) E D B D E C 1 kip E C D 1 kip C E D B -5 -1 -1 5 MQ, BV FQ, BV MQ, BH FQ, BH (kip-ft) (kips) (kip-ft) (kips) A δBV A A -12 A FL 1 1 1 1 1 58(12) 50(5) 5 430(5) 6 (430 466)(5) 6 CM1 M3 L FQ P EI AE 2 EI 3 EI EI 2 AE 696 26, 235.83 (1728) (12) δBV 8.63 in. ΣM A = 10(20) + 10(40) - Gy (80) = 0 10 kips Gy = 7.5 kips $Dx \Sigma Fy = Ay - 10 - 10 + 7.5 = 0 Ay = 12.5 kips$ Dy Utilizing the hinge at mid-span:, the horizontal reaction is found. $3 4 C 20 kN D E 90^\circ 3 4 3m 90^\circ F B J I H 3m 60 kN G A 4@4m P4.51$ Reactions + $\Sigma M A = 0$: $12^\circ 3 + 12^\circ 6 + 16^\circ 4 + 60^\circ 8 - 16 RG = 0 RG = 40.75 kN + \Sigma Fy = 0$: $RAY + 40.75 - 32 - 60 = 0 RAY = 51.25 kN + \Sigma Fx = 0$: $12^\circ 12 + 12^\circ - 10^\circ -$ RAX = 0 RAX = 24 kN Joint G Cut Vertical Section to Right of D + $\Sigma M A = 0.0 = FDE 3 - 40.75(4)$ FDE = 54.33 kN (C) 4 X EF = YEF = 54.33 kN I 17 ΣK S 144 FEM AB = -FEM BA = + AB ΣM A = 0 = 144 '12 + 203.3 - VBA 24 VBA = 80.47k + BC ΣM B = 0 = -203 -101.65 - VCB 18 VCB = 16.925k 11-7 Copyright © 2018 McGraw-Hill Education. (see 7.4 in text) æ 8hx ö WL2; T = H 1 + cc 2 ÷÷ è kh è L ø 2 H = æ h4 ö L, T = H 1 + cc 2 ÷÷ è kh è L ø 2 H = æ h4 ö L, T = H 1 + cc 2 ÷÷ è kh è L ø 2 H = æ h4 ö L , T = H 1 + cc 2 ÷ è kh è L ø 2 H = æ h4 ö L , T = H 1 + cc 2 ÷ è kh è L ø 2 $4 \ddot{o} \div T = 9001 + cc \div = 969.33k \dot{e}c 120 \ a$ For $h = 12 \ c$; $H = 6(120)2 = 450 \ kips$; and $T = 445.6 \ k \ a^{3}6 \ 6(120)2 = 225k$; and T = 442.53k For $h = 468.6 \ k \ a^{3}6 \ 6(120)2 = 225k$; and T = 442.53k For h = 402.5k For $h = 60 \ c$; $H = 8^{-}60 \ 1 \ h \ 1$ Note: For economy of materials should be in the range of to 3 4 L For h = 36 ¢; H = 6-19 Copyright © 2018 McGraw-Hill Education. P4.56. 6 kips - 7.5 4.5 - N 1.5 + 4.5 7.5 (T) - 25.5 (C) 1.5 19.5 (T) K H - 7.5 (C) 4.5 4.5 4.5 + 3 kips + 7.5 + 4.5 7.5 7.5 7.5 (T) + 7.5 J Ay = 18 kips Iv = 18 kips (b) Diagonals carry either tension or compression.; I = 500 in. Continued MQ $\theta ZL = \Sigma \circ MQ M P dx EI 3 \approx x + 3 \circ \div x dx dx (24 x) + \circ cc \div e 24(6 + x) - 24(3 + x) + \circ cc \div e 24(6 + x) - 24(3 + x) + EI 3 EI \circ 0 9 \theta = \circ L A 0 3 + \circ$ + 0.01345 X B -2/3 (C) A $\delta BB = 0.01345$ in. 12' 400 kip • ft B D C 15' E A P11.30 M = 100 k·ft = FEM DB 4 3æ I ö K AB = K DE = cc ÷÷ 4 èc15 ø I K BD = 24 6 5 DFBA = , DFBD = 11 11 Case A: No Sideway FEM BD = 6 EI\Delta L2 Assume Δ is selected so FEM's = 22 k · ft FEM AB = FEM DE = Case B: Sideway 11-43 Copyright © 2018 McGraw-Hill Education. K2x and K2y are stiffness coefficients. The shear diagram is colse to the exact values. (b) Compute the vertical deflection of joint B. "C": + $\Sigma Fx = 0$; - 55 + 45 + 20 - CI x = 0 CI x = 10 kN \neg C CI y = 10 (2) = 4 kN C 5 $\Sigma Fy = 0$; - 8 - 4 + GC = 0 GC = 12 kN T JT. vertical displacement at joint B. Then distribute the base shear along the height of the building. Construct the influence lines for RC and MC in Figure P12.58, using the Müller-Breslau method. (b) Compute the force in the post causes a moment equal in magnitude, but opposite in direction, to the moment in the beam. 9 C A P12.58 B 12' C 12' D 6' MA, RA, MC, and VC (to left of support C) P12.11 Müller-Breslau 12-12 Copyright © 2018 McGraw-Hill Education. P = 16 kips w = 4 kips/ft A B 9' 9' 20' 9' P11.9 Modify Stiffness 3 I I 5 = = 4 18 24 120 1 I I 3 = = = 2 20 40 120 8 \Sigma K Sc = 120 K AB = DFAB = 5/8 K BC DFBC = 3/8 & wL2 PL ö ÷ FEM AB = -ccc + ÷ ÷ cè 12 8 ÷ ø 4 $(18) 2 = -FEM BC = 12 wL2 = 12 + 16 (18) 8 2 4 (20) 12 D C = -144 kip \cdot ft = -133.33 11-11 Copyright © 2018 McGraw-Hill Education. E D P4.23. while the load acts, recompute the reactions and bar forces. C3 40' 20' Building Section C1 P2.8 & 40 20 ö 2 + ++; cc +$ = 900 ft, K LL = 4, AT K LL = $3600 > 400 \div c 2 \text{ ec} 2 2 \text{$ the influence lines for forces in bars AL and KJ in Figure P12.39. 30' A P6.14. Compute the reactions and draw the shear and moment curves for the beam in Figure P9.21. Explain your results. EI 5 1 kip 30 kips ()() 4 1 12 2 4 $\Delta 3 = tCB = 15 15 5 2 EI 3 5 900 4 = (1728) = 0.31104$ in. and support D settles 1 in. Draw the influence lines for the reactions at supports A and D, the shear and moment at D. 1) (x - 4) 8 x - 34(x - 4) + 3(x - 4) + 3(x5m P10.4. Analyze the beam in Figure P10.4 by slope-deflection and draw the shear and moment diagrams for the beam. \acute{e} 2(30000)240 \grave{u} \acute{e} 0.00071 - 0.007813 \grave{u} - 768 \acute{u} 1 M AB = \hat{e} \ddot{e} \hat{u} \hat{e} \hat{u} 12 192 ft k M AB = -108.4 (b) Reactions Σ M A = 0; 60(10) -16 RB - 108.4 = 0 RB = 30.73k Σ Fy = 0; 60 - RB - RA = 0 RA = 29.27k (c) Δ C 1 24 tCB - (4) (3¢) (1728) = 0.023¢¢ 3 EI $\Delta C = 0.5$ ¢ + 0.034 + 0.023 = 0.557¢¢ 10-10 Copyright © 2018 McGraw-Hill Education. C A D 6' F 8' E 12' 8' P9.30 Compute $\Delta DX \Delta T$, Top Chord has $\Delta T = +50^{\circ}F \Delta L AB \cdot \Delta DX = 2FQ \cdot \Delta L \Delta DX = 2FQ$ 0.625(.039)2 + 0.5(0.0468) k Select RDX as the Redundant Δ DX = 0.07215 in FP = FQ Compute δ DX FQ L 2 1k $\cdot \delta$ DX = 0.07215 + RDX 0.001 = 0 RDX = 0.001 in 10 $^{\prime}30,000$ P-System; Temp Increase of 50° Compatibility Eq. Δ DX + RDX δ DX = 0.07215 + RDX 0.001 = 0 RDX = 0.07215 + RDX 0.001 = 0 RDX = 0.001 in 10 $^{\prime}30,000$ P-System; Temp Increase of 50° Compatibility Eq. Δ DX + RDX δ DX = 0.07215 + RDX 0.001 = 0 RDX = 0.001 in 10 $^{\prime}30,000$ P-System; Temp Increase of 50° Compatibility Eq. Δ DX + RDX δ DX = 0.07215 + RDX 0.001 = 0 RDX = 0.001 in 10 $^{\prime}30,000$ P-System; Temp Increase of 50° Compatibility Eq. Δ DX + RDX δ DX = 0.07215 + RDX 0.001 = 0 RDX = 0.001 in 10 $^{\prime}30,000$ P-System; Temp Increase of 50° Compatibility Eq. Δ DX + RDX δ DX = 0.07215 + RDX 0.001 = 0 RDX = 0.001 in 10 $^{\prime}30,000$ P-System; Temp Increase of 50° Compatibility Eq. Δ DX + RDX δ DX = 0.07215 + RDX 0.001 = 0 RDX = 0.001 in 10 $^{\prime}30,000$ P-System; Temp Increase of 50° Compatibility Eq. Δ DX + RDX δ DX = 0.07215 + RDX 0.001 = 0 RDX = 0.001 in 10 $^{\prime}30,000$ P-System; Temp Increase of 50° Compatibility Eq. Δ DX + RDX δ DX = 0.07215 + RDX 0.001 = 0 RDX = 0.001 in 10 $^{\prime}30,000$ P-System; Temp Increase of 50° Compatibility Eq. Δ DX + RDX δ DX = 0.07215 + RDX 0.001 = 0 RDX = 0.001 in 10 $^{\prime}30,000$ P-System; Temp Increase of 50° Compatibility Eq. Δ DX + RDX δ DX = 0.07215 + RDX 0.001 = 0 RDX = 0.001 in 10 $^{\prime}30,000$ P-System; Temp Increase of 50° Compatibility Eq. Δ P-S δ DX = 0.001 + 0.0-72.15 kips ¬ 6' Q-System For ΔDX P and Q System For ΔDX All Forces in kips Final Results 9-32 Copyright © 2018 McGraw-Hill Education. ΔTEMP = 60(6.5'10-6)(48'12) = 0.2246 Δ BX, TOTAL = 5.662 ¢¢ + 0.23¢¢ = 5.887 in. In addition to the applied loads, supports A and D 2 settle by 2.16 in. B C D E F G H 4' L 16' K M J 36' I A 6 panels @ 24' = 144' P12.40 (b) Dead Load Forces in Bars CM and ML é1 ù 1 FCM = $4.8k / \text{ft} \hat{e} (65)(-1083) + (81)(0.25)\hat{u} \hat{e} \hat{e} 2 \hat{u}\hat{u} 2 = 387.07$ kips Compression (c) Live Load Forces in Bars CM 1 kips + 20(-1.083) Max.Compr. (c) If the live load is represented by a uniformly distributed load of 0.8 kip/ft of variable length and a concentrated load of 20 kips, determine the maximum force in bar CM produced by the live load. WL2 20 P7.3. Compute the maximum deflection. Assume that a fixed-end condition at the bottom of the walls at A and D is produced by the connection to the foundation mat. Given: $E = 30,000 \ 2 4$ kips/ft A C B 0.3" 8' 2' P9.8 $E = 30,000 \ ksi$ and I = 240 in 4 Selecting By as the redundant, the compatibility equation is: ($\Delta BS + \Delta BO$) + $\delta BB \ X B = \Delta B$ $0.003 \text{ rad } A \Delta BS \Delta BS = 0.003(8)12 = 0.288 \text{ in}$. Eq. (2) into Eq. 1 2 EI æc 242 ÷ ö ÷ - 30 = 18.4 kN · m c 10 èc EI ÷ ø 4 EI æc 242 ö ÷ = ÷ + 196 = 230.57 kN · m c 14 cè EI ÷ ø M AB = M BA MCB Compute Shears + AB $\Sigma M A = 0 = 18.4 + 126.8 + 24^{\circ} 5$ · VBA 10 + BC $\Sigma M B = 0 = 168^{\circ} 7 + 106^{\circ} 2$ EI æc 242 ö ÷ = ÷ + 196 = 230.57 kN · m c 14 cè EI ÷ ø M AB = M BA MCB Compute Shears + AB $\Sigma M A = 0 = 18.4 + 126.8 + 24^{\circ} 5$ · VBA 10 + BC $\Sigma M B = 0 = 168^{\circ} 7 + 106^{\circ} 2$ EI æc 242 ö ÷ = ÷ + 196 = 230.57 kN · m c 10 èc EI ÷ ø A B = 0 = 168^{\circ} 7 + 106^{\circ} 2 EI æc 242 ö ÷ = ÷ + 196 = 230.57 kN · m c 10 èc EI ÷ ø A B = 0 = 168^{\circ} 7 + 106^{\circ} 2 EI æc 242 ö ÷ = ÷ + 196 = 230.57 kN · m c 10 èc EI ÷ ø A B = 0 = 168^{\circ} 7 + 106^{\circ} 2 EI æc 242 ö ÷ = ÷ + 106 = 230.57 kN · m c 10 èc EI ÷ ø A B = 0 = 168^{\circ} 7 + 106^{\circ} 2 EI æc 242 ö ÷ = ÷ + 106 = 230.57 kN · m c 10 èc EI ÷ ø A B = 0 = 168^{\circ} 7 + 106^{\circ} 2 EI æc 242 ö ÷ = ÷ + 106 = 230.57 kN · m c 10 èc EI ÷ ø A B = 0 = 168^{\circ} 7 + 106^{\circ} 2 $230.57 - 126.8 - VCB 14 VBA = 26.52 kips VCB = 91.41 kN + \Sigma Fy = 0 = VAB - 24 + 26.52 + \Sigma Fy = 0 = VAB - 24 + 26.52 + \Sigma Fy = 0 = VAB - 24 + 26.52 + \Sigma Fy = 0 = VAB - 24 + 26.52 kips VCB = 91.41 kN + \Sigma Fy = 0 = VAB - 24 + 26.52 kips VCB = 91.41 kN + \Sigma Fy = 0 = VAB - 24 + 26.52 kips VCB = 91.41 kN + \Sigma Fy = 0 = VAB - 24 + 26.52 kips VCB = 91.41 kN + \Sigma Fy = 0 = VAB - 24 + 26.52 kips VCB = 91.41 kN + \Sigma Fy = 0 = VAB - 24 + 26.52 kips VCB = 91.41 kN + \Sigma Fy = 0 = VAB - 24 + 26.52 kips VCB = 91.41 kN + \Sigma Fy = 0 = VAB - 24 + 26.52 kips VCB = 91.41 kN + \Sigma Fy = 0 = VAB - 24 + 26.52 kips VCB = 91.41 kN + \Sigma Fy = 0 = VAB - 24 + 26.52 kips VCB = 91.41 kN + \Sigma Fy = 0 = VAB - 24 + 26.52 kips VCB = 91.41 kN + \Sigma Fy = 0 = VAB - 24 + 26.52 kips VCB = 91.41 kN + \Sigma Fy = 0 = VAB - 24 + 26.52 kips VCB = 91.41 kN + \Sigma Fy = 0 = VAB - 24 + 26.52 kips VCB = 91.41 kN + \Sigma Fy = 0 = VAB - 24 + 26.52 kips VCB = 91.41 kN + 25 kips VCB = 91.41 k$ kN Rby 24 Rbx = = 4 3 5 kN; Rbx = 19.2 + Σ Fx = 0; B 5 Rby = 14.4 kN RB 15kN - 19.2 kN + Ax = 0 + 4 3 4m A Ax Ax = 4.2 kN Σ Fy = 0; C 15 kN 3m 3m -14.4 kN + Ay - 20 kN = 0 Ay = 34.4 kN D 20 kN Ay 3-4 Copyright © 2018 McGraw-Hill Education. Also draw the deflected shape. 20(12) L 5 6 1 2 é 1 0 -1 0 ù 5 ê ú ê 0 0 ú 6 0 0 ú = K1 = 250 êê 1 0úú 1 ê-1 0 ê 0 0 0 0úú 2 ëê û sin f = y] - y I 20 Adjust sealar so Matrices can be Combined = 0-0 = 0 20 5 é 1.25 ê ê 0 200 êê 1.25 0úú 1 0 0 0úú 2 û Member 2 20 - 0 15 - 3 cos f = sin f = = 0.8 = 0.6 25 25 AE 2(30,000) = = 200 k/in. Consider only the flexural deformations. D 24' P5.22. Clamp Joint (2) Apply Loads. Compute the reactions and draw the shear and moment curves for the beam if support C settles 24 mm and support A rotates counterclockwise 0.005 rad. (c) Determine the maximum value of live load moment at B produced by the set of wheel loads. Gilbert SOLUTIONS MANUAL CHAPTER 5: BEAMS AND FRAMES 5-1 Copyright © 2018 McGraw-Hill Education. (e) –2 Eqns. Axial Load: Assume load of 0.55 wL for exterior griders and 0.50 wL from interior griders. Determine all bar forces. E 4m P3.16. z V(x) P5.12. P11.13. β = 10 kips/in. P4.36. Represent the arch by a series of straight segments between joints. $\neg \Delta CO \ 1 \ C \ 3/4 \ (T) \ 0$ () 5 = $\Sigma FQ \ \Delta LP$ = - (-0.25) 4 ΔCO = 0.3125 in. (OK) Relative Displacement between 2nd and 3rd floors = 0.24 - 0.136 = 0.104 in., columns 2 4 A = 13.1 in.

 $52 - 18 \sin 52$) x = 13.52 x M AB P = 15 sin 21 - 18 cos 21 = -11.23k FBC M BC = 659.5 - 20.52 x Q Q FAB = FDE = -0.763 M AB = M DE = -0.763 M AB = -0.763 M AB = M DE = -0.763 M AB = -0.763 M 2018 McGraw-Hill Education., I of 4 beam = 1200 in. Select the origin at point C. Member Stiffness Matrices (See Equation 15.48) Member 1 x - xI 20 - 0 = =1 cos f = J 20 L AE 2(30000) = 250 k/in., 2 and members BD, DE, and CE have A = 1 in.

w = 5 kips/ft P13.1. Use an approximate analysis (assume the location of a point of inflection) to estimate the moment in the beam at support B (Figure P13.1). P10.30. 80 kN A B 8m C 4m D 4m E 4m P12.46 Construct Moment Envelopes. Compute the reaction at C if joint A settles by 0.5 in. The ordinates of the moment influence line at midspan B of a 2-span continuous beam are provided at every one-tenth of the span in Figure P12.56. 13-41 Copyright © 2018 McGraw-Hill Education. Consider the possibility of both tension and compression force in each bar. C P10.27. Given: I = 1500 in. Analyze the rigid frames in Figure P14.10, using symmetry to simplify the analysis. Continued Multiply Moments in Case B by 5.92 and Add to Case A 1.34 Results Shear (kips) Deflected Shape -16.7 Moment (k ft) 11-44 Copyright © 2018 McGraw-Hill Education. 2 3 B A KS = L 5EI L3 C D spring L L EI = constant P16.2 [K] From 16.5 ii-wL üi ï ï ï {F} = ï í D + 0 í \hat{i} eë L úû ii-5 ii L L L ii ii ii 6 i) wL4 D = 0.042 EI Force in Spring = Ks $\Delta = 0.208$ wL 16-6 Copyright © 2018 McGraw-Hill Education.

C P10.32. P3.27. Also compute the horizontal displacement of 2 joint B. Moment (kN) 0 P13.20. FGCX + 2 ΣFy = 0 = 21 + 9 - FBCY -12 \ FGCX = 4.5 kN FBCY = 18 kN compr.

Hinges Freebody left of Hinge to "C" 2 k/l (2.4 ¢) 2 HC = -23.04 ft · k - M C = (Equivalent to wL2) 12.5 13-7 Copyright © 2018 McGraw-Hill Education. D 10' B C 50 kips 10' A 12' 12' P10.34 Unknowns: θA , θB , θC , $\Delta \Psi AB = \Psi BA = \Delta 10 \Psi CD = \Psi DC = -\Delta 10$ (50)(24) PL == -150 k·ft 8 8 = 150 k·ft 8 8 = 150 k·ft FEM BC = FEMCB Member end Moments 2 EI æç $\Delta \ddot{o}$ 2 EI M BC = (2 θ B + θ B - 3 ÷÷ + 10 è 10 ø 10 è 10 ø 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ 2 EI M BC = (2 θ B + θ B - 3 ÷÷ + 10 è 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ 2 EI M BC = (2 θ B + θ B - 3 ÷÷ + 10 è 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ 2 EI M BC = (2 θ B + θ B - 3 ÷÷ + 10 è 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ 2 EI M BC = (2 θ B + θ B - 3 ÷÷ + 10 è 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ 2 EI M BC = (2 θ B + θ B - 3 ÷÷ + 10 è 2 EI M BC = (2 θ B + θ B - 3 ÷÷ + 10 è 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ = 2 EI M BC = (2 θ B + θ B - 3 ÷÷ + 10 è 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ = 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ = 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ = 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ = 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ = 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ = 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ = 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ = 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ = 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ = 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ = 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ = 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ = 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ = 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ = 2 EI M BC = (2 θ B + θ C) -150 24 æ $\Delta \ddot{o}$ = 2 EI M BC = (2 \theta B + \theta C) three-hinged circular arch.

3m B P9.30. To eliminate the 3-in. For the arch shown in Figure P6.20, the thrust force exceeded the abutment's lateral support capacity, which is represented by a roller at C.

, IBC = 900 in. The importance factor I is 1.15 and Kz 1.0. Use the simplified procedure to determine the design wind load, base shear, and building overturning moment. P5.55. 20 kips A C B D 12' E 9' F 9' 12' 9' 9' P10.16 Symmetric structure w/symmetric load & no lateral load, thus no sway. (in.3) 6780 Vol. P15.3. Form the structure stiffness matrix for the truss in Figure P15.3. Partition the matrix as indicated by Equation 15.30. D C P9.25. P6.20.

Sketch the deflected shape. 12' P11.30. Calculate the tributary areas for (a) floor beam B3, (b) floor beam B4, (c) girder G3, (d) girder G4, (e) edge column C3, and (f) corner column C4. 2 kips/ft P9.41. The slab weighs 2 125 lb/ft. -10.5 (C) + 7.5 7.5 -19.5 (C) 6 kips -3 (C) 7.5 C -3 (C) -7.5 (C) 6 kips -3 (C) 7.5 C -3 (C) -7.5 (C) 6 kips -3 (C) 7.5 C -3 (C) -7.5 (C) 6 kips -3 (C) 7.5 C -3 (C) -7.5 (C) 6 kips -3 (C) 7.5 C -3 (C) -7.5 (C) 6 kips -3 (C) 7.5 C -3 (C) -7.5 (C) 6 kips -3 (C) 7.5 C -3 (C) -7.5 (C) 6 kips -3 (C) 7.5 C -3 (C) -7.5 (C) 6 kips -3 (C) 7.5 C -3 (C) -7.5 (C) 6 kips -3 (C) 7.5 C -3 (C) -7.5 (C) 6 kips -3 (C) 7.5 C -3 (C) -7.5 (C) 6 kips -3 (C) 7.5 C -3 (C) -7.5 (C) 6 kips -3 (C) 7.5 C -3 0.55 111 ΣK S¢ = 80 FEM AB = FEMCD = 11-41 Copyright © 2018 McGraw-Hill Education. Gilbert SOLUTIONS MANUAL CHAPTER 8: WORK-ENERGY METHODS FOR COMPUTING DEFLECTIONS 8-1 Copyright © 2018 McGraw-Hill Education. B h hinge A C T 60' 60' P6.17 Plot the variations of thrust, T, at support A for values of h=12', 24', 36', 48' and 60'. P12.21. Determine all reactions and draw the shear and moment diagrams for beam BC in Figure P9.43. Compute the reactions and draw the shear and moment diagrams for beam BD in Figure P10.22. If the temperature of interior column BE is 70°F at all times but the temperature of the exterior columns in winter drops to 10°F, determine (a) the forces created in the columns and the truss bars by the temperature differences and (b) the vertical displacements of the tops of the columns at points D and E. 8 kN H 16 kN 2m I F 2m A C B 5m 5m D 5m E 5m P4.43 + Section (1): $\Sigma M A = 0$; 16 kN (5 + 10 m) + 8 kN (15 m) - E y (20 m) = 0 E y = 18 kN + \Sigma M C = 0; 16 kN (15 m) - E y (20 m) = 0 E y = 18 kN + $\Sigma M C = 0$; 16 kN (15 m) - E y (20 m) = 0 E y = 18 kN + $\Sigma M C = 0$; 16 kN (15 m) - E y (20 m) = 0 E y = 18 kN + $\Sigma M C = 0$; 16 kN (15 m) - E y (20 m) = 0 E y = 18 kN + $\Sigma M C = 0$; 16 kN (15 m) - E y (20 m) = 0 E y = 18 kN + $\Sigma M C = 0$; 16 kN (15 m) - E y (20 m) = 0 E y = 18 kN + $\Sigma M C = 0$; 16 kN (15 m) - E y (20 m) = 0 E y = 18 kN + $\Sigma M C = 0$; 16 kN (15 m) - E y (20 m) = 0 E y = 18 kN + $\Sigma M C = 0$; 16 kN (15 m) - E y (20 m) = 0 E y = 18 kN + $\Sigma M C = 0$; 16 kN (15 m) - E y (20 m) = 0 E y = 18 kN + $\Sigma M C = 0$; 16 kN (15 m) - E y (20 m) = 0 E y = 18 kN + $\Sigma M C = 0$; 16 kN (15 m) - E y (20 m) = 0 E y = 18 kN + $\Sigma M C = 0$; 16 kN (15 m) - E y (20 m) = 0 E y = 18 kN + $\Sigma M C = 0$; 16 kN (15 m) - E y (20 m) = 0 E y = 18 kN + $\Sigma M C = 0$; 16 kN (15 m) - E y (20 m) = 0 E y = 18 kN + $\Sigma M C = 0$; 16 kN (15 m) - E y (20 m) = 0 E y = 18 kN + $\Sigma M C = 0$; 16 kN (15 m) - E y (20 m) = 0 E y = 18 kN + $\Sigma M C = 0$; 16 kN (15 m) - E y (20 m) = 0 E y = 18 kN + $\Sigma M C = 0$; 16 kN (15 m) - E y (15 m) - $\Sigma M C = 0$; 16 kN (15 m) - $\Sigma M C = 0$; 16 kN (15 m) - $\Sigma M C = 0$; 16 kN (15 m) - $\Sigma M C = 0$; 17 kN (15 m) - $\Sigma M C = 0$; 17 kN (15 m) - $\Sigma M C = 0$; 17 kN (15 m) - $\Sigma M C = 0$; 17 kN (15 m) - $\Sigma M C = 0$; 17 kN (15 m) - $\Sigma M C = 0$; 17 kN (15 m) - $\Sigma M C = 0$; 18 kN (15 m) - $\Sigma M C = 0$; 18 kN (15 m) - $\Sigma M C = 0$; 18 kN (15 m) - $\Sigma M C = 0$; 18 kN (15 m) - $\Sigma M C = 0$; 18 kN (15 m) - $\Sigma M C = 0$; 18 kN (15 m) - $\Sigma M C = 0$; 18 kN (15 m) - $\Sigma M C = 0$; 18 kN (15 m) - $\Sigma M C = 0$; 18 kN (15 m) - $\Sigma M C = 0$; 18 kN (15 m) - $\Sigma M C = 0$; 18 kN (15 m) - $\Sigma M C = 0$; 18 kN (15 m) - $\Sigma M C = 0$; 18 kN (15 m) - $\Sigma M C = 0$; 18 kN (15 m) - $\Sigma M C = 0$; 18 FGX (4 m) -16 kN(5 m) + 22 kN(10 m) = 0 Σ Fy = 0; + Ay -16 kN (2) - 8 kN + E y = 0 FGX = -35 kN ¬ C Ay = 22 kN + Σ Fx = 0; E x = 0 JT. In addition to the applied load, the support at C settles by 0.1 m. : tCA = Δ CO x (Δ CS + Δ CO) + δ CC XC = Δ C æ 17, 496 ö 1125 çç0 (1728) ÷÷ + (1728) XC = -0.25 çè ø EI EI XC = C y = 14.3 kips 324 kip-ft $MA = 109.2 \text{ kips A B Locating maximum deflection, } \theta = 0: A 1 1 \Delta \theta \text{ Ax} = 0 = -109.2(5.032) + 85.9(3.968) + 2 2 1 2 + 14.3 x + (85.9 x - 14.3 x 2) 2 \text{ Ay} = 21.7 \text{ kips } 12.014 - 12.014 2 - 4(14.590) 2 \Delta C \theta = 0 21.7 \text{ 8 Shear (kips)} + 2 2 1 2 + 14.3 x + (85.9 x - 14.3 x 2) 2 \text{ Ay} = 21.7 \text{ kips } 12.014 - 12.014 2 - 4(14.590) 2 \Delta C \theta = 0 21.7 \text{ 8 Shear (kips)} + 2 2 1 2 + 14.3 x + (85.9 x - 14.3 x 2) 2 \text{ Ay} = 21.7 \text{ kips } 12.014 - 12.014 2 - 4(14.590) 2 \Delta C \theta = 0 21.7 \text{ 8 Shear (kips)} + 2 2 1 2 + 14.3 x + (85.9 x - 14.3 x 2) 2 \text{ Ay} = 21.7 \text{ kips } 12.014 - 12.014 2 - 4(14.590) 2 \Delta C \theta = 0 21.7 \text{ 8 Shear (kips)} + 2 2 1 2 + 14.3 x + (85.9 x - 14.3 x 2) 2 \text{ Ay} = 21.7 \text{ kips } 12.014 - 12.014 2 - 4(14.590) 2 \Delta C \theta = 0 21.7 \text{ 8 Shear (kips)} + 2 2 1 2 + 14.3 x + (85.9 x - 14.3 x 2) 2 \text{ Ay} = 21.7 \text{ kips } 12.014 - 12.014 2 - 4(14.590) 2 \Delta C \theta = 0 21.7 \text{ 8 Shear (kips)} + 2 2 1 2 + 14.3 x + (85.9 x - 14.3 x 2) 2 \text{ Ay} = 21.7 \text{ kips } 12.014 - 12.014 2 - 4(14.590) 2 \Delta C \theta = 0 21.7 \text{ 8 Shear (kips)} + 2 2 1 2 + 14.3 x + (85.9 x - 14.3 x 2) 2 \text{ Ay} = 21.7 \text{ kips } 12.014 - 12.014 2 - 4(14.590) 2 \Delta C \theta = 0 21.7 \text{ 8 Shear (kips)} + 2 2 1 2 + 14.3 x + (85.9 x - 14.3 x 2) 2 \text{ Ay} = 21.7 \text{ kips } 12.014 - 12.014 2 - 4(14.590) 2 \Delta C \theta = 0 21.7 \text{ 8 Shear (kips)} + 2 2 1 2 + 14.3 x + (85.9 x - 14.3 x 2) 2 \text{ Ay} = 21.7 \text{ kips } 12.014 - 12.014 2 - 4(14.590) 2 \Delta C \theta = 0 21.7 \text{ 8 Shear (kips)} + 2 2 1 2 + 14.3 x + (85.9 x - 14.3 x 2) 2 \text{ Ay} = 21.7 \text{ kips } 12.014 - 12.014 2 - 4(14.590) 2 \Delta C \theta = 0 21.7 \text{ B Shear (kips)} + 2 2 1 2 + 14.3 x + (85.9 x - 14.3 x + 14.3 x$ used in the north-south direction to resist the seismic forces, the value of R equals 8. 15 kips D 35' 20' A C 50' 25' P6.22 VE = 10.36 k (sin 35) = 5.94 k I. B 3m C 2m P12.2 12-3 Copyright © 2018 McGraw-Hill Education. When the support at D is a roller, the horizontal equilibrium of the entire structure requires that the reaction at support A (i.e., the shear of column AB) be equal to zero, As a result, column AB is subjected to an axial load only and beam BD acts as a simply supported beam. A P8.30. F G H 12' 12' D 12' 6 kips C 12 kips 12' B 18 kips 3 @ 18' = 54' P4.44 Structure Unstable. 6-22 Copyright © 2018 McGraw-Hill Education. A P9.28. 1.3: w = Σ PN 30 k (3) k ft = = 0.9375 96 ¢ L (EQN. 27.5 δ FV (in.) -0.554 δ FV (in.) -0.435 $(41.75)(8.246 \ 2)] \Delta CX = \Delta CX = 1577.67$ AE Tack CX as Redundant Compute $\delta CC = \delta CC = Force$ Due to Unit Value of Redundant 49.48 AE Compatability Eq. $\Delta CX + \delta CC = 0.1577.67$ AE Tack CX as Redundant Compute $\delta CC = 0.1577.67$ AE Tack CX as Redundant 49.48 AE Compatability Eq. $\Delta CX = 0.1577.67$ AE Tack CX as Redundant Compute $\delta CC = 0.1577.67$ AE Tack CX as Redundant 49.48 AE Compatability Eq. $\Delta CX = 0.1577.67$ AE Tack CX as Redundant 49.48 AE Compatability Eq. $\Delta CX = 0.1577.67$ AE Tack CX as Redundant Compute $\delta CC = 0.1577.67$ AE Tack CX as Redundant 49.48 AE Compatability Eq. $\Delta CX = 0.1577.67$ AE Tack CX as Redundant 49.48 AE Compatability Eq. $\Delta CX = 0.1577.67$ AE Tack CX as Redundant 49.48 AE Compatability Eq. $\Delta CX = 0.1577.67$ AE Tack CX as Redundant 49.48 AE Compatability Eq. $\Delta CX = 0.1577.67$ AE Tack CX as Redundant 49.48 AE Compatability Eq. $\Delta CX = 0.1577.67$ AE Tack CX as Redundant 49.48 AE Compatability Eq. $\Delta CX = 0.1577.67$ AE Tack CX as Redundant 49.48 AE Compatability Eq. $\Delta CX = 0.1577.67$ AE Tack CX as Redundant 49.48 AE Compatability Eq. $\Delta CX = 0.1577.67$ AE Tack CX as Redundant 49.48 AE Compatability Eq. $\Delta CX = 0.1577.67$ AE Tack CX as Redundant 49.48 AE Compatability Eq. $\Delta CX = 0.1577.67$ AE Tack CX as Redundant 49.48 AE Compatability Eq. $\Delta CX = 0.1577.67$ AE Tack CX as Redundant 49.48 AE Compatability Eq. $\Delta CX = 0.1577.67$ AE Tack CX as Redundant 49.48 AE Compatability Eq. $\Delta CX = 0.1577.67$ AE Tack CX as Redundant 49.48 AE Compatability Eq. $\Delta CX = 0.1577.67$ AE Tack CX as Redundant 49.48 AE Compatability Eq. 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F 8 kips I w = 3 kips/ft 9' A 2I B C 2I I I 16' 16' 6' P11.18 FEMCD = -8(6) - 2(6)(3) = -84 kip-ft K AB = K BC = 2I 1 = , 16 8 DFBA = DFBC = DFCB = -65.1 - 0.06 - 1.07 - 64 1 8 1 8 + 121 MAB C C FEM 1 8 + 121 MA12 1 8 1 10 1 8 + + -59.9 -0.12 0.33 -2.14 6 -64 MBA D D FEM 1 10 = 2 7 75.5 0.04 -0.06 0.66 -1.07 12 64 MBC D C D C D FEM BA 5/14 3/5 C -84 2/7 E 0 -1.7 -0.1 -1.8 FEM D D MBE 0 -0.85 -0.05 -0.9 FEM C C MEB D 10' E FEM AB = FEM BA = -FEM CB = w = 2 kips/ft -84 FEM MCD D 11-22 Copyright © 2018 McGraw-Hill Education. occurs w/hoist @ C = 0.667(60 k) = 40 M Max. 80 kN P9.36. -9.047 -243.39 8.995 M16 M17 M18 1 54.233 -11.081 22.292 3 54.233 -11.081 54.233 -11.08 161.854 0 -11.081 -44.191 B -100.249 88.336 0 -11.17 44.429 C -95.756 -.13 0 47.521 -11.17 -22.588 Total -400 5 47.521 -1 162.68 0 Joint.Displacements Joint Label X Translation Rotation (mm) (mm) (radians) A 0 0 -2.135e-1 B 0 0 -2.135e-1 B 0 0 -2.13e-1 F 906.276 .971 -2.611e-2 G 905.605 -.265 -3.064e-2 H 904.831 0 -3.59e-2 I 904.4337 -.976 -2.618e-2 K 992.263 1.079 -5.811e-3 L 992.064 -.297 - 5.925e-3 M 991.847 0 -5.82e-3 N 991.657 .3 -6019e-3 O 991.604 -1.086 - 5.955e-3 13-38 Copyright © 2018 McGraw-Hill Education. i.c. Kz 152 Face 2 140' E (a) Compute Variation of Wind Pressure on Windward qz = qs IK z K zt K d leeward C p = -0.5 qz = 49.05(1.52) = 74.556 GC p = 74.556 GC p = 74.556 (0.85)(-0.5) Eq 2.8 Eq 2.6a = 0.00256(140)2 p = -31.68 psf qs = 50.176 psf; Round to 50.18 psf ANS. B 12' I = 300 in.4 I = 75 in.4 A 0.48" 24' P10.24 IC = 4 IC a; E = 29,000 ksi $\theta C = -0.481 = (Counter Clockwise) 24(12) 600 \theta B = Unknown \acute{e} 2 EIC ù \acute{u} \acute{e} \theta ù \acute{e} 12(12) \acute{u} e B û e û \acute{e} 2 E(2) ਪ \acute{u} e B û e û e 2 E(2) U e B û e 0 e 2 E(2) U e$ \ddot{o} \dot{u} \hat{e} 2 θ B + 0.016 - 3c; -1 $\div \dot{u}$ (3) M BC = \hat{e} \hat{e} 24(12) \dot{u} \hat{e} \ddot{e} 600 $\div \phi$ \dot{u} \ddot{e} \hat{e} \hat{e} 0.016) + θ B - 3c; (4) MCB = \hat{e} $\div \dot{u}$ \hat{e} 600 $\div \phi$ \dot{u} \hat{e} 2 E (4 IC) \dot{u} \hat{e} \hat{e} \hat{u} \hat{e} 1 \hat{u} 2 θ B \dot{u} \hat{e} 2 E (4 IC) \dot{u} \hat{e} \hat{e} \hat{u} \hat{e} 1 \hat{u} 2 θ B \dot{u} \hat{e} 2 E (4 IC) \dot{u} \hat{e} \hat{e} \hat{u} \hat{e} 1 \hat{u} 2 θ B \dot{u} \hat{e} 2 E (4 IC) \dot{u} \hat{e} \hat{e} \hat{u} \hat{e} 1 \hat{u} 2 θ B \dot{u} \hat{e} 2 E (4 IC) \dot{u} \hat{e} \hat{e} \hat{u} \hat{e} 2 \hat{u} \hat{e} \hat{e} \hat{u} \hat{e} \hat{u} \hat{e} \hat{u} \hat{e} 2 \hat{u} \hat{e} \hat{u} \hat{e} \hat{u} \hat{e} \hat{u} \hat{e} \hat{u} \hat{u} \hat{e} \hat{u} \hat{u} \hat{e} \hat{u} \hat{u} \hat{u} \hat{e} \hat{u} $\hat{$ -0.007 rad Substitute θ B Back into slope Deflection Equations 2(29000)75 é ù ë-0.007 û 144 M BA = 2 M AB 29000(75) é 1 ù ê 2(-0.007) + 0.016 + ú M BC = ê 36 200 ú û 29000(75) é 1 ù ê 2(-0.007) + 0.005 û 36 M AB = M AB = -12.62 ft·k M BA = -35.24 ft·k M BC = 35.24 ft·k M BC = 35.24 ft·k M BC = 151.04 ft·k 10-27 Copyright © 2018 McGraw-Hill Education. Slotted truss connections at D and F have been designed to act as rollers and transmit vertical force only, and the connection at E is designed to act as a pin. at Joint B M AB = M AB + M AD = 0 (1) M BA + M BC = 0 (2) From Eqns. FBD "DE" + 21.18 III. Computer analysis of a truss. E C D 6' 35 kips hinge 3' B 10' 6' 8' A P5.42 40 kips FBD segment DEF C Σ M F = 8Dy + 6 Dx = 0 4 Dx = - Dy 3 Dx Dy 35 kips E Dy F Fy By FBD segment DEF 35.9 - 30.6 $-135.6\ 1.35'\ Fx = -22.59\ kips \neg FBD\ segment\ AB\ Fx = -22.6\ Shear\ (kips)\ \Sigma Fx = Bx + 35 - 22.59\ Fy = 16.94\ kips\ 22.6 - 22.6\ Dx = 22.59\ Kips\ \neg \Sigma Fy = By - 4 \cdot 10 + 16.94\ Fx\ Bx\ 37.2\ Moment\ (kip-ft)\ \Sigma M\ A = M\ A + 12.41 = 0\ M\ A = -74.46\ kip\ ft\ Ay = 23.06\ kips\ Ax = -12.41\ kips\ \neg MA = -74.5\ 90\ 0\ 90\ 0\ Deflected\ Shape\ 5-44\ Copyright\ Copyri$ Hill Education. Consider the beam in Figure P10.14 without the applied load. The cross section of the rectangular ring in Figure P11.15 is 12 in. M 2 M A B L P7.3 + Compute RA : $\Sigma M M x 2L d 2 y M x æ c M x + 01 E L 2 d x M x = M - RA x = M - E L 2 d x M x = M - E$ = (1) (2) Mx 2 Mx 3 + C1x + C2 (3) 2 12 L Substi y = 0 @ x = 0 in Eq (3) 0 = 0 - 0 + 0 + C2 < C2 = 0 Substi y = 0 @ x - L in Eq (3) ML2 ML2 + C1 L 2 12 5 C1 = - ML 12 dy Mx 2 5 = Mx - ML EI dx 4 L 12 2 3 Mx Mx 5 - MLx EIy = 2 12 L 12 0 = (2 a) Compute Dmax : Set dy = 0 in Eq 2a dx to locate position Dmax Mx 2 5 - ML 4 L 12 5L x 2 - 4 Lx = - (4 a) Compute Dmax : Set dy = 0 in Eq 2a dx to locate position Dmax Mx 2 5 - ML 4 L 12 5L x 2 - 4 Lx = - (4 a) Compute Dmax : Set dy = 0 in Eq 2a dx to locate position Dmax Mx 2 5 - ML 4 L 12 5L x 2 - 4 Lx = - (4 a) Compute Dmax : Set dy = 0 in Eq 2a dx to locate position Dmax Mx 2 5 - ML 4 L 12 5L x 2 - 4 Lx = - (4 a) Compute Dmax : Set dy = 0 in Eq 2a dx to locate position Dmax Mx 2 5 - ML 4 L 12 5L x 2 - 4 Lx = - (4 a) Compute Dmax : Set dy = 0 in Eq 2a dx to locate position Dmax Mx 2 5 - ML 4 L 12 5L x 2 - 4 Lx = - (4 a) Compute Dmax : Set dy = 0 in Eq 2a dx to locate position Dmax Mx 2 5 - ML 4 L 12 5L x 2 - 4 Lx = - (4 a) Compute Dmax : Set dy = 0 in Eq 2a dx to locate position Dmax Mx 2 5 - ML 4 L 12 5L x 2 - 4 Lx = - (4 a) Compute Dmax : Set dy = 0 in Eq 2a dx to locate position Dmax Mx 2 5 - ML 4 L 12 5L x 2 - 4 Lx = - (4 a) Compute Dmax : Set dy = 0 in Eq 2a dx to locate position Dmax Mx 2 5 - ML 4 L 12 5L x 2 - 4 Lx = - (4 a) Compute Dmax : Set dy = 0 in Eq 2a dx to locate position Dmax Mx 2 5 - ML 4 L 12 5L x 2 - 4 Lx = - (4 a) Compute Dmax : Set dy = 0 in Eq 2a dx to locate position Dmax Mx 2 5 - ML 4 L 12 5L x 2 - 4 Lx = - (4 a) Compute Dmax : Set dy = 0 in Eq 2a dx to locate position Dmax Mx 2 5 - ML 4 L 12 5L x 2 - 4 Lx = - (4 a) Compute Dmax : Set dy = 0 in Eq 2a dx to locate position Dmax Mx 2 5 - ML 4 L 12 5L x 2 - 4 Lx = - (4 a) Compute Dmax = - (4 a) Compute Dmax : Set dy = 0 in Eq 2a dx to locate position Dmax Mx 2 5 - ML 4 L 12 5L x 2 - 4 Lx = - (4 a) Compute Dmax = - (4 L2) 12 7 2 2 (x - 2L) = L 3 x = 0.4725L 0 = Mx - Substi x = 0.4725L into (Eq.3a) M éê (0.4725L) 2 (0.4725L of 4 kips/ft over the entire length of the top chord, determine the magnitude of the bar forces in members AD and CD. VF = -0.7(59.7k) + 0.714(110.2) - 0.7(2 k/ft)(14.3c) VF = 99.2 k M F = -59.7k (35c) + 110.2 k (14.3c) VF = -0.7(59.7k) + 0.714(110.2) + 0.7(2 k/ft)(14.3c) VF = -0.7(59.7k) + 0.714(110.2) + 0.7(2 k/ft)(14.3c) VF = -0.7(59.7k) + 0.714(110.2) + 0.7(2 k/ft)(14.3c) VF = -0.7(59.7k) + 0.7(59.7k) + 0.7(59.7k)moment distribution method. P9.37. 15' 30 k 20 k A Compact fill MA B I = 6000 in.4 C I = 4000 in.4 C I = 4000 in.4 C I = 4000 in.4 C I = 6000 lengthening of the bar? Fixed supports at A and C. E is constant, but I varies as indicated below. Gilbert SOLUTIONS MANUAL CHAPTER 16: MATRIX ANALYSIS OF BEAMS AND FRAMES BY THE DIRECT STIFFNESS METHOD 16-1 Copyright © 2018 McGraw-Hill Education.

w 47 @= (60 k) = 58.75k hoist 48 Ay max, occurs M max, occurs M max, occurs W @= -0.979 (60) = 58.751 k hoist 12-31 Copyright © 2018 McGraw-Hill Education. (1) and (2) (1) MCB + MCD = 0 (2) 1 3 rad., θC = rad. They can be transmitted directly by the continuous girder to the supports or they can be carried upward to the arch by the cables and then transmitted through the arch to the end supports. 20 k 20 k 24 k 24 k A y3 y2 y1 y2 y1 B 6 @ 10' = 60' P6.28 P Reactions are found as: y1 Ax = H Σ M A = 24(10 + 50) + 20(20 + 30 + 40) - By (60) = 0 By = Ay = 54 kips Ay Maximum compression, P, will occur at either end, and resultant reaction will be equal to compressive force. B P8.14. hinge P5.53. RB = 6 kN = RC () Σ Fy = 0 : RA + RB - 4(10) = 0 \ RA = 34 kN () Σ M A = 0 : (+) $\approx 10 \text{ ö M A} - 4(10) \text{ cc} \div \div + 10 \text{ RC} = 0 \text{ è}20 \text{ N A} = 140 \text{ kN} \cdot \text{m}$ () Shear Moment 9-23 Copyright © 2018 McGraw-Hill Education. Shear Case 1 RA = 49.67 kN = Vmax Controls Mmax = 28.5 (10.688 - 8) = 280.6 \text{ kN} \cdot \text{m} Controls Case 2 Case 4 RA = 49 kN Compute Abs. Entire Structure: -14.75k (sin 55) = -12.08k + 10 (35ϕ) + 15 (75ϕ) - C y (100ϕ) = $0 \Sigma M A = 0$; k k VE = $-6.14 k C y = 14.75k + \Sigma F y = 0$; FEC = $10.36 k (\cos 35) = 8.49k + 14.75k (\cos 55) = 8.46 k Ay + 14.75k (\cos 55) = 8.46 k Ay$ 21 - 30 k 15 c FKJX = 139.29 kips Cut Section 1-1 Force in bar LC + $\Sigma MO = 0 = -75$ 15 + FLCY 45 FKJY F 139.29k = KJX \ FKJY = 215 5 FKJY = 27.85k compr. Where $.3L = 0.9 \text{ m} + \Sigma MP$. Compute the maximum value of RA produced by the set of wheel loads.

for all members. = (63)(-1.083)0.82 ft = -48.95 kips 1 kips + 0.25(20) Max. P = 30 kips A w = 3 kips/ft C B 6' 6' RB = 85.49 kips 24' P5.35 5-37 Copyright © 2018 McGraw-Hill Education. P15.5. Continued AE = 250 k/in L C = -0.8, S = 0.6 C 2 = 0.64 S 2 = 0.36 CS = -0.48 Member 6 : 3478 é 0.64 -0.48 0.36 0.48 0.36 0.48 -0.36 0.48 -0.36 0.48 -0.64 0.48 0.36 0.48 0.36 0.48 -0.36 0.48 -0.36 0.48 -0.64 0.48 0.36 0.48 -0.36 0.48 -0.36 0.48 -0.36 0.48 -0.36 0.48 -0.36 0.48 -0.36 0.48 -0.36 0.48 -Copyright © 2018 McGraw-Hill Education.

Consider Joint F Force in Bar FC: FFCy 3m 4m 3m 5m FFC B FFCx B 4m 3m C 42 -42 18 42 F F 60 kN -42 FFE 60 kN -42 FFE 60 kN +2 FFE 60 k Vol. Classify the structures in Figure P5.53. Given: E = 29,000 2 4 kips/in. The equation for the parabolic arch is y = 4hx2/L2. 8-20 Copyright © 2018 McGraw-Hill Education. 5 6 7 4 1 8 1 3 9 2 120' 10 1 11 20 19 18 48 kips 17 16 15 14 13 continuous girder 10 @ 36' = 360' P6.33 Uniformly distributed dead load + 48-kip live load Reactions (kips), Deflections (in), and Deflected Shape 6-39 Copyright © 2018 McGraw-Hill Education. Continued K zt = 1.0, K d = 0.85, I = 1 P = qh G C p For h = 24 ¢; qh = qz = 14.36 lb/ft 2 For Wall qs = 0.00256 (100)2 = 25.6 lb/ft 2 qz = qs IK z K zt K d 0 -15¢; qz = 25.6 (1)(0.57)(1)(0.85) = 12.40 lb/ft Cp -0.2 for wall 0.6 for roof For Wall P = 14.36 (0.85)(0.2) P = 2.44 lb/ft 2 2 15-16 ¢; qz = 13.49 lb/ft 2 For Roof P = qz GCP Wall 0 - 15¢ P = 14.36 (0.85)(-0.6) = -7.32 lb/ft 2 (uplift) h = 24; qz = 14.36 lb/ft 2 Wind Pressure on Windward Wall & Roof P = qz GCP Wall 0 - 15¢ P = 12.40 × 0.85 × 0.80 P = 8.43 psf Wall, 15¢-16 ¢ P = 13.49 × 0.85 × 0 2-15 Copyright © 2018 McGraw-Hill Education. w = 1 kip/ft 1' 2' A C D 24' B E 12.5' 12.5' P3.18 + $\Sigma Fx = 0$; + $\Sigma M B = 0$; $\Sigma Fy = 0$; + $\Sigma M B = 0$; $\Sigma H = 0$; Moment Distribution Load @ 1 Distribution Factors: æ I ö3 I K AB = $c_{c} \div = ce^{18} \div a 4 24$ FBD: K BC = $+ \Sigma M B = 0$; RA (18¢) -1k (12¢) + 1.33 = 0 DFBC = 0.5 RA = +0.593 Load @ 2: Pab 2 -1(6)(12)2 = (18)2 L2 = +2.67 FEM AB = +Pa 2 b L2 = +1.33 Load @ 2: Pab 2 -1(6)(12)2 = (18)2 L2 = +2.67 FEM AB = (c) + 1.67 = 0 RA = +0.241 Load @ 3 FBD: + Load @ 3 SM = 0; RA (18¢) + 1.5 = 0 -PL = -3 8 +PL = +3 8 FEM BC = RA = -0.083 FEMCB 12-53 Copyright © 2018 McGraw-Hill Education. Consider both tension and compression. Spring CD has a stiffness of 5 kips/in. horizontally to the right. P5.48. Compute the reactions and bar forces in all members for the truss in Figure P9.24 if a downward load of 120 kips is applied at joint E. w = 3.6 kN/m B C A D 20 m 20 m P13.9 Approximate Analysis of Frame Since columns are stiff relative to the grider, assume P.I. in girder are located' 0.2L from ends. Compute the reactions at A and D. Mid-Span Moment 348 ftk 2-25 Copyright © 2018 McGraw-Hill Education

Continued 6.6 kips -50.8 kip-ft 6.6 kips 50.8 kip-ft 6.6 kips 50.8 kip-ft C 11.4 kips B 19.4 kips C 11.4 kips B 19.4 kips Ex = 11.4 kips B 19.4 kips D 19.4 kips D 19.4 kips D 19.4 kips Ex = 11.4 kips B 19.4 kips D 1 = -82 kip-ft MF = -132 kip-ft Ay = 6.6 kips Ey = 6.6 kips Scale by: 30/42.2 = 0.711 4.7 kips 36 kip-ft C 8.1 kips 13.8 kips 16 ft B 30 kips -72.0 kip-ft 4.7 kips 36 kip-ft 2.0 kip-ft 4.7 kips 13.8 kips -72.0 kip-ft 4.7 kips 21.9 kips 13.8 kips 13.8 kips 13.8 kips 13.8 kips 14.7 kips 21.9 kips 13.8 kips E Fx = 13.8 kips MA = -58.3 kip-ft D Ex = 8.1 kips ME = -58.3 kip-ft MF = -93.8 kip-ft Ay = 4.7 kips 11-36 Copyright © 2018 McGraw-Hill Education. P14.4. Analyze the beam in Figure P14.4. After member end moments are determined, compute all reactions and draw the moment diagrams. 25(12) L 3 4 1 2 0.48 -0.64 -0.48ù 3 é 0.24 ê ú ê 0.48 0.36 -0.48 -0.32 ú 4 ê ú K 2 = 200 ê ú1 0.64 0.48 0.64 0.48 ê ú ê -0.48 -0.36 0.48 0.36úú 2 ê ê û 15-6 Copyright © 2018 McGraw-Hill Education. E hinge B F 50' A r = 35' C 35° P12.23 FBD "FC" y A = 50 ¢ sin 35 = 28.68¢ x AB = 50 ¢ cos35 = 40.96 ¢ + æ 35 ö θ F = sin-1 çç ÷÷ = 44.43¢ è 50 ø x BF = 50 cos θ F = 35.7 ¢ Σ M F = 0; Entire Structure: $\Sigma F(NF) = 0$; $+\Sigma M A = 0$; $C x (28.68 c) - C y (90.96) + PA c = 0 N F - 0.714 C x - 0.7C y + 0.714 P = 0 N F = 0.714 C x + 0.7C y - 0.714 P (1) \Sigma F(VF) = 0$; FBD "BC": C x (35 c) - C y (14.3 c) + P (50 c - X 2) + M F = 0 M F = -C x (35 c) + C y (14.3 c) + C $0.7 P + \Sigma M B = 0$; $C \ge (50 \ c) - C \ge (50 \ c) + P \cdot x^2 = 0 C \ge -0.0292 Px^2 + 0.0161 PA \ c) + PA \ c) = -0.0292 Px^2 + 0.0161 PA \ c) + PA \ c) = -0.0292 Px^2 + 0.0161 PA \ c) + PA \ c) = -0.0292 Px^2 + 0.0161 PA \ c) + PA \ c) = -0.0292 Px^2 + 0.0161 PA \ c) + PA \ c) = -0.0292 Px^2 + 0.0161 PA \ c) + PA \ c) = -0.0292 Px^2 + 0.0161 PA \ c) + PA \ c) = -0.0292 Px^2 + 0.0161 PA \ c) + PA \ c) = -0.0292 Px^2 + 0.0161 PA \ c) + PA \ c) = -0.0292 Px^2 + 0.0161 PA \ c) + PA \ c) = -0.0292 Px^2 + 0.0161 PA \ c) + PA \ c) = -0.0292 Px^2 + 0.0161 PA \ c) + PA \ c) = -0.0292 Px^2 + 0.0161 PA \ c) =$ support, and the structure is symmetrical, lateral load is distributed equally to the columns. Joint I + $\Sigma Fx = 0$; FDIX - 50 = 0 F FDIX = 50 kN tension 50 - 40 + FDIY = 0 FDIX = 50 kN tension 50 - 40 + FDIY = 0 FDIX = 50 kN tension 50 - 40 + FDIY = 0 FDIX = 50 kN tension 50 - 40 + FDIY = 0 FDIX = 50 kN tension 50 - 40 + FDIY = 0 FDIX = 50 kN tension 50 - 40 + FDIY = 0 FDIX = 50 kN tension 50 - 40 + FDIY = 0 FDIX = 50 kN tension 50 - 40 + FDIY = 0 FDIX = 50 kN tension 50 - 40 + FDIY = 0 FDIX = 50 kN tension 50 - 40 + FDIY = 0 FDIX = 50 kN tension 50 - 40 + FDIY = 0 FDIX = 50 kN tension 50 - 40 + FDIY = 0 FDIX = 50 kN tension 50 - 40 + FDIY = 0 FDIX = 50 kN tension 50 - 40 + FDIY = 0 FDIX = 50 kN tension 50 - 40 + FDIY = 0 FDIY = -10 kN tension 50 - 40 + FDIY = 0 FDIY = -10 kN tension 50 - 40 + FDIY = 0 FDIY = -10 kN tension 50 - 40 + FDIY = 0 FDIY = -10 kN tension 50 - 40 + FDIY = 0 FDIY = -10 kN tension 50 - 40 + FDIY = -10 kN tension 50 + 40 + FDIY = -10 kN tension 50 + 40 + FDIY = -10 kN tension 50 + 40 + FDIY = -10 kN tension 50 + 40 + FDIY = -10 kN tension 50 + 40 + FDIY = -10 kN tension 50 + 40 + FDIY = -10 kN tension 50 + 40 + FDIY = -10 kN tension 50 + 40 + FDIY = -10 kN tension 50 + 40 + FDIY = -10 kN tension 50 + 40 + FDIY = -10 kN tension 50 + 40 + FDIY = -10 kN tension 50 + 4 limit the deflection to 0.25 in, the minimum required area is 6.7 in.2. Rounding to the nearest whole number makes the required area 7 in.2. 4-56 Copyright © 2018 McGraw-Hill Education. RD A P12.2. For the beam shown in Figure P12.2, draw the influence lines for the reactions MA and RA and the shear and moment at point B. The 2 2 area of all bars = 2 in. 150 kips P8.5. The pin-connected frame in Figure P15.4. Use the partitioned matrix to compute θA: Orgin Range MP MQ A $0 \le x1 \le 10\ 30x\ 1 - x/30\ A\ 10 \le x2 \le 20\ 300\ 1 - x/3\ C\ 0 \le x3 \le 10\ 30x\ x/30\ 10\ 20\ 10\ a$ are are a convergence of the equation of 2018 McGraw-Hill Education. Given: I = 1200 in. 40 kips w = 4 kips/ft B D A C 6' 4' 10' P5.15 5-17 Copyright © 2018 McGraw-Hill Education. P12.50. ... Following conditions exist. The roof truss is bolted to a reinforced masonry pier at A and connected to an elastomeric pad at C. K33 P16.4. Continued ¢ + M AB ¢¢ = 0 + M AB = M AB æ-44.735ö 6 EI eccentering centering cekip-ft MBC = M BC è EI ϕ + 12 cè EI ϕ + 12 cè EI ϕ + 22 cè EI ϕ + 38.877 + 64 EI ϕ + 38.877 + 64 EI ϕ + 30 + 0 c EI \phi + 30 + 0 c EI ϕ + 30 + 0 c EI ϕ + 30 + kip-ft 2 8 ¢ + MDC ¢¢ = 0 MDC = M DC æ ö æ ö 6 EI çæ-8.528÷ö ç 38.877 ÷ 2 EI ç-44.735÷ çè EI ϕ +÷ + 8 èç EI ϕ +÷ + 8 èç EI ϕ +÷ = M DC = -10.4 kip-ft 2 8 Ay = 10.2 Ax = 6 MA = 11.5 -6 37 10.2 -21.6 -9.8 +0 èç EI ϕ +÷ + 8 èç EI ϕ +÷ = M DC = -10.4 kip-ft 2 8 Ay = 10.2 Ax = 6 MA = 11.5 -6 37 10.2 -21.6 -9.8 +0 èç EI ϕ +÷ = M DC = -10.4 kip-ft 2 8 Ay = 10.2 Ax = 6 MA = 11.5 -6 37 10.2 -21.6 +0 èç EI ϕ +÷ = M DC = -10.4 kip-ft 2 8 Ay = 10.2 Ax = 6 MA = 11.5 +0 èç EI ϕ +÷ = M DC = -10.4 kip-ft 2 8 Ay = 10.2 Ax = 6 MA = 11.5 +0 èç EI ϕ +÷ = M DC = -10.4 kip-ft 2 8 Ay = 10.2 Ax = 6 MA = 11.5 +0 èç EI ϕ +÷ = M DC = -10.4 kip-ft 2 8 Ay = 10.2 Ax = 6 MA = 11.5 +0 èç EI ϕ +÷ = M DC = -10.4 kip-ft 2 8 Ay = 10.2 Ax = 6 MA = 11.5 +0 èç EI ϕ +÷ = M DC = -10.4 kip-ft 2 8 Ay = 10.2 Ax = 6 MA = 11.5 +0 èç EI ϕ +÷ = M DC = -10.4 kip-ft 2 8 Ay = 10.2 Ax = 6 MA = 11.5 +0 èç EI ϕ +÷ = M DC = -10.4 kip-ft 2 8 Ay = 10.2 Ax = 6 MA = 11.5 +0 èç EI ϕ +÷ = M DC = -10.4 kip-ft 2 8 Ay = 10.2 Ax = 6 MA = 11.5 +0 èç EI ϕ +÷ = M DC = -10.4 kip-ft 2 8 Ay = 10.2 Ax = 6 MA = 11.5 +0 èç EI ϕ +÷ = M DC = -10.4 kip-ft 2 8 Ay = 10.2 Ax = 6 MA = 11.5 +0 èç EI ϕ +÷ = M DC = -10.4 kip-ft 2 8 Ay = 10.2 Ax = 6 MA = 11.5 +0 èç EI ϕ +÷ = M DC = -10.4 kip-ft 2 8 Ay = 10.2 Ax = 6 MA = 11.5 +0 ex +0 lines for the moment and the vertical reaction RA at support A for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for the beam in Figure P12.51. Draw the shear and moment diagrams for 30 kips 13.93 kip • ft A 9' 4.64 kips w = 4 kips/ft B C 6' 17.97 kips 6' D 9' 40 kips P5.37 5-39 Copyright © 2018 McGraw-Hill Education. 6m 50 kN A 200 kN • m B 9m C 3m P11.4 Pb 2 a 50(3)2 (6) == -33.3 k · ft 2 L 92 Pba 2 = 2 = 66.7 k · ft L FEM AB = FEM BA Shear (kN) Moment (kN·m) Deflected Shape 11-5 Copyright © 2018 McGraw-Hill Education. 6m 50 kN A 200 kN • m B 9m C 3m P11.4 Pb 2 a 50(3)2 (6) == -33.3 k · ft 2 L 92 Pba 2 = 2 = 66.7 k · ft L FEM AB = FEM BA Shear (kN) Moment (kN·m) Deflected Shape 11-5 Copyright © 2018 McGraw-Hill Education. 6m 50 kN A 200 kN • m B 9m C 3m P11.4 Pb 2 a 50(3)2 (6) == -33.3 k · ft 2 L 92 Pba 2 = 2 = 66.7 k · ft L FEM AB = FEM BA Shear (kN) Moment (kN·m) Deflected Shape 11-5 Copyright © 2018 McGraw-Hill Education. 6m 50 kN A 200 kN • m B 9m C 3m P11.4 Pb 2 a 50(3)2 (6) == -33.3 k · ft 2 L 92 Pba 2 = 2 = 66.7 k · ft L FEM AB = FEM BA Shear (kN) Moment (kN·m) Deflected Shape 11-5 Copyright © 2018 McGraw-Hill Education. 6m 50 kN A 200 kN • m B 9m C 3m P11.4 Pb 2 a 50(3)2 (6) == -33.3 k · ft 2 L 92 Pba 2 = 2 = 66.7 k · ft L FEM AB = FEM BA Shear (kN) Moment (kN·m) Deflected Shape 11-5 Copyright © 2018 McGraw-Hill Education. 6m 50 kN A 200 kN • m B 9m C 3m P11.4 Pb 2 a 50(3)2 (6) == -33.3 k · ft 2 L 92 Pba 2 = 2 = 66.7 k · ft L FEM AB = FEM BA Shear (kN) Moment (kN·m) Deflected Shape 11-5 Copyright © 2018 McGraw-Hill Education. 6m 50 kN A 200 kN • m B 9m C 3m P11.4 Pb 2 a 50(3)2 (6) == -33.3 k · ft 2 L 92 Pba 2 = 2 = 66.7 k · ft L FEM AB = FEM BA Shear (kN) Moment (kN·m) Deflected Shape 11-5 Copyright © 2018 McGraw-Hill Education. 6m 50 kN A 200 kN • m B 9m C 3m P11.4 Pb 2 a 50(3)2 (6) == -33.3 k · ft 2 L 92 Pba 2 = 2 = 66.7 k · ft 2 L 92 Pba 2 = 2 = 66.7 k · ft 2 L 92 Pba 2 = 2 = 66.7 k · ft 2 L 92 Pba 2 = 2 = 66.7 k · ft 2 L 92 Pba 2 = 2 = 66.7 k · ft 2 L 92 Pba 2 = 2 = 66.7 k · ft 2 L 92 Pba 2 = 2 = 66.7 k · ft 2 L 92 Pba 2 = 2 = 66.7 k · ft 2 L 92 Pba 2 = 2 = 66.7 k · ft 2 L 92 Pba 2 = 2 = 66.7 k · ft 2 L 92 Pba 2 = 2 = 66.7 k · ft 2 L 92 Pba 2 = 2 = 66.7 k · ft 2 L 92 Pba 2 =

Next sketch the deflected shape of the frame

P = 16 kips P11.9. Analyze by moment distribution. E C 6' 55 kips 6' B 15' 12' A P3.22 By 10 kips/ft B Bx C 6 ft 12 ft Dy Fx A B Ax Bx A[MA 15 ft 12 ft Dy Fx A B Ax Bx A[MA 15 ft 12 ft Dy Fx A B Ax Bx A] HA 15 ft 12 ft Dy Fx A B Ax Bx A[MA 15 ft 12 ft Dy Fx A B Ax Bx A] HA 15 ft 12 ft Dy Fx A B Ax Bx A[MA 15 ft 12 ft Dy Fx A B Ax Bx A] HA 15 ft 12 ft Dy Fx A B Ax Bx A[MA 15 ft 12 ft Dy Fx A B Ax Bx A] HA 15 ft 12 ft Dy Fx A B Ax Bx A[MA 15 ft 12 ft Dy Fx A B Ax Bx A] HA 15 ft 12 ft Dy 15 $\Sigma Fx = Dx + Fx = 0$ Figure (b): Fx = -87.2 kips $\neg \Sigma M B = 6(55) + 7.5(10)(15) - 12 Dx + 15 Dy = 0 0 = 1455 - 12 Cx + 15 Dy 0 = 1455 - 12 Cx +$ kips ΣM A = M A -12 Bx = 0 M A = -386.4 kip-ft (CW) 3-23 Copyright © 2018 McGraw-Hill Education. Also E = 200 GPa and 6 4 I = 25 10 mm for all members.

B C A hinge 9' 8' D 18' 6' P8.37 Range Mg (ft k) Mp (ft k) A In2 -51.75 9' 4 160 3 8 -51.75 9' 4 160 5 8 -86.25 10' 3 - Fp (k) - 3 8 9x - 0 - Segment Orgin δ BV AB A $0 \le x_1 \le 9 x 2 27 x - BC C 0 \le x_2 \le 9 CD - x 2 0$ (ft.) L (ft) Fg (k) Computing 4x2 2 L AE (1x - 51.75)(18'12) = -0.096 in $\neg 4(29,000) O \cdot \delta$ CH = Σ FO · FP 1k · δ CH L dx + $\Sigma \circ MO M P AE$ EI æ 5 öæ (-86.25)(10 '12) ö÷æç 3 öé -51.75(18'12) ù ÷ ÷ ç ç ú+ = ç- ÷ ç ÷ + ç ÷ ÷ ê çè 8 ÷ øèç ç ÷ ÷ ê úû 3E 4E ø è8ø ë 9 9 æxö æ x ö dx dx + ò çç ÷ ÷ 9 x çè 2 ÷ ø EI EI 1k · δ BV = UQ = Σ FQ FP δ BV 0 P-System for δ BV , δ CH Q-System for δ BV 0 2156.25 1047.94 13.5 éë 9ùû (1728) + + + 3(160)E E E 45(9)3 (1728) (1728) + + + 3(160)E E E 45(9)3 (1728) (9)4 (1728) + = 3(160)E + (3.2)2 = 6.8k Beams "EF", "FG" and "GH" are simply supported (pinned each end) \ uniform load 4 k/ft is distributed by tributary area. 36(6) 12 M = 18 - 18 = 0 M = 12 + 6 - 5-4 Copyright © 2018 McGraw-Hill Education. A beam that is part of a rigid frame has end moments for dead, live, and earth-quake loads shown below. Ignore axial deformation of the beam. F P12.38. (a) Compute all reactions and draw the shear and moment curves for the beam in Figure P10.11. w = 18 kips/ft 34 kips 1 1 12' 2 12' 2 18' P5.56 RISA 2D - Results From Computer Analysis: 1y = -1.9 kips 3y = 122.7 kips M3 = 345.2 kip \cdot ft Maximum Deflections: Shear (kips) Moment (kip-ft) 0.112 in at 14.875 ft from left -0.193 in at 32.375 ft from left Deflected Shape 5-60 Copyright © 2018 McGraw-Hill Education. B 10' C 5' D 5' E 10' F 10' P12.54 Qualitative Influence Lines 12-58 Copyright © 2018 McGraw-Hill Education. = +67.2ft k 2 æ 1 öæ 1ö ç - RA max. The multispan girder in Figure P3.37 has two shear plate connections that act as hinges at C and D. This means AB and BF are much more influential in controlling the deflection of point F.

For all bars, A = 2 in. Continued FBD "AB" + $\Sigma MC = 0$; $2(751 \cdot k) - VBC = 7kK + \Sigma M B = 0$; $A = 75ft \cdot k$ (b) Risa Results for Fixed Bases: (a) Risa Results for Fixed Bases: (a) Risa Results for Fixed Bases: (b) Risa Results for Fixed Bases: (c) Risa Results for Fixed Bases Results for Fixed Bases Results for Fixed Bases Results for Fixe McGraw-Hill Education. 6 kips $(6 \text{ k})8\emptyset + (6 \text{ k})16 \& - R \text{ fy } 16 \& = 0 \text{ R fy} = 9 \text{ kips}$ Compute Rhy using entire truss: + $\Sigma Fy = 0$; - 6k(4) + 4.5k + Rhy + 9k = 0 Rhy = 10.5 kips 3-17 Copyright @ 2018 McGraw-Hill Education. Also estimate the shear and moment at each end of beams IJ and JK. It supports light machinery with an operating weight of 4000 lbs, centrally located. P12.43. I = 360 in.4 12' 30 kips w = 2.4 kips/ft B C I = 600 in.4 12' I = 360 in.4 A 12' 12' P11.32 FEM AB = -FEM BA -2.4(12) = -28.8 kip-ft 12 = -FEMCB 2 Case A -73.3 ... -3.2 6.5 -12.3 24.6 -90 Laterally Restrained = FEM BC = -30(24) = -90 kip-ft 8 & 0.6 I \ddot{o} \div 3 K AB = KCD = ccc \div 3 K AB = KCD = ccc \div 3 K AB = -FEM BA -2.4(12) = -28.8 kip-ft 12 = -FEMCB 2 Case A -73.3 ... -3.2 6.5 -12.3 24.6 -90 Laterally Restrained = FEM BC = -30(24) = -30(24) = -90 kip-ft 8 & 0.6 I \ddot{o} \div 3 K AB = KCD = ccc \div 3 K AB = -FEM BA -2.4(12) = -28.8 kip-ft 12 = -FEMCB 2 Case A -73.3 ... -3.2 6.5 -12.3 24.6 -90 Laterally Restrained = FEM BC = -30(24) = FEM D MAB 73.3 ... 3.2 -6.5 12.3 -24.6 90 MBC ... C D C D FEM A 28.8 14.4 22.2 5.8 1.5 ... 73.3 MCB ... C D C D FEM C D D D ... MCD 15 kips = Dy 0.0375 = 0.474 0.07917 8.3 kips = D x 2.4 kips/ft 20.5 kips RB = -41 kips DFBC = DFCB = FEM D MDC 0.474 DFBA = DFCD = 0.474 0.07917 8.3 kips = D x 2.4 kips/ft 20.5 kips RB = -41 kips DFBC = DFCB = FEM D MDC 0.474 DFBA = DFCD = 0.474 0.07917 8.3 kips = D x 2.4 kips/ft 20.5 kips RB = -41 kips DFBC = DFCB = FEM D MDC 0.474 DFBA = DFCD = 0.474 0.07917 8.3 kips = D x 2.4 kips/ft 20.5 kips RB = -41 kips DFBC = 0.474 0.07917 8.3 kips = D x 2.4 kips/ft 20.5 kips RB = -41 kips DFBC = 0.474 DFBA = DFCD = 0.474 DFBA Diagram (kips) Moment Diagram (kip-ft) Observation: A continuous uniformly distributed load, acting on a parabolic arch will produce a resultant axial compression force acting at the centroid of all cross sections. We assume that the tributary area for each beam is shown by the dashed lines in the Figure P5.55, and the weight of the beams and their fireproofing is estimated to be 80 lb/ft. D P5.15. If the sag at midspan of each cable is 4 ft, determine the tensile force each cable applies to the compression ring.

Compute the deflections at points B and C. The beam span is 30 ft. Use IBC = 200 in. 15 kN 20 kN 24 kips P4.4. Determine the forces in all bars of the trusses. 2 E is constant and equals 30,000 kips/in. RB = 3.187k 9-19 Copyright © 2018 McGraw-Hill Education. P4.55. C D A B 4' P = 10 kips 9' 4' P5.4 For 4 ft < x < 13 ft Σ Fy = 0 = -V (x) + 10 - 5 x 5 kips/ft V (x) = -5 x + 10 kips M(x) A V (13) = VC = -55 kips æxö $\Sigma M z = 0 = -M (x) - 5 x cc \div \div + 10 (x - 4) + 20 kip \cdot ft 2 M (13) = MC = -332.5 kip \cdot f$ with an exact analysis. occurs w/hoist @ = (60 k) = 58.75 k 48 k Ax, Max.

Determine the sag at points B and D.

(ft) 0 35 70 105 140 p (psf) 34.36 39.69 44.69 48.03 50.70 2-17 Copyright © 2018 McGraw-Hill Education. The diagonal brace member CH is pinned at each end.

Since deflections of beams and onestory rigid frames are due almost entirely to moment and not significantly affected by the area of the member's cross-section, substitute the gross area in the Member Properties Table. Compute all reactions and draw the shear and moment curves.

1 P12.27. A simply supported crane runway girder has to support a moving load shown in Figure P12.49. EI 20/EI $\Delta 3 \Delta 2 C 12/EI \Delta 1 1$ kip xB () 4 8 (12 + 20) $\Delta 2 = \theta B LBC = (15) EI 5 2 1920 4 = (1728) = 0.6636$ in. Compute the reactions and draw the shear and moment curves for the beam in Figure P12.49. EI 20/EI $\Delta 3 \Delta 2 C 12/EI \Delta 1 1$ kip xB () 4 8 (12 + 20) $\Delta 2 = \theta B LBC = (15) EI 5 2 1920 4 = (1728) = 0.6636$ in. Compute the reactions and draw the shear and moment curves for the beam in Figure P12.49. Hill Education. (a) Find the horizontal deflection at joint B produced by the 40-kip load in Figure P8.14. 13-27 Copyright © 2018 McGraw-Hill Education. 20 kips 24 k By 24 kips 20 kips 20 kips 20 kips 20 kips 20 kips 24 kips Hy = M 65.6(y1) = 540 y1 = 8.23 ft 6 @ 10 ft = 60 ft 65.6(y2) = 840 54 y2 = 12.8 ft 30 ft 30 ft 30 ft 30 ft 30 ft 30 kN C H I J 6m 30 kN 12 kN B K 6m L 8m 8m A 8m P4.24 + Σ M L = 0; - 30(8 m) - 6(18) -12(12) -12(6) + Ay (8) = 0 Ay = 70.5 kN + Σ Fy = 0; - 30 - 70.5 + Ly = 0 Ly = 100.5 kN + Σ Fy = 0; - 30 - 70.5 + Ly = 0 Ly = 100.5 kN + Σ Fy = 0; - 30 - 70.5 + Ly = 0 Ly = 100.5 kN + Σ Fy = 0; - 30 - 70.5 kN + Σ Fy = 0; Moment Distribution D. Though the truss appears to be a very different structural system with it's depth and moment of inertia significantly higher than the girder = 1350, the analysis approach is the same. Extend FGC to G and break into components. 6 kN/m 20 kN 3m 12 kN C 3m A 4m 4m P3.20 Dy Dx 20 kN 3m D 6 kN/m 12 kN 3m Ax Z A Ay 4m 4m 2M Z = 4(Ay) + 3(12) + 2(6)(4) + 20(4) = 0 Ay = -41 kips Σ Fy = Ay - (6)(4) - 20 + Dy = 0 Dy = 85 kips Dx 4 = (Similar Triangles) Dy 3 Dx = 113.3 kips D = Dx 2 + Dy 2 D = 141.6 kips Σ Fx = Ax + 12 + Dx = 0 Ax = -125.3 kips -3-21 Copyright © 2018 McGraw-Hill Education. wearing surface of $k (36 \ c) = 0 \ CE \ y = 10 \ k \ CE \ x \ CE \ y = 3 \ 4 + \Sigma Fx = 0$; Shear & Moment Curves MBR "ABCD": CE $x = 7.5k \ 30 \ k - BE \ x + 7.5k = 0 \ BE \ x = 37.5k \ \neg \ \Sigma Fy = 0$; + Section 1-1 30 kips - 40 k + BE y + 10 k - 20 k = 0 BE y = 50 k Support "E": 5-52 Copyright © 2018 McGraw-Hill Education. Force in Bar KJ. Plot the deflected shape. The coefficient of -6 temperature expansion $\alpha t = 6.6 \times 10$ (in./in.)/°F 2 and E = 29,000 kips/in. 12' P14.1. The structure in Figure P14.1 is composed of three pin-connected bars.

For the truss in Figure P8.12, compute the vertical displacement at joint G. (c) Repeat (b) if segment AB has 2.5EI. also $\Delta B = \Delta E = \Delta 2$. Since all vertical cables carry a portion of the live load applied to the stiff continuous girder, the distribution on the arch's cross section

Construct the envelope of maximum positive shear assuming the beam supports a 6 kN/m uniformly distributed load of variable length. Abs. P11.18. Take the origin at A. 4' B C 12' A D 20' P10.28 EI is Constant Unknowns: θB , θD , $\Psi EI = K 12 EI KA = = 6K 20$ For Columns KC = For Girders PB 2 A 30(16)2 (4) = = -76.8 kip \cdot ft L2 20 2 PBA2 30(16) (4)2 = = 19.2 kip · ft 2 20 2 L FEM BC = FEMCB (1) Use Slope-Defl. × 2 in. Determine the maximum reaction applied to an end support due to dead, live, and impact loads. ; girder A = 16.2 in. 100 kN K L M N O4 m F G H I J A B C D E 300 kN 6m 5m 8m 8m P13.19 Analysis by Portal Method (a) Cut Horizontal Section through P.I. in Second Story $0.053 - 0.112 \ 0.007 - 0.191 \ 0.066 \ 0 \ 0 \ 0.206 - 0.034 \ 0 \ 0 - 0.217 - 0.364 \ 0.172 \ \delta Cx = -0.217 \text{ in. wI } 2 \ \epsilon 12 \ E = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = = -90 \ kip \cdot \text{ft } 8 \ PL \ FEM23 = -90 \ kip \cdot 12 \ FEM23 = -90$ settles 0.6 in. 8 kips 12' D E 24 kips 12' D E 24 kips 8 A 16' 16' P4.9 + Σ MA = 0; 8 (24 ¢) + 36 (16 ¢) + 24 (12 ¢) - By (32 ¢) = 0 k k 5.32 \Sigma Fy = 0; 4 12 ft Ay = 3k A 7 9.33 36 kips + FCEX = 5.33k E -4 16 ft Ay = 3 kips 8k (24 ¢) - FCEY (48¢) = 0 By = 33 kips 8 kips FCEY = 4 k FCEY F = CEX 12 ¢ $16 \notin 38.67 29 \text{ B} 16 \text{ ft} \Sigma MA = 0$; 24 kips +29.33 +38.67 Ax = 32 kips Force in Bar CE: Consider Joint C D FCE FAC E 24 ft FCE A FCEy B FCEx 4-10 Copyright © 2018 McGraw-Hill Education. 2 T = 159.4 kips (OK) 54 34 Therefore, sag is found from the General Cable Theorem. w = 5 kips/ft P10.23. For the steel frame in Figure P8.41, compute the horizontal displacement of joint B. at Joint C MCB + MCD = 0(2) M BA + M BC = 0(1) From Equil.

A C B 9' D 12' 6' P8.36 Compute: $\delta CY WQ = \delta 6 1k \cdot \delta Cy = \delta 0 MQ M P dx FP L EI AE 12 (-1 \cdot x)(-16 x) dx (-0.5 x)(-8 x) dx (-0.5 x)(-8 x) dx (-0.5 x)(-8 x) dx (-0.5 x)(-8 x) dx (-0.5 x)(-16 x) dx (-0.5 x)(-0.5 x)(-$ Increase in Length 8-42 Copyright © 2018 McGraw-Hill Education. 10 kips/ft P3.22. (a) Determine the moments, shear, and axial forces in the members of the frame in Figure P13.18, using the portal method. Joints B and D are rigid. $P = 4(0.45 \text{ wL}) = 4(0.45 \text$ HG. EQN.

150 kips P8.6. For the pin-connected frame in Figure P8.5, in addition to the vertical loads a lateral load of 30 kips also acts to the right at joint B.

 $RA = RO = 20 \text{ kN } 2. \text{ Continued Axial Force Diagram (kips)} \text{ Relative Displacement between 1st and 2nd floors} = 0.136 \text{ n.} + D E F G L M pinned connections typical A K 6 @ 5' = 30' P13.21 \Sigma Fy = 0; \ Ax = Kx = 5k \neg Ay = 6.67k Ky =$ H B Assume the lateral load is shared equally to each column Entire Structure $\Sigma M K = 0$; C 10 kips -M BA - 5k (15¢) = 0 FBD Col. and 4 I = 240 in. P P10.1. Using Equations 10.12 and 10.13, compute the fixed-end moments for through 4, 5 in. 30' B P8.26. D 30 kips C 12' B E 16' A F 25' P13.17 The figure to the right of its specified location. 6-36 Copyright © 2018 McGraw-Hill Education. 13-8 Copyright © 2018 McGraw-Hill Education. The exterior columns, which are uninsulated, are exposed to the outside ambient temperature. Again, determine all the forces acting at each joint of the arch, the joint displacements, etc., and compare results with those in (a). B 4m C D 4m A 200 kN P9.31 L AE $\Delta CO = \Sigma FQ = AE \Delta A = 0.00469 \text{ m} = 4.69 \text{ mm} = 4.69 \text{ mm} = 4.69 \text{ m}$ Compatibility Eq. ΔCO + RC δCC = 0 Q-System 9.66 -1131.82 + RC = 0 AE AE RC = 117.17 kips Δ AX = 0 (Symmetry) Final Result 9-33 Copyright © 2018 McGraw-Hill Education. A cable ABCD is pulled at end E by a force P (Figure P6.12). 1 Condition Eq. at Hinge \ Indet 1º Remove 3 Restraints: Cut @ E \ Indet 3º 5-58 Copyright © 2018 McGraw-Hill Education. A cable ABCD is pulled at end E by a force P (Figure P6.12). 1 Condition Eq. at Hinge \ Indet 1º Remove 3 Restraints: Cut @ E \ Indet 3º 5-58 Copyright © 2018 McGraw-Hill Education. A cable ABCD is pulled at end E by a force P (Figure P6.12). 1 Condition Eq. at Hinge \ Indet 3º 5-58 Copyright © 2018 McGraw-Hill Education. A cable ABCD is pulled at end E by a force P (Figure P6.12). 1 Condition Eq. at Hinge \ Indet 3º 5-58 Copyright © 2018 McGraw-Hill Education. A cable ABCD is pulled at end E by a force P (Figure P6.12). 1 Condition Eq. at Hinge \ Indet 3º 5-58 Copyright © 2018 McGraw-Hill Education. A cable ABCD is pulled at end E by a force P (Figure P6.12). 1 Condition Eq. at Hinge \ Indet 3º 5-58 Copyright © 2018 McGraw-Hill Education. A cable ABCD is pulled at end E by a force P (Figure P6.12). 1 Condition Eq. at Hinge \ Indet 3º 5-58 Copyright © 2018 McGraw-Hill Education. A cable ABCD is pulled at end E by a force P (Figure P6.12). 1 Condition Eq. at Hinge \ Indet 3º 5-58 Copyright © 2018 McGraw-Hill Education. A cable ABCD is pulled at end E by a force P (Figure P6.12). 1 Condition Eq. at Hinge \ Indet 3º 5-58 Copyright © 2018 McGraw-Hill Education. A cable ABCD is pulled at end E by a force P (Figure P6.12). 1 Condition Eq. at Hinge \ Indet 3º 5-58 Copyright © 2018 McGraw-Hill Education. A cable ABCD is pulled at end E by a force P (Figure P6.12). 1 Condition Eq. at Hinge \ Indet 30 Copyright © 2018 McGraw-Hill Education. A cable ABCD is pulled at end E by a force P (Figure P6.12). 1 Condition Eq. at Hinge \ Indet 30 Copyright © 2018 McGraw-Hill Education. A cable ABCD is pulled at end E by a force P (Figure P6.12). 1 Condition Eq. at Hinge \ Indet 30 Copyri Hill Education. If supports A and E in Figure P8.26 are constructed 30 ft and 2 in. P3.2. Determine the reactions for the structure.

Continued Equilibrium of Column AB: MBA = 240 kip-ft ΣM A = 0 = 240 - VBA (16) B VBA = 15 kips A VAB = V2 = 15 kips From Joint B: ΣFx = 0 = VBC - VBA + FBE FBE = 0 (Right side of frame internal forces are all equal and opposite to left side forces.) Summary of Cantilever Method Member End Forces 90 kip-ft SM A = 0 = 240 - VBA (16) B VBA = 15 kips A VAB = V2 = 15 k ft 90 kip-ft 30 kips -15 kips (C) 15 kips 15 kips 26.4 kips 26.4 kips 26.4 kips 26.4 kips (C) 7.2 kips (T) 90 kip-ft 12 ft 90 kip-ft 15 kips 33.6 kips 15 kips 33.6 kips (T) 240 kip-ft 25 ft 33.6 kips (C) 7.2 kips (C) 7.2 kips (T) 90 kip-ft 16 ft 15 kips 33.6 kips 15 kip same for this frame. Determine the reactions and all bar forces for the truss in Figure P9.34. A B 9' C 6' P9.1 Δ CO (Use Moment-Area) 1 324 17, 496 Δ CO = (9)'12 = EI 2 EI PL3 1(15)3 1125 = δ CC = EI 3E 3EI See Table A.3 Compatibility Equation 17496 1125 + RC = 0 EI EI RC = 15,552 kips Δ C = 0; - RA = 36 - RC = 20.448k 1 90.72 93,312 (10.563)(4.437) + EI 2 EI 2 291.57 θ C = EI θ C = -9-2 Copyright © 2018 McGraw-Hill Education. P13.21.

1 kip/ft P3.6. Determine the reactions for the structure. P6.2. The cable in Figure P6.2 supports four simply supported girders uniformly loaded with 6 kips/ft. B C 2m 3m F D E A 30 kN 3m 3m P4.40 Σ Fx = 0 \ Ax = 0 Cut Section (2)–(5) Σ Fy = 0; Σ M f = 0; Ay - 30 = 0 Ay = 30 kN Ay 3 - FBC 2 = 0 FBC = + Σ Fx = 0; 3 A = 45 kN 2 Y - FBC + FFE0 FFE = FBC = 45 kN Entire Structure $\Sigma M D = 0$; 30 - 30 - 30 + RD = 0; 30 - 30.393(60) = 0.2315.71 ft 15° Ay ¢ = 3.93 kips 1 30 ΣMC 3.93(30) MC 0.197(30) 2.3 MC 88.26 kip-ft 0.197 kips/ft 0.393 kips/ft MC HhC 88.26 H (4) C H 22.1 kips Equilibrium of the cable: 1 60 ΣM B = 22.1(12) - 0.393(60) + Ay (60) 2.3 Ay = -0.49 kips A'y B'y MC Moment (kip-ft) H 6 ft Wring = 24(0.49) = 11.8 kips By 1 ΣFy = -0.393(60) - 0.49 + By = 0 2 By = 12.3kips 4 ft H Ay Tmax = H 2 + By2 = 22.12 + 12.32 30 ft 30 ft Tmax = 25.3 kips A req = 25.3 = 0.23 in 2 110 6-16 Copyright © 2018 McGraw-Hill Education. Continued Axial Load in girders depends on Axial Stiffess. For all segments of the 2 4 arch A = 70 in. 60 kips P12.51. 4m P6.32. P10.31. B P4.25. Compute the slope at support A and the deflection at B in Figure P8.18. The weight of the girder is included in the 4 kips/ft uniform load. 13-54 Copyright © 2018 McGraw-Hill Education. = $0.556\ 16\ 80\ I\ 4I\ K\ BC = = 20\ 80\ D.F$. (Equal to Case B scaling factor) 7.73 M BA = $-173\ kip-ft\ M\ BC = -73.3 + 46.4(5.30) = -172.6$ » M BC = -73.3 + 46.4(5.30) = -172.6 » M BC = $-173\ kip - 100\ km^2$ kip-ft MCB = 73.3 - 46.4(5.30) = -172.6 » MCB = -173 kip-ft MCD = -73.3 + 46.4(5.30) = 172.6 » MCD = -73.3 + 46.4(5.30) = -172.6 » MCD = -73.3 + 46.4(5.30) = 172.6 » MCD = -73.3 + 46.4(5.30) = -172.6 » MCD = -73.4 + 46.4(5.30) = -172.6 » MCD = -73.4 + 46.4(5.30) = -172.6 = -172.6 + 46.4(5.30) = -172.6 = -172.6 + 46.4(5.30) = -172.6 + 46.4(5.30) = -172.6 + 46.4(5.30) = -172.6 + 46.4(5.30) = -172.6 + 46.4(5.30) = -172.6 + 46.4(5.30) = -172.6 + 46.4(5.30) = -172.6 + 46.4(5.30) = -172.6 + 46.4(5.30) = -172.6 + 46.4(5.30) = -172.6 + 46.4(5.30) = -172.6 + 46.4(5.30) = -172.6 + 46.4(5.30) = -172.6 + 46.4(5.30) = -172.6 + 46.4(5.30) = -172.6 + 4 $\delta BB RB + \delta BM M B \theta B = 0 = \theta BO + \alpha BY RB + \alpha BM M B In Terms of Displacements L3 L2 - WL4 RB M + 8EI 3EI 2 EI B L3 L - WL3 0 = R + M 6 EI 2 EI B EI B 0 = Solving Gives: RB = WL 2 MB = WL2 12 9-21 Copyright © 2018 McGraw-Hill Education. FCG = +9.75 kips Tension 12-40 Copyright © 2018 McGraw-Hill Education. D 3' A P12.10. P = 100$ kips 15' B 15' D A E 20' 20' P8.1 All Bars: A = 4 in 2, E = 24,000 ksi BAR Fp FQ1 (δ By) FQ2 (δ Bx) L FpFQ1L (δ By) FQ2L (δ Bx) AB BC CD DE AE BE BD 0 + 125 - 75 0 0 + 150 - 125 0 0 0 0 0 + 0.75 - 1.25 25' 25' 30' 20'' 15'' 25'' - - - - - - 1687.5 3906.25 \Sigma = -2250 \Sigma = 5593.75 \deltaBy : Σ Q1 · δ By = Σ FQ1 FP L AE -2250 -2250(12%) 1 $\cdot \delta By = = AE 4(24000) \delta By = 0.281 \text{c}$ k $\delta Bx : \Sigma O2 \cdot \delta Bx = \Sigma FO2$ Fp L P-System for $\delta Bx = 0.70 \text{ c}$ O1-System for 6.713 75.195 4 -.089 6.713 81.908 M1 1 -161.854 52.056 0 5 -.089 6.713 88.621 2 -161.854 52.056 78.084 1 -89.032 100.047 0 3 -161.854 52.056 156.168 2 -89.032 100.047 150.071 4 -161.854 52.056 156.168 2 -89.032 100.047 150.071 4 -161.854 52.056 156.168 2 -89.032 100.047 150.071 4 -161.854 52.056 78.084 1 -89.032 100.047 0 3 -161.854 52.056 156.168 2 -89.032 100.047 150.071 4 -161.854 52.056 156.168 2 -89.032 100.047 150.071 4 -161.854 52.056 156.168 2 -89.032 100.047 150.071 4 -161.854 52.056 78.084 1 -89.032 100.047 0 3 -161.854 52.056 156.168 2 -89.032 100.047 150.071 4 -161.854 52 600.2832 - 26.84620.7564.621 - 16.2425.9659.723 - 26.84620.7564.621 - 16.2425.965356844 - 26.84620.75666.8834 - 16.2425.96565113.5782162.6851.89205 - 16.2425.965113.5782162.6851.89205 - 16.2425.96565113.5782162.6851.89205 - 16.2425.96565113.5782162.6851.89205 - 16.2425.96565113.5782162.6851.89205 - 16.2425.96565113.5782162.6851.89205 - 16.2425.9655113.5782162.6851.89205 - 16.2425.96565113.5782162.6851.89205 - 16.2425.96565113.5782162.6851.89205 - 16.2425.9655113.5782162.6851.89205 - 16.2425.9655113.5782162.6851.89205 - 16.2425.96565113.5782162.6851.89205 - 16.2425.96565113.5782162.6851.89205 - 16.2425.96565113.5782162.6851.89205 - 16.2425.9655113.5782162.6851.89205 - 16.2425.9655113.5782162.6851.89205 - 16.2425.9655113.5782162.6851.89205 - 16.2425.9655113.5782162.6851.89205 - 16.2425.9655113.5782162.6851.89205 - 16.2425.9655113.5782162.6851.89205 - 16.2425.9655113.5782162.6851.89205 - 16.2425.9655113.5782162.6851.89205 - 16.2425.9655113.5782162.6851.89205 - 16.2425.9655113.5782162.8851.89205 - 16.2425.96551.89205 - 16.2425.9655113.5782162.8851.89205 - 16.2425.9655113.5782162.8851.89205 - 16.2425.9655113.5782162.8851.89205 - 16.2425.9655113.5782162.8851.89205 - 16.2425.9655113.5782162.8851.89205 - 16.2425.9655113.5782162.8851.89205 - 16.2425.9655113.5782162.8851.89205 - 16.2425.9655113.5782162.8851.89205 - 16.2425.96551 $162.68\ 51.892\ 233.515\ 3\ 268.7\ -135.007\ 5\ 162.68\ 51.892\ 311.354\ 4\ 268.7\ -135.007\ -177.806\ 1\ 27.409\ 21.556\ 2.27.409\ 21.556\ 2.27.409\ 21.556\ 2.27.409\ 21.556\ 4.839\ 3\ 193.462\ -62.437\ 118.297\ 4\ 27.409\ 21.556\ 4.839\ 3\ 193.462\ -62.437\ -135.007\ -345.565\ 2\ 27.409\ 21.556\ 4.839\ 3\ 193.462\ -62.437\ -135.007\ -345.565\ 2\ 27.409\ 21.556\ 4.839\ 3\ 193.462\ -62.437\ -135.007\ -345.565\ 2\ 27.409\ 21.556\ 4.839\ 3\ 193.462\ -62.437\ -135.007\ -345.565\ 2\ 27.409\ 21.556\ 4.839\ 3\ -135.007\ -345.565\ 2\ 27.409\ 21.556\ 4.839\ -345.565\ 2\ 27.409\ 21.556\ 4.839\ -345.565\ 2\ 27.409\ 21.556\ 4.839\ -345.565\ -2.437\ -135.007\ -345.565\ -2.437\ -135.007\ -345.565\ -2.437\ -135.007\ -345.565\ -2.437\ -135.007\ -345.565\ -2.437\ -135.007\ -345.565\ -2.437\ -135.007\ -345.565\ -2.437\ -135.007\ -345.565\ -2.437\ -135.007\ -345.565\ -2.437\ -135.007\ -345.565\ -2.437\ -135.007\ -345.565\ -2.437\ -135.007\ -345.565\ -2.437\ -135.007\ -345.565\ -2.437\ -135.007\ -345.565\ -2.437\ -345.56\ -2.437\ -345.56\ -2.437\ -345.56\ -2.437\ -345.56\ -2.437\ -2.437\ -2.437\ -2.437\ -2.437\ -2.437\ -2.437\ -2.437\ -2.437\ -2.437\ -2.437\ -2.437\ -2.437\ -2.437\ -2.437\ -2.437\$ 135.271 347.173 3 15.766 25.01 61.776 2 30.336 -135.271 178.084 4 15.766 25.01 86.787 3 30.336 -135.271 -160.094 1 -.13 95.756 0 5 30.336 -135.271 -128.084 4 15.766 25.01 111.797 4 30.336 -135.271 -128.084 4 15.766 25.01 111.797 4 30.336 -135.271 -128.084 -13 -.13 95756 574.535 4 79.244 -26.845 -33.786 1 -.089 6.713 61.769 5 79.244 -26.845 67.344 M2 M3 M4 M5 M6 M7 M8 M9 M10 M11 M12 M13 M14 M15 13-37 Copyright © 2018 McGraw-Hill Education. (b) Use the equivalent lateral force procedure to determine the seismic base shear and overturning moment. 3 @ 12' = 36' P2.9. The building section associated with the floor plan in Figure P2.4 is shown in Figure 2 P2.8. Assume a live load of 60 lb/ft on all three floors. Consider only flexural deformations. Draw the shear and moment curves for each member of the frame in Figure P5.42. square cross section running from support A to joint E is added, determine the lateral deflection at joint D. Continued Case B: Sidesway Correction to Eliminate 0.97k Holding Force Multiply Moments in case B by 0.97/6.62 and Add to Case A. P13.31. (c) Tabulate your results showing bar forces, cross sectional areas, and lengths. Analyze the frame in Figure P11.29 by moment distribution. 20 kN/m L1 2 5 kN/m C A B L1 Case 1: L1 = 3 m Case 2: L1 = 12 m 6m P13.2 Check your results by using moment distribution. 40 kN 2m A 15 kN·m B C 4m E D hinge 6m 30 kN·m 2 kN/m 4m 8m P3.14 40 N Cy 15 kN-m 2 kN/m 4m $0 \text{ Cx} = 40 \text{ kips} \neg \text{Figure (b)}$: $\Sigma \text{Me} = -30 - 6(2)(12) + 8\text{Dy} + 12\text{Cy} = 0 \text{ Dy} = 31.5 \text{ kips}$ $\Sigma \text{Fy} = \text{Dy} + \text{Cy} - 12(2) + \text{Ey} = 0 \text{ Ey} = -1 \text{ kips}$ $\Sigma \text{Fx} = \text{Cx} + \text{Ex} = 0 \text{ Ex} = -40 \text{ kips} \neg 3-15 \text{ Copyright } \mathbb{C} \text{ Symbol s}$ $\Sigma \text{Fy} = \text{Dy} + \text{Cy} - 12(2) + \text{Ey} = 0 \text{ Ex} = -40 \text{ kips} \neg 3-15 \text{ Copyright } \mathbb{C} \text{ Symbol s}$ distance above support A and to the right and immediately below joint B. Label all ordinates of the curves. The values of SDS and SD1 are equal to 0.9g and 0.4g, respectively. (b) Compute the vertical deflection of the girder at point C produced by the 60-kip load. Continued L A L (12) A Member FP (k) FQ2 (k) L (ft) A(in 2) AB +150 -5/8 0 25 5 - 5625 0 BC + 60 - - - - - CD - 100 - - - - - CD - 100 - - - - - DE - 90 + 3/8 - 1 15 3 - 2025 + 5400 EA - 90 + 3/8 0 15 3 - 2025 0 EE 0 - - - - - BD - 150 - 5/8 0 25 5 + 5625 0 $\Sigma = -4050 + 5400 \text{ FP}$ (a) $\delta \text{EH L} 5400 = \text{AE} 30,000 = 0.18 \text{ c}$ 1k · $\delta \text{EH} = \Sigma \text{FO} 2 \text{ FP} \delta \text{EH} \delta \text{EV} \text{ L} - 4050 = \text{AE} 30,000 = -0.138 \text{ c}$ 1b · $\delta \text{EV} = 270 \text{ FP} \delta \text{EV}$ (b) δEV Due to settlememt & Fabrication Error: ö æ ö kæ 1 ç 1 ¢¢ ÷ 5 ç 3 ¢¢ ÷ 1k · δEV - ççç ÷÷ = - ççç ÷÷ 2 Bars 2 çèç 4 ÷÷ø 8 ççè 4 ÷÷ø δEV (Settlement) & Fabrication Error) = -0.81 8-17 Copyright © 2018 McGraw-Hill Education. For the trusses in Figure P9.27, compute the reactions and bar forces produced by the applied loads. w WL2 30 A D B L/4 C L/4 L/2 P7.2 1 w 2 æç 1 ö÷ 9 wL2 x ç x ÷ - wLx + 2 L cè 3 ÷ø 60 30 w 3 9wL wL2 M (x) = x + x6L 60 30 d2 y w 3 9wL wL2 = EI x + x6L 60 30 dx dy w 4 9wL 2 wL2 = EI x + x + c1 24 L 120 30 dx w 5 9wL 3 wL2 2 EI y = x + x x + c1 24 L 120 x + c1 4 L 120 x + c1 4 L + + c1 x + c2 120 L 360 60 dy at x = 0, = 0 c1 = 0; and at x = 0, $y = 0 c2 = 0 dx \Sigma M A = 0 = M (x) + dy w 3wL 2 wL 2 = x4 + xx dx 24 EIL 40 EI 60 EI <math>\theta = 532532$ Compute $\Delta B = -w LwL L wL 2 LwL 4 + - = -0.000659 EI 120 EIL 4 0 EI 4$ $= L : 2 \Delta B = -w L wL L wL 2 L wL 4 + - = -0.001302 EI 120 EIL 2 40 EI 2 60 EI 2 \Delta C = 1.98 \Delta B 7-3 Copyright © 2018 McGraw-Hill Education. 5 kN/m 40 kN A B 10 m C 10 m 5m P5.31 Member DE All Forces = Zero Since No Loads Applied Member DE All Forces = Zero Si$ - 25 - 40 + 76.67 = 0 By = 11.67 kN Σ Fx = 0: E D Entire Structure Bx = 0 Member AB 5-33 Copyright © 2018 McGraw-Hill Education. 13-55 Copyright © 2018 McGraw-Deflected Shape 6-37 Copyright © 2018 McGraw-Hill Education. P10.16. (c) Write the shear and moment equations for girder BC. Finally, calculate the debris impact load to be applied to the free-standing column CD. A B L C L P9.13 Comparison of roadway vertical deflections An alternate modification to reduce deflections of the floor system is to add diagonal cables extending from the floor system to the arch as shown in Fig P6.34. The shear plate at B is assumed to act as a hinge. We will assume the P.I. edjecent to B at 0.3L and the one on the right at 0.1L as shown above. P2.18. To prove that the vertical reaction at support 4 is always equal to 75 kips, simply consider the whole structure as a freebody and take moment about support 1.) A (in. 13-4 Copyright © 2018 McGraw-Hill Education. 4 kips/ft P5.42. w3 = 2 kips/ft w1 = 3 kips/ft C A 30 kip • ft B 9' 9' 9' P5.26 + - 30 ft k + 27(4.5) + 13.5(6 ¢) + 36 k (15¢ + 18¢) - By $\Sigma M A = 0$; By = 75.58k $\Sigma Fy = 0$; -27 -13.5 - 2(36) + 75.58 + Ay = 0 + Ay = 36.92 k FBD: Sim-Triangles 3k / ft y = 9¢ x Y = 31 æxö $\Sigma Fy = 0$; 36.92 k - 3x - x cc; $\div \div \div = 0.2$ èc 3 ø V = 0@ x = 8.394 @ x = 9c V = -36.92 k + 3x + 202 cc 3 w = -36.92 cc 3 w $171.4 \text{ ft} \cdot \text{k} - 30 \text{ ft} \cdot \text{k} = 141.5 \text{ ft} \cdot \text{k} \text{ Max} + \text{M} = 36.92 \text{ x}$ o Shear 2 8.4 - Moment Deflected Shape 5-28 Copyright © 2018 McGraw-Hill Education. and 2 E = 29,000 kips/in. Computer Study—comparison of approximate analysis with exact analysis. The functual contract of the arch., and $\alpha = 6.5 \times 10 \text{ (in./in.)}/\text{°F}$. Moment Case 3 Mmax = 24.84 $^{\prime}$ 9.31 = 231.26 kN \cdot m Case 5 32 kN load at £ Resultant = 8 + 32 + 24 = 64 kN R \cdot x = 32 $^{\prime}$ 8 + 24 $^{\prime}$ 11 x = 5.625 m Moment of midspan M £ = 39^{\prime}12 - 24 $^{\prime}$ 8 = 276 kN \cdot m 12-46 Copyright © 2018 McGraw-Hill Education. 30 kips 15 kips/ft 150 kip \cdot ft A hinge D C B spring, k = 30 kips/in. FBD "AB" IV. at Joint B M BA + M BC = 0 162 θ B = EI \ M AB =-58.8 k · ft, M BA = 107.4 k · ft M BC =-107.4 k · ft M BC = 142.2 k · ft Segment BC Shear Moment 10-7 Copyright © 2018 McGraw-Hill Education. 5m P13.19. AB = 0.48 ÷ = 40 4 çè 15 ÷ø 3 æ 600 ö÷ K BC = çç ÷ = 30 4 èç 20 ÷ø 3 æ 200 ö÷ K BD = çç ÷ = 13.33 4 èç 15 ø÷ ΣK S¢ = 83.33 D.F. BC = 0.36 D.F. BD = cç + 13.33 4 èç 15 ø* ΣK S¢ = 83.33 D.F. BC = cc + 13.33 4 èc $0.16 \Delta LTNT = \alpha T (L)\Delta T = 6.6(10-6)(15^{+} + 20 c)(20)(\Delta LAB) = 0.2218cc \Delta 6(29,000)(200)(\Delta LAB) = 8.5 tr k FEM DB = 6 EI 2 = (15 12\%)(12)^{-} L Where \Delta LAB = 6.6(10-6)(15^{+} + 20 c)(20)(\Delta LAB) = 0.09504 cc 11-34 Copyright © 2018 McGraw-Hill Education. 60 kips 25' B 60 kips C D 25' E A 50' 50' P6.24 4h 2 x L2 dy 8hx = 2 dx L @ x = 25c; Y = @ x = 25c; Y = @ x = 25c; Y = (15 12\%)(12)^{-} L Where \Delta LAB = 6.6(10-6)(15^{+} + 20 c)(20)(\Delta LAB) = 0.09504 cc 11-34 Copyright © 2018 McGraw-Hill Education. 60 kips 25' B 60 kips C D 25' E A 50' 50' P6.24 4h 2 x L2 dy 8hx = 2 dx L @ x = 25c; Y = @ x = 25c; Y = (15 12\%)(12)^{-} L Where \Delta LAB = 6.6(10-6)(15^{+} + 20 c)(20)(\Delta LAB) = 0.09504 cc 11-34 Copyright © 2018 McGraw-Hill Education. 60 kips 25' B 60 kips C D 25' E A 50' 50' P6.24 4h 2 x L2 dy 8hx = 2 dx L @ x = 25c; Y = (15 12\%)(12)^{-} L Where \Delta LAB = 6.6(10-6)(15^{+} + 20 c)(20)(\Delta LAB) = 0.09504 cc 11-34 Copyright © 2018 McGraw-Hill Education. 60 kips 25' B 60 kips C D 25' E A 50' 50' P6.24 4h 2 x L2 dy 8hx = 2 dx L @ x = 25c; Y = (15 12\%)(12)^{-} L Where \Delta LAB = 6.6(10-6)(15^{+} + 20 c)(12)^{-} L Where \Delta LAB = 6.6(10-6)(15^{+} + 20 c)(12)^{-} L Where \Delta LAB = 6.6(10-6)(15^{+} + 20 c)(12)^{-} L Where \Delta LAB = 6.6(10-6)(15^{+} + 20 c)(12)^{-} L Where \Delta LAB = 6.6(10-6)(15^{+} + 20 c)(12)^{-} L Where \Delta LAB = 6.6(10-6)(15^{+} + 20 c)(12)^{-} L Where \Delta LAB = 6.6(10-6)(15^{+} + 20 c)(12)^{-} L Where \Delta LAB = 6.6(10-6)(15^{+} + 20 c)(12)^{-} L Where \Delta LAB = 6.6(10-6)(15^{+} + 20 c)(12)^{-} L Where \Delta LAB = 6.6(10-6)(15^{+} + 20 c)(12)^{-} L Where \Delta LAB = 6.6(10-6)(15^{+} + 20 c)(12)^{-} L Where \Delta LAB = 6.6(10-6)(15^{+} + 20 c)(12)^{-} L Where \Delta LAB = 6.6(10-6)(15^{+} + 20 c)(12)^{-} L Where \Delta LAB = 6.6(10-6)(15^{+} + 20 c)(12)^{-} L Where \Delta LAB = 6.6(10-6)(12)^{-} L Where \Delta LAB = 6.6(10-6)(12)$ $4(20)(25) = 5 + 1000 \text{ dy } 8(20)(25) = 0.4 \text{ dx } (100) 2 \text{ Y} = 4 = 0.371 \text{ 10.77 } 10 \cos \theta = 0.928 \text{ 10.77 } \sin \theta = + \Sigma M D = 0; 60(50 \text{ c}) - H(20) - 60(25) = 0 \text{ H} = 75 \text{ k} (a) \text{ dx before the} 1/4 \text{ point } (\text{left of "D"}) + \Sigma Fy = 0; 55.68 \text{ k} - 27.83 \text{ k} - VDL = 0 \text{ VDL} = 27.85 \text{ k} + \Sigma M D = 0; 60(25 \text{ c}) - 75(15) - M = 0 \text{ M} = 375 \text{ k} + \Sigma M D = 0; 60(25 \text{ c}) - 75(15) - M = 0 \text{ M} = 375 \text{ k} + 22.86 \text{ k} - T = 0 \text{ T} = 91.86 \text{ k} (b)$ dx after the 1/4 point (right of "D") Add Components of the 60k Load @ D to the Values in (a) k M = 375 (same) M = 375k T = 91.86 - 22.6 T = 69.3k VDR = 27.83k 6-27 Copyright © 2018 McGraw-Hill Education. P10.27., 4 2 IAD = 400 in. (b) Position the AASHTO HL-93 design tandem and lane loads shown in Figure 12.25(g) to produce the maximum positive moment. Earthquake load can act in either direction, generating both negative and positive moments in the beam. $w = 1.2 \text{ kips/ft A B 2I 12' I 8' P8.23 Segment "BC" MQ1 = 1k x1 M P1 = 1.2 x12 + 6 x1 = 0.6 x12 + 6 x12 = 0.6 x12 + 6 x12 = 0.6 x12 + 6 x12 = 0.6 x12 + 0.6 x$ $0.6(x^2 + 8)^2 + 6(x^2 + 8)\hat{u}\hat{u}\hat{e}\hat{u}^2 EI EI 0 2 1 12 c dx dx + \delta 0.6 x^{23} + 20.4 x^{22} + 211.2 x^2 + 691.2 x^2 +$ 72" P2.1. Determine the deadweight of a 1-ft-long segment of the prestressed, reinforced concrete tee-beam whose cross section is shown in Figure P2.1. Beam is constructed with 3 lightweight concrete which weighs 120 lbs/ft . 5 kips 5' C 15' 1.5I 4I P11.8 FEM BA = 5(9) = 45 kip-ft P (b 2) a 5(10 2) 8 = 12.35 kip-ft L2 182 - P(b)a 2 5(10)82 = -9.88 kip-ft FEMCB = L2 182 P(b)a 2 5(20 2) 5 5(20)52 + = + = 20 kip-ft FEMCD = -FEM DC = L2 L2 252 252 FEM BC = 4I 4 = 25 25 DFCB 5 kips A 1 12 1 12 + 254 5 kips B 9 ft 1 8 ft = 0.34, DFCB 5 kips 10 ft C 0.34 4 25 1 12 + 254 = 0.66 5 kips 5 ft D 15 ft 0.66 5 ft 45 FEM 12.35FEM 45 MBA -57.35 C -3.16 D 0.27 C -0.27 D -45 MBC By = 6.6 kips Dy = -4.2 kips M D = -13.4 kN FEM 20 FEM 12.25 D -20 FEM 6.13 C 0.52 C -13.4 MDC D 3.16 C y = -12.4 kips -9.88 -28.68 C 6.31 D -1.58 C 0.54 D -33.3 MCB 1.04 33.3 D MCD 11-9 Copyright © 2018 McGraw-Hill Education. B 3 8 113.78 = EI 3E (1.5 I) = t BA = xAM/EI \delta BB = x $\Delta \text{ BO AM/EI} = x = \Delta \text{ BO } 10(20) 2(8) = 661.33 3 3 \ddot{o} 2(8) \approx 2 \ddot{o} \div 10(20) \approx 3 cc \div c 10 - 2 \div \phi \Rightarrow 3 ec 4 3 ec 4 \phi \div 661.33 661.33 2446.05 = 5.548 = EI 1.5EI \delta \text{BB C B } 4 \text{ kips/ft} = 5.548 \text{ ft tBA} = \Delta \text{ BO } 1.5EI \text{ CB A } + 2.548 \text{ ft tBA} = \Delta \text{ BO } 1.5EI \text{ CB A } + 2.548 \text{ ft tBA} = 2.548 \text{ ft tBA} = 5.548 \text{ ft tBA} = 2.548 \text{ ft tBA} = 2.548$ = 21.1 kips 9-10 Copyright © 2018 McGraw-Hill Education. 5' 20 kips B 10' w = 2 kips/ft C D 10' A E P11.24 FEM BC = - PL FEMCD = - wL 8 = - 20(10) 8 2 12 = - 16.67 k · ft, FEM DC = 16.67 k · ft, FEM DC = 16.67 k · ft, FEM DC = - 16.67 k · ft, FEM DC = - 16.67 k · ft, FEM DC = - 25 k · ft, FEM DC = - 25 k · ft, FEM DC = - 25 k · ft, FEM CD = - 25 k · ft, FEM CD = - 25 k · ft, FEM DC = - 25 k · Copyright © 2018 McGraw-Hill Education. 6 kN/m A B C 3m 12 m P13.6 wL2 6(12)2 = 72 kN · m FEM BC = M BA Since moments on joint B are equal and opposite, joint does not rotate : Segment BC acts like a fixed-ended beam. The continuous beam in Figure P5.56 is constructed from a W12 × 152 wide 2 flance steel section with A = 44.7 in. (d) Evaluate the maximum moment between points B and C. The slab is protected by a 2-in. = O - Sys.) $\delta BV \delta CV dx dx = \Sigma M P 2 EI EI 15 10 dx 2 dx 12 \cdot \delta CV = \hat{o} (84 - 12 x) + \hat{o} (9.6 x) 2 EI EI 0 0 e 15 e 16 e 126 2 9 3 u 92.16 e 441x \delta CV = e x + x u + e12 EI e 0 0 e 15 e 16 e 0.96 cc O \cdot \delta P = \Sigma M O M P e x3$ \dot{u} $\hat{e}e$ \dot{u} \hat{u} 0 12 10 \dot{u} \dot{u} \dot{u} (1728) \dot{u} \dot{u} $\delta BV = \dot{0}$ (15 - x) (84 -12 x) 0 $\delta BV = \dot{P}$ & O-System for δCV dx EI 2 3 \dot{u} \dot{e} \hat{e} 315 x - 66 x + 3 x \dot{u} (1728) \hat{e} 2 3 \dot{u} \hat{u} \dot{u} \dot{u} taller and it's lateral load is applied at 20ft up from the base compared with the parallel chord truss in problem P13.21, which has a height of 15 ft with it's lateral load applied at the 15 ft top chord. Factor of 1, assume 2 1 M = 306.4 kN \cdot m 2 B Shear in Columns MA = Σ M A = 0; 612.8 + 306.4 · VBA (5) = 0 VBA = VAB = 183.84 kN Reactions 13-13 Treat the shear plate connection at B as a hinge. H P12.40. 15 kips P O N M 12' 25 kips I J K L 12' 25 kips G H F E 16' A B 15' C 15' D 15' P13.18 (a) Analysis by Portal Method 13-28 Copyright © 2018 McGraw-Hill Education. A P12.19. w = 1 kip/ft P9.17. hinge P 40 kN C 3m B D y1 6m A E 4m 4m 6m 3m P6.29 I. B + $\Sigma M B = 0$; M BA + M BC + M BD = 0 (3.1330B + 0.03556\Delta x) EI = 0 0B = -0.011349\Delta x + \neg (5¢) = 0 $\Sigma M A = 0$; M AB + M BA - VBA Segment AB M AB + M BA (1.20B - 0.04\Delta x) EI = 5¢ 5¢ VBA = (0.240B - 0.008\Delta x) EI = 0.011349\Delta x VBA = VBA = -0.0107Δ x EI 10-30 Copyright © 2018 McGraw-Hill Education. If stable, indicate whether determinate or indeterminate. 60 kN P8.31. 16 kips P8.36. The live load may be assumed to act along the centerline of the deck slab and divide equally between the two girders. (a) Sketch the deflected shape of the frame in Figure P6.23. 6 kips B C 3 kips I = 240 in.4 12' I = 120 in.4 18' I = 150 in.4 A D 3@8' = 24 P11.28 Distribution Factors Joint B D.F. 3 I 3 (120) = 7.5 4 L 4 12 240 10 = 24 \Sigma K S¢ = 17.5 K AB = K BC 7.5 = 0.43 17.5 10 0.57 = 17.5 1.00 Joint C K BC FEM BC = 2 PL 2 '6 ' 24 = 32 kip · ft 9 9 KCD D.F. 240 = 10 24 150 8.33 = 18 \Sigma K S¢ = 18.33 10 = 0.55 18.33 8.33 = 0.45 18.33 Case A: No Sidesway Reactions: Case A 11-38 Copyright © 2018 McGraw-Hill Education. 8' A P12.18. 30 kips P9.42. B C 8" 8" 6' 8" D A w = 2 kips/ft 12' P10.12 By symmetry, θ A = -θB = -θD = θC Thus only one unknown FEM AD = WL2 2(12)2 = 24 k·ft 12 12 (a) Member end Moments 2 EI EI = (2θA + θD) + 24 = θ + 24 12 6 A 2 EI EI = (2θA + θB) + 0 = θ 6 3 A M AD = M AB Equilibrium at Joint A M AD + M AB = 0 48 EI = 16 k·ft , $MAB = -16 \text{ k} \text{ ft} \quad \theta A = - \ MAD (b) \text{ Axial force in } AD = 0 \text{ k} \text{ Axial force in } AB = 12 \text{ k} 10-13 \text{ Copyright } \mathbb{C} 2018 \text{ McGraw-Hill Education. } 10' \text{ A MA B C L} = 40' \text{ RA } + 2 \text{ e} 64 \text{ L} 2 \text{ Pba } 3L = 2 = 64 \text{ L} 2 \text{ Pba } 3L = 2 \text{ Pba } 3L \text{ Pba } 3L = 2 \text{ Pba } 3L \text{ Pba } 3L \text{ Pba } 3L = 2 \text{ Pba } 3L \text{$ 30 cc ÷÷ = 85.31k èc 32 ø k 1 Load@ L : 4 1 Load@ L : 2 3 Load@ L : 4 30 kips 10' Influence Lines k 27 9(40) (10) = 2.81 ft · k 32 64 16 8(40) (10) = 0.32 64 5 3(40) (10) = -0.31 ft · k 32 64 12-55 Copyright © 2018 McGraw-Hill Education. Draw the influence lines for bar forces in members HD and HC of the truss shown in Figure P12.33. $\Sigma Fy = 0$; + - (FJKY + FJIY) + FJD = 0 FJD = 55.72 k tension By inspection FBL = 0 4-37 Copyright © 2018 McGraw-Hill Education. B h = 45' 40' rod A C turnbuckle 25' 60' 60' P6.21 Entire Structure: + $\Sigma M A = 0$; 25k (25¢) + 15k (60 ¢) - C y (120 ¢) $= 0 \text{ Cy} = 12.71 \text{k} + \Sigma \text{Fy} = 0$; -25 k - 15 k + cy + Ay = 0 Ay = 27.29 k FBD "AB": $+ \Sigma \text{M} \text{B} = 0$; Ay (60 ¢) - Ax (45¢) - 25 k (35¢) = 0.27.29 [TAC] Ax = Tension in rod "AC" = 16.94 k 6-23 Copyright © 2018 McGraw-Hill Education. (braced frame is very effective in reducing lateral stiffness) 5-63 Copyright © 2018 McGraw-Hill Education. Assume that the diagonals can carry either tension or compression force. Continued + $\Sigma M B = 0 = -RDY B - 103.03(4) + 200' 4 RDY = 48.485 kN (Symmetry) RDX = RDY = 48.485 kN ($ 24 Load @ x = 24: M = (1¢) = 0.5 48 $\Sigma M = 0$; If the hoist load is 60 kips, determine maximum reaction Ay and maximum moment @ and respective locations of the hoist. 30 kN 3m w = 8 kN/m A B RA = 9 kN RB = 105 kN 6m D C RC = 105 kN 6m D C kN m P9.3. Compute the reactions and draw the shear and moment curves for the beam in Figure P9.3. EI is constant.

I = 300 in. Draw the shear and moment curves for each member of the beam in Figure P5.31.

relative to the top of the interior column at point E. to the right. 2 The area of all bars is 5 in.

w P9.19. The internal bars are required for other loads and to brace bars ABCD and DEFG. E = 30,000 kips/in. (a) Draw the shear and moment curves for the frame in Figure P5.40.

(downward) (b) Axial Force (kip) Bending Moment (kip-ft) Axial force in Members 5 and 6: F = 57.04 kips Maximum bending moment (at Joint 6): M = 7.23 kip-ft = 86.76 kip-in $\sigma = F$ Mc 57.04 (86.76)1.5 + = + = 25.46 + 43.09 = 68.55 ksi > 25 ksi (NG) 2.24 3.02 A I 4-61 Copyright © 2018 McGraw-Hill Education. B 15' C 15' P12.8 15 ft Maximum values for reactions & MB occur when hoist load is @ end of cantilever. Vertical members EN and GL are 18-ft long, FM is 16 ft. 400 kip • ft 8 C 12' D A 16' P11.12 3æ I ö K BC = cc ÷÷ 2 cè16 ÷ø 2 DFBA = , 3 DFBC = 1 3 Shear (kips) Moment (k ft) 11-14 Copyright © 2018 McGraw-Hill Education. Deflection @ Midspan Since member end moments established, use slope-defl EQ to compute displacement at midpoint Vertical force transmitted by V. $\theta A = \theta F = \theta$ and $\theta C = \theta D = -\theta$. A P12.52.

Use BCD Compute $\Delta CO : 1kN \cdot \Delta CO = \Sigma \circ M E M P + dx EI RD = 5.463kN \approx 2 \circ 2 \approx x \circ dx dx x c - 6 x - 4 x + ÷ ÷ + o c - (12 x) + o c - (12 x$ $dx dx \div = \delta ccc + x 2 + \delta + \delta (-4 x 2) cc 3 \phi + 2 EI EI EI 6 \Delta CO 0 0 0 3 dx + \delta 2 x 2 - 6 x - 36 EI 0 9 - 14 Copyright © 2018 McGraw-Hill Education. S = 0.97k 6 EI \Delta L2 Assume \Delta is Selected so FEM's = 40 ft + k 5I I K AB = = D.F. P4.54. too low and the support at C is accidentally constructed at a slope of 0.016 rad clockwise from a vertical axis$ through C, determine the moment and reactions created when the structure is connected to its supports. 6' F P13.5. Continued Check result by moment distribution KCB = 3I = 0.0625 4 12 DFCB = 0.6 KCD = 1I = 0.0417 2 12 DFCD = $0.4 \Sigma K = 0.10417 1.0$ (6)2 = $361 \text{ ft} \cdot \text{k} 2$ = FEMCB = FEMCD FEM BA = 2 k/l FEM BC = 2 k/l (12)2 = $24 \text{ ft} \cdot \text{k} 12$ + Mmax Member "CD" + 2 k/l (12 ¢) - VDC (12 ¢) = 0 2 = 12 k \Sigma MC = 0; - 21.6 + 21.6 + VDC = VCD P.I. 14.4 ft · k = 2 (L ¢) / 8 L ¢ = 7.6 ft ¢ ¢ (12 - L) = 2.21 ft from C & D 2 k / ft P.I. @ = 0.184 L f accurate. Indeterminate, stable Determinate, stable b=7 b = 10 (g) r = 3 n = 5 r = 4 n = 7 (b + r = 10) > (2n = 12) (b + r = 12) > (2n = 12) (b conclusions can you draw about the truss? w = 2 kips/ft P16.1. Using the stiffness method, analyze the two-span continuous beam shown in Figure P16.1 and draw the shear and moment curves EI is constant. 18 kips 2" asphalt 8" slab 51 kips 94 kips A slab B C 94

A P4.52 Two Trusses Support Load. (a) Load is applied to the three-hinged trussed arch in Figure P12.40 through the upper chord panel points by a floor beam and stringer floor system. B P8.10. For this configuration, determine the vertical displacement of all the floor system joints. P4.40. 1 P16.2. Write the stiffness matrix corresponding to the degrees of freedom 1, 2, and 3 of the continuous beam shown in Figure P16.2. 2 3 B A KS = 5EI L3 L C D spring L L EI = constant P16.2 Stiffness Matrix K Joint "B": + $\Sigma Fy = 0$; + $\Sigma M B = 0$; 2(12 EI) 5EI + 3 - K11 = 0 L3 L 29 EI K11 = 3 L - K 21 K 21 = 0 Unit Vertical Displacement $\Delta 1 = 16 EI + 2 = 0 L2 L$ Joint "B": Joint "C": Joint "C": + $\Sigma MC = 0$; 6 EI = 0 L2 6 EI K31 = 2 L K31 - Unit Rotation @ Joint "B": + $\Sigma Fy = 0$; -K12 K12 = 0 + $\Sigma M B = 0$; 6 EI 6 EI + 2 = 0 L2 L 2(4 EI) = 0 L 2 EI K32 = L K32 - Joint "C": + $\Sigma MC = 0$; 2 EI = 0 L 2 EI K32 = L K32 - Joint "C": + $\Sigma MC = 0$;

Reactions and moment curves are given. D 6' 4' E C 6' F G 4' H I 16' 6' 16' B A N M L 6 @ 18' = 108' RA P12.31 12-34 Copyright © 2018 McGraw-Hill Education. The loads shown represent the total dead and live loads. 2 The 43 -in. These directions range from approximately 3/4 inches near the supports to nearly 2 inches at midspan where the cables are longest. 100 kN P9.46. 3' 10' 3' hanger hanger 2.5' vertical lateral bracing, located on 4 sides of framing (shown dashed) mechanical unit 5' hanger 2.5' edge of mechanical support framing beyond hanger hanger mechanical unit floor grating mechanical unit floor grating mechanical unit floor grating mechanical support framing beyond hanger hanger = 1.20 (4 kips) = 4.8 k Uniform LL = ((10' × 16') - (5' × 10')) (0.04 ksf) = 4.4 k k Total LL = 9.2 : Total' LL Acting on One Hanger = 9.2 / 4 Hangers = 2.3 k klps c) Total DL Floor Framing = 10' × 16' (0.025 ksf) = 4 k : Total DL + LL on One Hanger = 4.4 Hangers = 1.3 kips 2-12 Copyright © 2018 McGraw-Hill Education. D 4' w = 3 kips/tt P10.9. (a) Under the applied loads supports to reach structure in Figure P10.35. Entire Structure 5.71 - 20 + 28.62 - 30 + RC = 0 RC = 15.67 kN Colclusion: Exact and Approximate values of force in each member of the truss in Figure P13.13. E 4m A B 3m C 3m D 6m P11.21 FEM AC = - PL M 50(6) 150 PL M + = + = 0, FEMCA = + = 75 kN · m 8 4 8 4 4 Moment (kN·m) Shear (kN·m) S

w = 2 kips/ft B C 15' P = 30 kips 10' 10' A D 30' P11.11 Consider Symmetry, No Joint Translation Joint B Distribution Factors I 25 1æ I $\ddot{o} = cc \div \div 2 \ \dot{e}c 30 \ \sigma \text{ K BA } 12 = \Sigma \text{K S} \ 17 \text{ K 5} = BC = \Sigma \text{K S} \ 17 \text{ K 5} = BC = \Sigma \text{K S} \ 17 \text{ K 5} = BC = \Sigma \text{K S} \ 17 \text{ K 5} = BC = \Sigma \text{K S} \ 17 \text{ K 5} = BC = \Sigma \text{K S} \ 17 \text{ K 5} = BC = \Sigma \text{K S} \ 17 \text{ K 5} = BC = \Sigma \text{K S} \ 17 \text{ K 5} = BC = \Sigma \text{K S} \ 17 \text{ K 5} = BC = \Sigma \text{K S} \ 17 \text{ K 5} = BC = \Sigma \text{K S} \ 17 \text{ K BA} = D \cdot \text{FBA} = \text{K DC } D \cdot \text{FBC } \Sigma \text{K S} \ 17 \text{ K 5} = BC = \Sigma \text{K S} \ 100 \text{ K 5} \ 1$

Continued Express equilibrium EQ's in forms of displacements 6.4 K0B + 1.2 K0C - 6 KV = 76.8 1.7 K0B + 6.4 K0C - 6 KV = 76.8 1.7 K0B + 76.8 K0C + 76.8 K

B 4 kips C 12' A 18' P9.43 Selecting Cy as the redundant, $\Delta 2$ the compatibility equation is: ($\Delta CS + \Delta CO$) + δCC XC = $\Delta C \Delta 1 = \theta B LBC = 12 q C \Delta 1 = \theta B LBC = 12 q C + 2 + 12 q C = 12 q C + 2 + 12 q C = 12 q C + 2 + 12 q C = 1$

E P5.23. (a) Determine the wind pressure distribution on the four sides of the 10-story hospital shown in Figure P2.15. P5.8. Write the equations for shear V and moment M in terms of distance x along the length of the beam in Figure P5.8. Take the origin at point A. Continued-Part(a) Member Properties: Moment Diagram (kip-ft) and Deflected Shape: (b) Pin Support at D Moment Diagram (kip-ft) and Deflected shape of AB, CB, FE and DE are identical. 6 kips P5.10.

, and A = 3 in. El ACO = tCA = (ACS + ACO) + bCC XC = -6B iCA = ACO s² 24/EI XC K X (0 - 7.188) + 1.214 XC = -C 5 XC, beem = C y = 5.08 kips AC = -5.08 = -1.02 in. betermine the lactations at points A and D in Figure P10.30. (b) Determine thruss bar forces. (a) All beams of the frame in Figure P13.15 have the same cross section and carry a uniformly distributed gravity load of 3.6 kips/ft. Ex - CBx = 0 CBx = 40 K \neg æ 36 CBy = 40 K s $\varphi + \varphi = 50 + -30 k + 30 k + 30 k + 29 = 0 Dy = 30 k + 49 Copyright © 2018 McGraw-Hill Education. Indicate whether stated analysis. Also determine the lateral deflection of the cable tower. = (1.5(9))1.2 k / ft. For the trusses in Figure P2.8, compute the reactions and bar forces produced by the applied loads. Draw the shear and moment curves for the beam in Figure P5.34. 2-21 Copyright © 2018 McGraw-Hill Education. Entire Structure: <math>+ 2MA = 0$; P (4m) + 20 kN (8m) + 40(14m) - 2 y (17m) = 0 E y = 0.2335P + 42.353 + 2ME = 0; A y (17m) - P (13) - 20(pright © 2018 McGraw-Hill Education. Dy P6.15. P5.54. 85 kip + ft CB A 15' 15' P8.21 1 kip 3 kips/ft P-System (kip-ft) A 85 kip-ft B C 6BV Q-System (kip-ft) - 4.8 30.9 7.5 Shear (kip) + 0.2 kip 2.2 (10 (122) K 60 C) = 0.2 kip 2.2 kip

= RA 18 - 90 k['] 9 ¢ = 0 RA = 45 kips Moment Distribution Results + $\Sigma MC = 0 = 45k(44) + RB 20 - 220(22) RB = 143k \Sigma Fy = 0 = -220 + 45 + 143 + RC RC = 32 kips Results of Approx. Express the equilibrium equations in terms of the appropriate displacements. <math>\Delta BO = (10 \text{ m})(0.005 \text{ rad.}) = 0.05 \text{ m} \delta BB [xRB] \delta BB = 0.0278 \text{ m} (See P9.21) \delta S = 1$ = 0.025 m 40 Compatibility Equation: $\Delta BO + \delta BB RB = -0.025 RB RB = 0.947 \text{ kN}$ () RA = -0.947 kN () RA = -0.947 kN () RA = -0.947 kN () M A = 10 RB = 9.47 kN · m () Moment Shear 9-24 Copyright © 2018 McGraw-Hill Education. w = 2.8 kips/ft P10.5. Compute the reactions at A and C in Figure P10.5. Draw the shear and moment 4 diagram for member BC. 30 kips D 15' A C B 20' 20' P9.33 Select RC as Redundant (a) A = 2 in 2 E = 30,000 kis Geometry $\Delta CY = 0$ Deflections Dco Released Structure w/Applied Load WQ = UQ = ΣFQ FP L/AE é -22.5(18'12) ù $\hat{\omega} + c_{c} \div \hat{e} \hat{u} 1 k \Delta CO = 2 \hat{e} \hat{e} \hat{u} \hat{u} \hat{e} 3 \hat{\sigma} \div \hat{e} \hat{u} \hat{u} AC AE \Delta CO = 0.448k \delta cc WO = UQ = <math>\Sigma FO2$ LAE / Released Structure w/Unit Value of RC also use as Q-System for ΔCO and $\delta CC \approx 5 \ddot{o} (25'12)(2) (2) (15'12) 1k \delta CC = c_{c} \div \div \div \hat{e} \hat{u} 2 \delta CC \Delta CO + \delta CC XC = 0$ Original Structure w/Final Results -0.448 + 0.054 XC = 0 XC = RC = 8.3k 9-36 Copyright © 2018 McGraw-Hill Education. If indeterminate, give the degree. Gilbert

Solutions MANUAL CHAPTER 4: TRUSSES 4-1 Copyright © 2018 McGraw-Hill Education. B C H D I 3@5m É F K J 2.5 m 2.5 m G 2@5m P12.20 12-21 Copyright © 2018 McGraw-Hill Education. Given: AE = constant, 2 A = 1000 mm, and E = 200 GPa. 30 kN B 4m D A C 2m 18 kN 8m 8m P9.27 Determine Reactions and Bar Forces., and 2 A = 15 in. P5.28. P4.43. 15 kips P6.21. vertical displacement of joint A. 3 $\approx 600 + \ddot{\circ}$ K AB = cc D.F. (a) Considering the windward pressure in the east-west direction, use the tributary area concept to compute the resultant wind force at each floor level. Vol. -84 D 10' P11.19. Compute the horizontal and vertical components of displacement of joint C. P12.46. 6 28.3 The member changes in Part (a) increased volume by 72.5% and decreased deflection 21.5%. The pad, which can apply vertical restraint in either direction but no horizontal restraint, can be treated as a roller. (BLx) bM B = 50 lt' k = L5 50 · k(15¢) = 15k BLx = (5¢)(10 ¢ h)^3 - 47 Capyright © 2018 McGraw-Hill Education. Beactions and Bar Forces. , and 2 A = 15 is given. Assume moment at end of grider HG ≈ 0.5 FEM of which 45% goes to bottom columAH. = 0.016 rad P10.24. w = 6 kips/tP5.9. Write the equation for moment between points B and C for the rigid frame in Figure P5.9 Section (1) Member BC 0 f x f 16 ¢ + exö IM 4 core is a core is 2 e × - M cè 2 e × M = -60 + 48 x - 3 x 2 5 -10 Copyright © 2018 McGraw-Hill Education. and I = 600 in. + Σ M E = 0; FCD + Σ Fy = 0; Check: + Σ E x = 0; 2 F (2) = 0 2.236 CD E y = 90 + 15 = 105 kN 90 - E y + 2(-33.54) + E x = 0 2.236 m E x = 30 kN - 10 + 10 + Bar Bed 3-30 Copyright © 2018 McGraw-Hill Education. C D P = 240 kN A E F 8m 8m 8m P8.10 E = 200 GPa. and A = 2,400 mm2 240 (T) C 240 (T) Bx Q-System (kN) D 1 kip B -120 (C) -120 (C) A E F 0.5 (T) F 30 mm 0.5 (T) E 1 36 mm 0.5 (Member FP (kN) FQ (kN) 0.5 L (m) Bx 169.706 240 240 - 339.411 - 120 - 120 0 - 169.706 0 AB BC CD DE EF FA FB BE EC (a) FP L AE 6Bx = 3.657 mm (b) 0.707 0 0 0 0.5 0.5 0 - 0.707 0 2Q6P = Σ FQ FQ E Q CMP = Σ FQ FQ Q 240 kN 0

P13.19. Compute the reactions and draw the shear and moment curves for the beam in Figure P3.1 (a) Solve Problem P9.1 for the loading shown if support C settles 0.25 in. w = 3 kN/m MA = 12 kN • m A B 6 m P5.3 + Σ M B = 0; 12 kN • m A B 6 m P5.3 + Σ M B = 0; 12 kN • m A B 6 m P5.3 + Σ M B = 0; 12 kN · m A A = -1 kN 6 1 æ x õ x 2 m Weer RA = X A A Z = -20 mm A A D + 6A A X = -20 mm A A D + 6A A X = -20 mm A A D + 6A A X = -20 mm A A A + 6A A Final Bar Forces & Reactions 9-41 Copyright © 2018 McGraw-Hill Education. Continued 16 17 l8 19 20 21 5 13.648 00 1 0 0 0 1 20.315 0 0 2 20.315 0 0 4 0 0 4 20.315 0 0 5 0 0 5 20.315 0 0 1 - 9.428 0 0 2 - 9.428 0 0 2 - 9.428 0 0 4 - 9.428 0 0 5 - 9.428 0 0 1 - 6.667 0 0 1 - 6.667 0 0 1 - 6.667 0 0 1 - 6.667 0 0 1 - 6.667 0 0 1 - 6.667 0 0 1 - 6.667 0 0 1 - 6.667 0 0 1 - 9.428 0 0 1 - 9.428 0 0 1 - 9.428 0 0 2 - 9.428 0 0 1 - 9.428 0 0 1 - 9.428 0 0 2 - 9.428 0 0 1 - 9.428 0 0 2 - 9.428 0 0 1 - 9.428 0 0 1 - 9.428 0 0 1 - 9.428 0 0 1 - 9.428 0 0 2 - 9.428 0 0 1 - 9.428 0 0 1 - 9.428 0 0 1 - 9.428 0 0 1 - 9.428 0 0 2 - 9.428 0 0 3 - 9.428 0 0 1 - 9.428 0 0 1 - 9.428 0 0 1 - 9.428 0 0 2 - 9.428 0 0 3 - 9.428 0 0 1 - 9.428 0 0 3 - 9.428 0 0 1 - 9.428 0

The trusses are loaded at their top chord panel points by the reactions from a stringer and floor beam system that supports a roadway slab. w = 4 kips/ft 15 kips A B C 8 kips 10' 6' P5.8 Σ Fy = -V + 8 - 4 x = 0 V = 8 - 4 x kip æxö Σ M = -M - $cc \div \div \div \cdot 4 x + x (8) = 0$ èc 2 ø M = 8 x - 2 x 2 kip \cdot ft 5-9 Copyright © 2018 McGraw-Hill Education. C D hinge 12' A E 15' 15' 0.75" 2" P8.26 δ CH WQ = UQ 0.5k (-2 cc) + 0.4 k (0.75cc) + 1k δ CH = 0 δ CH = 0.7 cc δ CV WQ = UQ 0.625k (-2 cc) + 0.5k (0.75cc) + 1k δ CV = 0 δ CV = 0.875cc $\Delta \theta$ AB WQ = UQ 1k 1k (2 cc) + 0.75cc) = 0 24 30 $\Delta \theta$ AB = 0.00486 Radians 1 ft k ($\Delta \theta$ AB) - 8-32 Copyright © 2018 McGraw-Hill Education. 30' P3.24. Σ Fy = 0 4-17 Copyright © 2018 McGraw-Hill Education. 10-40 Copyright © 2018 McGraw-Hill Education. = 0 = RB 22.5 - 85.5(14.25) The P.I. in span BC is 5.82' to the left of support C rather than 7.5' assumed. when the load is applied. P12.44.

6 kips P11.28. D P10.29. 15' B C 4I w = 1 kip/ft I 12' Å 30' P14.5 -wL2 = -12 kip-ft 12 -PL = -112.5 kip-ft FEM23 = -FEM32 = 8 FEM12 = -FEM32 =

P13.15. 100 kips 15' D 15' B A C 20' 20' P9.34 E Selecting Ay as the redundant, the compatibility equation is: $(\Delta AS + \Delta AO) + \delta AA X A = \Delta A O () 2 20(12) () 3(29,000) 25(12) 15(12) 4 + (2)2 + 2 3(29,000) 1(29,000) 3 2 \delta AA = 0.0538$ in. Determine all reactions.

 $\frac{6}{6} BB For 1 in. P11.25. Continued (Risa Data) 8 Member Section Forces Member Section Label 1 2 3 4 5 - 6 7 Axial Shear (k) Moment (k) 1 - 6.667 (k-ft) 4.921 0 9 1 4.764 0 0 2 4.764 0 0 3 4.764 0 0 2 4.764 0 0 3 4.764 0 0 1 - 1.903 0 0 2 - 6.667 4.921 36.91 2 - 1.903 0 0 4 - 6.667 4.921 55.364 3 - 1.903 0 0 5 - 6.667 4.921 73.819 \\ -1.903 0 0 1 - 6.667 - 14.764 73.819 5 - 1.903 0 0 1 - 8.57 0 0 10 2 - 6.667 - 14.764 36.91 2 - 8.57 0 0 1 - 2.667 - 14.764 36.91 2 - 8.57 0 0 1 - 2.6667 - 14.764 36.91 2 - 8.57 0 0 1 - 2.6667 - 14.764 36.91 2 - 8.57 0 0 1 - 1.903 0 0 2 - 6.667 4.921 36.91 2 - 8.57 0 0 1 - 1.903 0 0 4 - 6.667 - 14.764 36.91 2 - 8.57 0 0 1 - 1.903 0 0 4 - 6.667 - 14.764 36.91 2 - 8.57 0 0 1 - 1.903 0 0 4 - 6.667 - 14.764 36.91 2 - 8.57 0 0 1 - 1.903 0 0 4 - 6.667 - 14.764 36.91 2 - 8.57 0 0 1 - 1.903 0 0 4 - 6.667 - 14.764 36.91 2 - 8.57 0 0 1 - 1.903 0 0 4 - 6.667 - 14.764 36.91 2 - 8.57 0 0 1 - 1.903 0 0 4 - 6.667 - 14.764 36.91 2 - 8.57 0 0 1 - 1.903 0 0 4 - 6.667 - 14.764 18.455 3 - 8.57 0 0 1 - 1.903 0 0 4 - 6.667 - 14.764 18.455 3 - 8.57 0 0 1 - 1.903 0 0 4 - 6.667 - 14.764 18.455 3 - 8.57 0 0 1 - 1.903 0 0 4 - 6.667 - 14.764 18.455 3 - 8.57 0 0 1 - 1.903 0 0 4 - 6.667 - 14.764 18.455 3 - 8.57 0 0 1 - 1.903 0 0 4 - 6.667 - 14.764 18.455 3 - 8.57 0 0 1 - 1.903 0 0 4 - 6.667 - 14.764 0 4 - 8.57 0 0 1 - 1.903 0 0 4 - 6.667 - 15.236 0 0 1 - 1.908 0 0 1 - 6.667 - 15.236 0 0 1 - 1.908 0 0 1 - 6.667 - 15.236 0 0 1 - 1.908 0 0 1 - 6.667 - 15.236 0 0 1 - 1.908 0 0 1 - 6.352 0 0 1 2 1 18.097 0 0 2 - 6.352 0 0 1 2 1 18.097 0 0 2 - 6.352 0 0 2 - 6.352 0 0 4 18.097 0 0 3 - 6.352 0 0 5 - 6.$

The addition of diagonal cables produces a very effective stiffening system.

Continued (b) Add to the Above Analysis Support Settlement Final Bar Forces & Reactions (b) Recompute Bar Forces & Reac

However, the variation of forces between corresponding members of each truss is relatively small so the additional area required for the compression members does not significantly change the overall weight. P = 12 kips 6 kips/ft hinge C B D 12' 27 kips A x 9 kips 20 kips 32 kips 9' 9' 6' P5.48 Segment AB y 6 x; m = x 12 2 æ x ö x 2 R = y cç \div $= cc^2 + acc^2 + acc^2 + bcc^2 +$

5 @ 20' = 100' P6.1 Use General Cable Theorem: Hh = M @C H (24 ¢) = 1800 ft · k thus H = 75k and Ax = H = 75k @B 75k (h) = 1200 ft · k thus h = 16 ft@B + Check M @ B : Σ MB = 0; 75k (16 ¢) - 60 k (20 ¢) = 0 ok TAB = TEF = (60) + (75) = 96 kips k 2 k 2 TDE = TBC = (30 k) 2 + (75k) 2 = 80.78 kips TCD = H = 75k Length of Cable = (16 ¢) 2 + (20 ¢) 2 (2) + (8) 2 = (20) 2 (2) + 20 = 114.3 ft 6-2 Copyright © 2018 McGraw-Hill Education. Draw the shear and moment curves for the beam in Figure P5.35. Q · δ BV = FQ (Δ LDE) -64 k (0.75 in.) = 12 k (Δ LDE) -60 kips t = 10 kips t

0.2 L = 0.2(40 m) = 8 m FBD of Grider between P.I.'s E·F By symmetry V = V = 1 Load 2 38 + 3.6(12 \text{ m}) = 62.2 \text{ kN} 2 \text{ FBD} of grider between B and P.I. $\Sigma M B M B = 62.2(8) + 28.8(4) = 612.8 \text{ kN} \cdot \text{m}$ $\Sigma Fy VB = 28.8 + 62.2 = 91 \text{ kN}$ FBD of Column Based on GO. The 10-kip load can act at any point. 20 kN w = 6 kN/m B C 2I I 4m A 3m 12 m P14.7 $\theta 2$ is the Unknown Displ. When P on "DC" P12.24.

Take advantage of symmetry and use the end moment as the redundant. (a) Determine the members of the rectangular ring in Figure P10.12, and draw the shear and moment curves for members AB and AD. A C B 24' 16' 99.16 Compatibility Equation: $\Delta A O + 6AA X = 0.0B :$ From Table A.3 Pa 2 (L + a) 3E1 LI (A O + 6AA X = 0.6B : From Table A.3 Pa 2 (L + a) 3E1 LI (A O + 6AA X = 0.6B : From Table A.3 Pa 2 (L + a) 3E1 LI (A O + 6AA X = 0.6B : From Table A.3 Pa 2 (L + a) 3E1 LI (A O + 6AA X = 0.6A = Released Structure w/puild Loads 4096 7680 [XA] = 0 + EI EI A A O + 6AA X = 0.6B : From Table A.3 Pa 2 (L + a) 3E1 LI (A O + 6AA X = 0.6B : From Table A.3 Pa 2 (L + a) 3E1 LI (A O + 6AA X = 0.6B : From Table A.3 Pa 2 (L + a) 3E1 LI (A O + 6AA X = 0.6B : From Table A.3 Pa 2 (L + a) 3E1 LI (A O + 6AA X = 0.6B : From Table A.3 Pa 2 (L + a) 3E1 LI (A O + 6AA X = 0.6B : From Table A.3 Pa 2 (L + a) 3E1 LI (A O + 6AA X = 0.6B : From Table A.3 Pa 2 (L + a) 3E1 LI (A O + 6AA X = 0.6B : From Table A.3 Pa 2 (L + a) 3E1 LI (A O + 6AA X = 0.6B : From Table A.3 Pa 2 (L + a) 3E1 LI (A O + 6AA X = 0.6B : From Table A.3 Pa 2 (L + a) 3E1 LI (A O + 6AA X = 0.6B : From Table A.3 Pa 2 (L + a) 3E1 LI (A O + 6AA X = 0.6B : From Table A.3 Pa 2 (L + a) 3E1 LI (A O + 6AA X = 0.6B : From Table A.3 Pa 2 (L + a) 3E1 LI (A O + 6AA X A = 0.6B : From Table A.3 Pa 2 (L + a) 3E1 LI (A O + 6AA X A = 0.6B : From Table A.3 Pa 2 (L + a) 3E1 LI (A O + 6AA X A = 0.6B : From Table A.3 Pa 2 (L + a) 3E1 LI (A O + 6AA X A = 0.6B : From Table A.3 Pa Field (D = 0.2B : From Table A.3 Pa Field (D = 0.2B : From Table A.3 Pa Field (D = 0.2B : From Table A.3 Form Table A.3 Pa Field (D = 0.2B : From Table A.3 Pa Field (D = 0.2B : From Table A.3 Pa Field (D = 0.2B : From Table A.3 Pa Field (D = 0.2B : From Table A.3 Form Table A.3 Pa Field (D = 0.2B : From Table A.3 Form Table A.3 Pa Field (D = 0.2B : From Table A.3 Form Table A.3

JIHGF15'KLCAB15'D4@20' = 80'P4.14 Freebody Left of Section (1): + $\Sigma MC = 0$; 30 k (40 ¢) - FIH (15¢) = 0 FIH = 80 k compression + $\Sigma MI = 0$; 30 k (20 ¢) + FBCX (30 ¢) = 0 FBCX = 20 k tension FBCX F = BCY ¢ 20 15¢ FBCY = 15k FBC = (20 k)2 + (15k)2 = 25k tens. BC 35' DL = 70' 35' + 0.038L 0.076L 0.116L 0.158L 0.203L 0.203 0.152L 0.105L 0.064L 0.028L 0 0.021L 0.036L 0.045L 0.045L 0.047L 0.042L 0.034L 0.024L 0.024L 0.024L 0.024L 0.026L 0.162 0.162 0.162 0.1690L RTandem = 50 kips 0.1884L 0.1440L 8 kips 32 kips 25 kips 0.0776L 0.64 kips/ft 0.64 kips/ft 14 ft 4 ft 0 0.038L 0.076L 0.116L 0.158L 0.203L $0.152L \ 0.105L \ 0.064L \ 0.028L \ 0 \ 0.038L \ 0.076L \ 0.116L \ 0.158L \ 0.203L \ 0.152L \ 0.105L \ 0.064L \ 0.028L \ 0 \ 14 \ ft$ (a) Due to distributed lane load M B, w = $0.64(70)(7) \ (0.038 + 0.076 + 0.116 + 0.158 + 0.203 + 0.152 + 0.105 + 0.064 + 0.028) = 294.7 \ kip-ft$ Due to HL-93 truck point loads: Case 1, centered about resultant: MB, HL-93 = $32(70)(0.144 + 0.028) = 294.7 \ kip-ft$ Due to HL-93 truck point loads: Case 1, centered about resultant: MB, HL-93 = $32(70)(0.144 + 0.028) = 294.7 \ kip-ft$ Due to HL-93 truck point loads: Case 1, centered about resultant: MB, HL-93 = $32(70)(0.144 + 0.028) = 294.7 \ kip-ft$ Due to HL-93 truck point loads: Case 1, centered about resultant: MB, HL-93 = $32(70)(0.144 + 0.028) = 294.7 \ kip-ft$ Due to HL-93 truck point loads: Case 1, centered about resultant: MB, HL-93 = $32(70)(0.144 + 0.028) = 294.7 \ kip-ft$ Due to HL-93 truck point loads: Case 1, centered about resultant: MB, HL-93 = $32(70)(0.144 + 0.028) = 294.7 \ kip-ft$ Due to HL-93 truck point loads: Case 1, centered about resultant: MB, HL-93 = $32(70)(0.144 + 0.028) = 294.7 \ kip-ft$ Due to HL-93 truck point loads: Case 1, centered about resultant: MB, HL-93 = $32(70)(0.144 + 0.028) = 294.7 \ kip-ft$ Due to HL-93 truck point loads: Case 1, centered about resultant: MB, HL-93 = $32(70)(0.144 + 0.028) = 294.7 \ kip-ft$ Due to HL-93 truck point loads: Case 1, centered about resultant: MB, HL-93 = $32(70)(0.144 + 0.028) = 294.7 \ kip-ft$ Due to HL-93 truck point loads: Case 1, centered about resultant: MB, HL-93 = $32(70)(0.144 + 0.028) = 294.7 \ kip-ft$ Due to HL-93 truck point loads: Case 1, centered about resultant: MB, HL-93 = $32(70)(0.144 + 0.028) = 294.7 \ kip-ft$ Due to HL-93 truck point loads: Case 1, centered about resultant: MB, HL-93 = $32(70)(0.144 + 0.028) = 294.7 \ kip-ft$ Due to HL-93 truck point loads: Case 1, centered about resultant: MB, HL-93 = $32(70)(0.144 + 0.028) = 294.7 \ kip-ft$ Due to HL-93 truck point loads: Case 1, centered about resultant: MB, HL-93 = 32(70)(0.144 + 0.00.169 + 8(70)(0.0776) = 744.6 kip-ft Case 2, with middle load at peak ordinate: MB,HL-93 = 32(70)(0.203 + 0.116) + 8(70)(0.105) = 773.4 kip-ft Controlling moment at B due to AASHTO HL-93 Truck and lane load is: MB,max 294.7 773.4 1068.1 kip-ft (b) Due to Tandem point loads: Case 1, centered about resultant: MB,tandem = 25(70)(0.1901 + 0.116) + 8(70)(0.105) = 773.4 kip-ft (b) Due to Tandem point loads: Case 1, centered about resultant: MB,tandem = 25(70)(0.1901 + 0.116) + 8(70)(0.105) = 773.4 kip-ft (b) Due to Tandem point loads: Case 1, centered about resultant: MB,tandem = 25(70)(0.1901 + 0.116) + 8(70)(0.105) = 773.4 kip-ft (b) Due to Tandem point loads: Case 1, centered about resultant: MB,tandem = 25(70)(0.1901 + 0.116) + 8(70)(0.105) = 773.4 kip-ft (b) Due to Tandem point loads: Case 1, centered about resultant: MB,tandem = 25(70)(0.1901 + 0.116) + 8(70)(0.105) = 773.4 kip-ft (b) Due to Tandem point loads: Case 1, centered about resultant: MB,tandem = 25(70)(0.1901 + 0.116) + 8(70)(0.105) = 773.4 kip-ft (b) Due to Tandem point loads: Case 1, centered about resultant: MB,tandem = 25(70)(0.105) = 773.4 kip-ft (b) Due to Tandem point loads: Case 1, centered about resultant: MB,tandem = 25(70)(0.1901 + 0.116) + 8(70)(0.105) = 773.4 kip-ft (b) Due to Tandem point loads: Case 1, centered about resultant: MB,tandem = 25(70)(0.105) = 773.4 kip-ft (b) Due to Tandem point loads: Case 1, centered about resultant: MB,tandem = 25(70)(0.105) = 773.4 kip-ft (b) Due to Tandem point loads: Case 1, centered about resultant: MB,tandem = 25(70)(0.105) = 773.4 kip-ft (b) Due to Tandem point loads: Case 1, centered about resultant: MB,tandem = 25(70)(0.105) = 773.4 kip-ft (b) Due to Tandem point loads: Case 1, centered about resultant: MB,tandem = 25(70)(0.105) = 773.4 kip-ft (b) Due to Tandem point loads: Case 1, centered about resultant: MB,tandem = 25(70)(0.105) = 773.4 kip-ft (b) Due to Tandem point loads: Case 1, centered about resultant: MB,tandem = 25(70)(0.105) = 773. 0.1884) = 662.4 kip-ft Case 1, with one load at peak ordinate: MB,tandem = 25(70)(0.177 + 0.203) = 665 kip-ft Controlling moment at B due to AASHTO Tandem and lane load is: MB,max = 294.7 + 665 = 959.7 kip-ft Therefore: HL-93 Truck loading with lane load (a) is the controlling load pattern for positive moment at point B. EI is constant, I = 120 in. Determine the reactions, plot the shear and moment curves and the deflected shape. 760 380 k·ft \ M AB = -95.4 , M BA = -90.1k·ft , M CD = -42.4 k·ft , M CD Due to a construction error, the support at D has been constructed 0.6 in. Given: I = 46×10 mm, E = 200 GPa. Treat rocker at E as a roller. Compute all reactions and the 2 strut member forces. (ft) 0 35 70 105 140 Kz 1.03 1.19 1.34 1.44 1.52 (b) Variation of Wind Pressure on Windward and Leeward Sides qz = 50.18 (1.15)(K z) 1 (0.85) qz = 49.05 K z Compute Wind Pressure "p" on Windward Face p = qz GC p = 49.05 K z GC p where G 0.85 for natural period less than 1 sec. Use 3-ft segments. Continued Member Section Forces 1 - .964 13.363 - .964 13.363 9.819 4 - .964 13.363 49.908 5 - .964 13.363 89.996 1 - .543 5.118 - 23.024 87.074 2 - .543 5.118 - 23.024 87.074 2 - .543 5.118 - 23.024 87.074 2 - .543 5.118 - 23.024 87.074 2 - .543 5.118 - 23.024 87.074 2 - .543 5.118 - 23.024 87.074 2 - .543 5.118 - 23.024 87.074 2 - .543 5.118 - 23.024 87.074 2 - .543 5.118 - 23.024 87.074 2 - .543 5.118 - 23.024 87.074 2 - .543 5.118 - 23.024 87.074 2 - .543 5.118 - 7.671 - 30.594 3 - .543 5.118 7.682 4 - 543 5.118 23.034 5 - .543 5.118 23.034 5 - .543 5.118 38.387 1 - .082 19.966 - 176.218 48 739 2 - .082 19.966 - 16.49 2.243 - 1.419 4 - .082 19.966 - 16.49 2.243 - 1.419 4 - .082 19.966 - 16.49 2.243 - 1.419 4 - .082 19.966 - 16.49 2.243 - 1.419 4 - .082 19.966 - 16.49 2.243 - 1.419 4 - .082 19.966 - 16.49 2.243 - 1.419 4 - .082 19.966 - 16.49 2.243 - 1.419 4 - .082 19.966 - 16.49 2.243 - 1.419 4 - .082 19.966 - 16.49 2.243 - 1.419 4 - .082 - 19.966 - 16.49 2.243 - 1.419 4 - .082 - 19.966 - 16.49 2.243 - 1.419 4 - .082 - 19.966 - 16.49 2.243 - 1.419 4 - .082 - 19.966 - 16.49 2.243 - 1.419 - .082 - 19.966 - 16.49 - 2.243 - 1.419 - .082 - 19.966 - 16.49 - 2.243 - 1.419 - .082 - 19.966 - 16.49 - 2.243 - 1.419 - .082 - 19.966 - 16.49 - 2.243 - 1.419 - .082 - 19.966 - 16.49 - 2.243 - 1.419 - .082 - 19.966 - 16.49 - 2.243 - 1.419 - .082 - 19.966 - 16.49 - 2.243 - 1.419 - .082 - 19.966 - 16.49 - 2.243 - 1.419 - .082 - 19.966 - 16.49 - 2.243 - 1.419 - .082 - 19.966 - 16.49 - 2.243 - 1.419 - .082 - 19.966 - 16.49 - 2.243 - 1.419 - .082 - 19.966 - 16.49 - 2.243 - 1.419 - .082 - 19.966 - 16.49 - 2.243 - 1.419 - .082 - 19.966 - 16.49 - 2.243 - 1.419 - .082 - 19.966 - 16.49 - 2.243 - 1.419 - .082 - 19.966 - 16.49 - 2.243 - 2.361 - 2.243 - 2.361 - 2.243 - 2.361 - 2.243 - 2.361 - 2.243 - 2.361 - 2.243 - 2.361 - 2.243 - 2.361 - 2.243 - 2.361 - 2.243 - 2.361 - 2.243 - 2.361 - 2.243 - 2.361 - 2.243 - 2.361 - 2.243 - 2.361 - 2.243 - 2.361 - 2.361 - 2.361 - 2.361 - 2.361 - 2.361 - 2.361 - 2.361 - 2.361 - 2.361 - 2.361 - 2.361 - 2.361 - 2.361 - 2.361 - 2.361 - 2.361 - 2.361 - 2.361 -12.303 -111 952 3 .771 13.371 9.683 2 24.92 12.303 -62.739 4 .771 13.371 49.796 3 24.92 12.303 -13.527 5 .771 13.371 49.796 3 24.92 12.303 35.685 1 .441 5.231 -23.713 5 24.92 12.303 84.898 2 .441 5.231 -8.019 1 9.967 6.655 -31.304 3 .441 5.231 7.675 2 9.967 6.655 -11.339 4 .441 5.231 23.369 3 9.967 6 655 8.627 5 .441 5.231 39.054 4 41 5.231 39.054 4 $9.967\ 6.655\ 28.593\ 1\ 19.01\ -15.113\ 117.668\ 5\ 9.967\ 6.655\ 48.559\ 2\ 19.01\ -15.113\ 60.994\ 1\ 2.464\ 2.408\ -9.165\ 3\ 19.01\ -15.113\ -109.029\ 4\ 2.464\ 2.408\ 12.505\ 1\ 12.243\ -14.1\ 105.817\ 5\ 2.464\ 2.408\ 19.729\ 2\ 12.243\ -14.1\ 52.941\ 1\ .049\ 20.13\ -177.591\ 3$ $12.243 - 14.1 - 52.811 \ 3.049 \ 20.13 - 97.071 \ 4 \ 12.243 - 14.1 - 52.811 \ 3 \ .049 \ 20.13 - 16.551 \ 5 \ 12.243 - 14.1 - 105.686 \ 4 \ .049 \ 20.13 \ 63.969 \ 1 \ 5.648 - 14.953 \ 52.02 \ 3 \ 5.648 - 14.953 \ -14.953 \$ 12.601 36.67 5 -24.888 12.601 1 -9.775 6.611 2 -9.775 6.611 -10.761 3 -9.775 6.611 9.072 4 -9.775 6.611 9.072 4 -9.775 6.611 1 -2.361 2.243 2 -2.361 3 M2 M3 M4 M5 M6 M7 Axial (k) Shear (k) M8 Moment (k-ft) M9 M10 M11 M12 M13 M14 M15 13-32 Copyright © 2018 McGraw-Hill Education. $Ax = 0.667k + \Sigma M = 0; -M - Ax(18c) + Ay(1) @ x = 1c M = -0.479 @ x = 1c M = -0.479$ and the respective locations of the hoist. 2 4 tubes with A = 3.59 in. C P10.25. 12 kN P3.13. Continued 45 kips = Fx F 9 ft C 0.9 kips 45 k kips 25 kips C 6 ft C -84 kip-ft 20 kips 2 kips 20 kips B 10 ft E 0.3 kips 46.8 kips -1.8 kip-ft 0.3 kips 75.5 kip-ft 0.9 kips B 23 kips 2 kips 75.5 kip-ft MBE = -0.9 kips 75.5 kip-ft MBE = -1.8 kip-ft MBE = -1.8 kip-ft MBE = -1.8 kip-ft MBE = -0.9 kips Ay = -0.9 kips Ay = -0.9 kips 75.5 kip-ft MBE = -1.8 kip-ft MBE = -0.9 kips -1.8 kip-ft -1.8 kip-ft -1.8 kip-ft -1.8 kip-ft -1.8 kip 24.2 kips MA = -65.1 kip-ft MEB = -0.9 kip-ft MCD = -84 kip-ft E x = 0.3 kips \neg E y = 46.8 kips ME = -0.9 kip-ft F x = 0.9 kips \neg F y = 45 kips MA = -65.1 kip-ft MCD = -84 kip-ft ACD - 23.8 -25 3.33 ft 9.31 ft -61.7 -65.1 0.3 -59.9 75.5 0.3 E Moment (kip-ft) 3 ft E -0.9 11-23 Copyright © 2018 McGraw-Hill Education. P11.17.

C P4.32. The truss in Figure P4.55 is constructed of square steel tubes welded to form a structure with rigid joints. P3.37. E P4.49. when the load acts and support A is constructed 0.24 in. Note that the vertical reactions at supports 1 and 4 are not affected by the change of load path. b = 14 b = 6 (h) r = 3 n = 8 r = 4 n = 5 (b + r = 17) > (2n = 16) (b + r = 10) > (2n = 10) 1 deg. Case 2: If the bases of the columns at point A and F are attached to the foundations by fixed supports, how much is the lateral deflection at joint D reduced? 30 kN 30 kN 2m 3m 10 kN A B D C 4m G F E 6 @ 3 m = 18 m P12.38 é 9 ù 1 3 Cmax = ê (30) - (30)ú êi 16 úù 2 16 1 = éi-16.875 · 5.625ùû 2 = 11.25 kN per Truss Max. 20 kN P9.20. Compute the vertical deflection of joint C in Figure P8.36. Member CD Member AB Deflected Shaps 5-27 Copyright © 2018 McGraw-Hill Education. 13 wind pressures in h/ft2 Vm = 0.055(60)/4 11 SDS W 0.9(3870) = = 8/1 kJ = 435 kips 2-19 Copyright © 2018 McGraw-Hill Education. Continued (a) Moment Diagram (kips) Axial Force (kips) Relative Displacement between 1st and 2nd floors = 5.81 - 0 = 5.81 is 0.75(8/1) SDS W 0.9(3870) = = 8/1 kJ = 435 kips 2-19 Copyright © 2018 McGraw-Hill Education. w = 8 kips/ft P13.7. The beam in Figure P13.7 is indeterminate to the second degree. FEM BA = FEM EF = FEM AB = FEMFE = Arbitrarily LET 2 EI 3EI (0 + 0 - 3\Psi) = \Psi 4 2 3EI \Psi = 90 kN \cdot m 2 D.F. Analyze the frame in Figure P10.27. Tension = (81)(0.25)0.8 2 ft = 13.1 kips 12-43 Copyright © 2018 McGraw-Hill Education. w = 8 kips/ft P10.20.3. 25' A C B 60 kips D 60 kips 20' 20' 5' P3.48 Releasing the rod at E, and neglecting the axial stiffness of the beam, the compatibility equation becomes: 6Cable E ($\Delta CS + \Delta CO$) + $\delta CC = \Delta C 5 \delta V$, Cable $\delta CC = \delta V$, Cable $\delta C = \delta V$, Cable δC

For the semi-circular, three-hinged arch ABC, shown in Figure P12.23, construct the influence lines for reactions at A and C, and shear, axial load and moment at F.

Continued Compute Shears & Force R1 R1 = $3 + 1.58 - 1.67 = 2.91k + \Sigma M B = 0 = 6'8 + 6'16 - VCB 24 + 20.86 - 19 VCB = 6.04 k VBC = 5.96 k + \Sigma M A = 0 = 19 - VBA 12 VBA = 1.58 + \Sigma M D = 0 = -20.06 - 10.03 + VCD 18 VCD = 1.67 kips Introduce A 1" Displ. and 4 I = 400 in. 60 kN 20 kips P4.17. Verify that both procedures give the same value of moment at point C. 2-3 Copyright © 2018 McGraw-Hill Education. Continued Axial Force Diagram (kips) Moment Diagram (kip-ft) Observation: The addition of the 48-kip concentrated load at joint 18 will produce large values of bending moment in the arch and large deflections (Over 6 feet at some joints) in both the vertical and horizontal directions.$

Compute all reactions and draw the shear and moment diagrams.

(a) Compute the vertical displacement of the hinge at C in Figure P8.34. B A C 3m F D E 4m 4m P11.31 Analyee Half of Structure: Structure symmetrical about A horiz. Cx P13.5. Estimate the moment in the beam in Figure P13.5 at support C and the maximum positive moment in span CD by guessing the location of one of the points of inflection in span CD. The three-hinged parabolic arch in Figure P6.24 supports 60-kip loads at the quarter points. F P3.14. D 6m P4.22.

C P12.26. B 6' C 7' 8' P = 12 kips P8.30 I = 360 in4; E = 30,000 ksi Compute Segment Orgin Range MP MQ δ CV (MQ = MP) AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ CD C 0 $\leq x 2 \leq 159.6x$ AB A 0 $\leq x 1 \leq 1584 - 12x$ A $\leq 1584 - 12x$

Compare the results with those produced by the cantilever method. The building is located near the Georgia coast where the wind velocity contour map in the ASCE Standard specifies a design wind speed of 140 mph. At C where the bottom flange of the girder is bolted to a cap plate welded to the end of the column, the joint can be assumed to act as a hinge (it has no significant capacity to transmit moment). A B C moment P5.14 5-16 Copyright © 2018 McGraw-Hill Education. w2 = 6 kips/ft P5.26. 2 For member BCD, A = 6000 mm and 6 4 I = 600 × 10 mm . (1) and (2) 1000 2000, $\theta = \theta B = 57 \text{ EI C } 19 \text{ EI 2 EI 200 } (2 \theta B) + 10 3 2 \text{ EI 200 MCB} = (2 \theta C + \theta B) + 10 3 2 \text{ EI 2 EI } (2 \theta C)$, M DC = (θ) MCD = 15 15 C Equilibrium at Joint C M BA = MCB + MCD = 0 (2) \ M AB = -63.1k ft, M BC = -80.7 k ft, M BC = -80.7 k ft, M BC = -80.7 k ft, M BE = 73.7 k ft, M BC = -80.7 k ft, M BE = 73.7 k ft, M BE = 73.7 k ft, M BE = 73.7 k ft, M BE = -73.7 k ft, M BE = -80.7 k ft, M

The pin joint at D acts as a hinge. plywood weighs 3 lb/ft . (NG) Deflected Shape 7-56 Copyright © 2018 McGraw-Hill Education. Compute the reactions at supports A, E, and F. E = 24 GPa. Base your analysis on the properties of 0.5IG.

P11.30. Compute the seismic base shear V. Use I = 1 qhGCp 20 m 9m 40 m (not to scale) P2.12 Total Windforce, FW, Windward Wall qs = 0.613 V (Eq. 2.4b) 2 = 0.613(40) = 980.8 N/m 2 FW = 481.8[4.6 \times 20] + 532.9[1.5 \times 20] + 532.9[1.5 \times 20] + 555.6[1.4 \times 20] 2 qz = qs IK z K zt K d FW = 91,180 N qz = 980.8(1)(K z)(1)(0.85) = 833.7 K z For Leeward Wall 0 - 4.6 m: qz = 833.7(0.85) = 708.6 N/m 2 4.6 - 6.1m: qz = 833.7(0.90) = 750.3 N/m 2 6.1 = 7.6 m: qz = 833.7(0.94) = 783.7 N/m 7.6 = 9 m -0.2 = -138.9 N/m 2 2 Total Windforce, FL, on Leeward Wall FL = (20 $^{\prime}$ 9)(-138.9) = -25,003 N* Total Force = FW + FL = 91,180 N + 25,003 p = 0.68 qz = 116,183.3 N 0 - 4.6 m p = 481.8 N/m 2 4.6 - 6.1 m p = 510.2 N/m 2 *Both FL and FN Act in Same Direction. Assume member AB has 2EI while BC has EI. Which case produces a larger deflection? Draw the shear and moment curves for each member of the frame in Figure P5.27. (one-half the depth of the bridge and its deck as well as produce discomfort for people driving over the excessively flexible bridge. Consider the strain energy produced by both axial and flexural deformations. P15.6. Determine all joint displacements, reactions, and bar forces for the truss in 2 Figure P15.6. For all bars, A = 1500 mm and E = 200 GPa. 4 1 3m 3 2 3m 1 2 80 kN 3 4m 5m P15.6 Analyze by Stiffness Methoo: All Bars A = 1500 mm 2 E = 200 GPa L=5m AE = K is Constant for all Bars L B hinge 75' A C 150' 150' P3.19 + $\Sigma M A = 0$; 900 \uparrow 75 - Rcy 300 = 0 Rcy = 225 kips + $\Sigma Fy = 0$; - 900 + RAy + 225 = 0 RAy = 675 kips Freebody Diagram Right of Hinge @ B: + $\Sigma Mb = 0$; 0 = Rcx \uparrow .75 - 225(150) Rcx = 450 kips \neg + $\Sigma Fx = 0$; RAx - 450 = 0 RAk = 450 kips 3-20 Copyright © 2018 McGraw-Hill Education. (a) Compute the slope at A and the horizontal displacement of joint B in Figure P8.32. A P12.13. Compute the horizontal and vertical components of the deflection at C in Figure 3 2 P8.29. 5 kips P8.28. Use E = 29,000 ksi. Interpolate in Table P2.10 C p = 0.2 + (33.69 - 30) - 0.1 (35 - 30) C p = 0.2738 (Roof only) From Table 2.4 (see p48 of text) K z = 0.57, 0 - 15¢ = 0.62, 15¢ - 20¢ Mean Roof Height, h 24 ft = 0.66, 20¢ - 25¢ = 0.70, 25¢ - 30¢ æ 16 ¢ \ddot{o} θ = tan-1 çç ÷ = 33.69 (for Table 2.10) çè 24 ¢ ÷ θ = 0.76, 30¢ - 32¢ 2-14Copyright © 2018 McGraw-Hill Education. Indicate tension or compression. δ of Exterior Columns/1kip PL 1k (360)12 = 0.004966 ¢¢ AE 30 29,000 Displacement of Points D & F of Truss. A B 8m C 4m D 4m F E 4m RA, RB, RF, MF, VB (to left of support B), VB (to right of Construction) of Exterior Columns/1kip PL 1k (360)12 = 0.004966 ¢¢ AE 30 29,000 Displacement of Points D & F of Truss. A B 8m C 4m D 4m F E 4m RA, RB, RF, MF, VB (to left of support B), VB (to right of Construction) of Exterior Columns/1kip PL 1k (360)12 = 0.004966 ¢¢ AE 30 29,000 Displacement of Points D & F of Truss. A B 8m C 4m D 4m F E 4m RA, RB, RF, MF, VB (to left of support B), VB (to right of Construction) of Construction D & F of Truss. A B 8m C 4m D 4m F E 4m RA, RB, RF, MF, VB (to left of support B), VB (to right of Construction) of Construction D & F of Truss. A B 8m C 4m D 4m F E 4m RA, RB, RF, MF, VB (to left of support B), VB (to right of Construction) of Construction D & F of Truss. A B 8m C 4m D 4m F E 4m RA, RB, RF, MF, VB (to left of support B), VB (to right of Construction) of Construction D & F of Truss. A B 8m C 4m D 4m F E 4m RA, RB, RF, MF, VB (to left of support B), VB (to right of Construction) of Construction D & F of Truss. A B 8m C 4m D 4m F E 4m RA, RB, RF, MF, VB (to left of support B), VB (to right of Construction) of Construction D & F of Truss. A B 8m C 4m D 4m F E 4m RA, RB, RF, MF, VB (to left of support B), VB (to right of Construction) of Construction D & F of Truss. A B 8m C 4m D 4m F E 4m RA, RB, RF, MF, VB (to left of Support B), VB (to right of Construction) of Construction D & F of Truss. A B 8m C 4m D 4m F E 4m RA, RB, RF, MF, VB (to left of Support B), VB (to right of Construction) of Construction D & F of Truss. A B 8m C 4m D 4m F E 4m RA, RB, RF, MF, VB (to left of Support B), VB (to right of Construction) of Construction D & F of Truss. A B 8m C 4m D 4m F E 4m RA, RB, RF, MF, VB (to right of Construction) of Construction D & F of Truss. A B 8m C 4m D 4m F E 4m RA, RB, RF, MF, VB (to right of Constructio support B), VE P12.14 12-15 Copyright © 2018 McGraw-Hill Education. Compute the vertical deflection at C in Figure P2.4. Consider the floor plan shown in Figure column C 2 3 6 @ 6.67' = 40' C 4 A B 2 G 1 B 4 B 3 2 @ 10' = 20' C 2 B G 4 G 3 G 2 B 1 C 5 @ 8' = 40' C 3 40' C 1 20' P 2.4 (a) Method 2: AT 8 8 40 A 320 ft 2 2 1 320 4 4(4) A 288 ft 2 6.67 ft 2 10 ft 2 10 ft 2 10 ft 7 6.67 ft 2 10 ft 20 10(10) 2 AT 166.7 ft Left Side 2 6.66 ft 4 ft 2 G2 G2 2 1 4(4) 2 Method 2: AT 1080 2 B4 2 AT, C2 40 20 ; A 20 ft 2 2 40 20 ; A 22 2 2 T T AT, C1 900 ft 6.67 ft G1 36 ft (d) Method 1: AT AT 1080 ft 6.67 ft G1 1 3.33(3.33) 2 1 5(5) 2 2 AT 180.6 ft 5 ft Right Side Method 2: AT 180.6 ft 5 ft Right Side Method 2: AT 1080 ft 6.67 ft G1 1 3.33(3.33) 2 1 5(5) 2 2 AT 180.6 ft 5 ft Right Side Method 2: AT 1080 ft 6.67 ft G1 36 ft (d) Method 1: AT AT 1080 ft 6.67 ft G1 1 3.33(3.33) 2 1 5(5) 2 2 AT 180.6 ft 5 ft Right Side Method 2: AT 180.6 ft 5 ft Right Side Method 2: AT 180.6 ft 5 ft G1 36 ft 4 ft 40 20 36 2 2 AT 1096 ft 5 ft G1 36 ft (d) Method 1: AT AT 1080 ft 6.67 ft G1 1 3.33(3.33) 2 1 5(5) 2 2 AT 180.6 ft 5 ft Right Side Method 2: AT 180.6 ft 5 ft Right Side Method 2: AT 180.6 ft 5 ft Right Side Method 2: AT 180.6 ft 5 ft Right Side Method 2: AT 180.6 ft 5 ft G1 36 ft 4 ft 40 20 36 2 2 AT 180.6 ft 5 ft G1 36 ft 4 ft 40 20 36 2 2 AT 180.6 ft 5 ft Right Side Method 2: AT 180.6 ft 5 ft Right Side Method 2: AT 180.6 ft 5 ft Right Side Method 2: AT 180.6 ft 5 ft Right Side Method 2: AT 180.6 ft 5 ft Right Side Method 2: AT 180.6 ft 5 ft Right Side Method 2: AT 180.6 ft 5 ft Right Side Method 3: AT AT 166.7 2 (f) AT 6.66 ft T (c) Method 1: AT (e) AT 4 ft 2 2 2-5 Copyright © 2018 McGraw-Hill Education. P2.3. A wide flange steel beam shown in Figure P2.3 supports a permanent concrete masonry wall, floor slab, architectural finishes, mechanical and electrical systems. girder deflection = 1.17 in.) (b) Shear (kips) Moment (kip-ft) Axial Force (kips) Lateral Deflection of Tower (= 0.63 in., and I = 288 in. Note that the wind pressure can act toward or away from the windward roof surface. Joints B and C are rigid. 10.5 kips P = 36 kips 3' P5.34. 24 kN P13.6. Estimate the moment at support C in Figure P13.6. Based on your estimate, compute the reactions at B and C. C 12 kN 12 kN 2m B D 2m A G F 3@4m P4.28 Joint "A" + $\Sigma Fy = 0.21$ - FABY = 0 FABY = 21 kN FABX F = ABX = 21 3 2 3 FABX = (21) = 31.5 kN 2 Joint G + $\Sigma Fx = 0$ Cut A Section 1-1 = 22.5 - 31.5 + 4.5 + FBGX + FBGX = 4.5 kN compr. 5' G P5.40. For the three-hinged arch shown in Figure P6.22, compute the reactions at A and C. P12.23. Area of 2 2 bars 1 and 2 = 2.4 in. The spring stiffness is 10 kips/in. Estimated DEFL: $\Delta = 2^{-1.15} = 2.3$ in 13-18 Copyright © 2018 McGraw-Hill Education. G 1.8 kips 7.2 kips 8' C D 7.4 kips 24 (c) + 7.4 (16 (c) - E (y) (32 (c)) = 0 E (y) = 10.45 (x) + 7.2 (k) - 7.4 (k) + 10.45 (k) + 4y = 0 (Ay = 11.35 (k) + 2.4 (k)12(15'106) = 104.2 k (10'12)3 K11 = 3750 + 104.2 = 3854.2 K 21 = 0 EA(1) (0.45'106)(1) = 3750 k L 10'12 K 31 = -6250 k 2 = 3750 k 10'12 k 31 = -6250 k 2 = 3750 k 10'12 k 31 = -6250 k 2 = 3750 k 10'12 k 10'12 k 31 = -6250 k 2 = 3750 k 10'12 k 10'12 k 31 = -6250 k 2 = 3750 k 10'12 k 101,000,000 é3854.2 0 -6250ùú ê [K] = êê 0 3854.1 6250ùú ê ú 0 1,000,000ú êë-6250 û 16-12 Copyright © 2018 McGraw-Hill Education. The Bayonne Bridge, one of the text). Determine the reactions at supports A and E in Figure P9.46. and I = 4 1430 in. Determine all reactions and draw the moment and shear curves. Compute all joint displacements and reactions using Equations 2 15.34 and 15.35. Q1 = 48 kips Q5 = 108 kips -58.3 (C) Q2 = 7.72 kips Q6 = 66.26 kips 15-10 Copyright © 2018 McGraw-Hill Education. D P9.38. P2.21. K AB = K FE I I = L 4 I 4 = 1 3I 3 4 K AF = K AF 3 I 3 I = = 2 3 2L 2I 4 = 2 3I 3 4 \Sigma K S ¢ = 3 I 4 11-45 Copyright © 2018 McGraw-Hill Education. w = 1.6 kips/ft P9.47. FEMBA P10.2. Using Equations 10.12 and 10.13, compute the fixed-end moments for the fixed-end moments fo $\delta BB X B = \Delta B 0$ () 2 20(12) 15(12) + (1) () 3(29,000) 1(29,000) 25(12) 2 + 2 3(29,000) 3 2 2 B 3 in 2 Δ BS = -0.25 in 2 P15.5. AE is constant for all bars. 1 P = 8 kips w = 3 kips/ft A C B 5' 4' 16' P5.12 1 + ΣM B = 0; 8' 4 - 48'8 + 14 RC = 0 8 kips RC = 22 kips $\Sigma Fy = 0; -8 - 48 + RB + 22 = 0 + \Sigma M z = 0; + \Sigma M z = 0; + \Sigma M z = 0; 16' Cy = 22 kips 8 kips 3(x - 4) 8 kips V(x) M(x) - 8 - V = 0 M(x) A 4' V = -8k M + 8x = 0 x z (c) Locate Point V = 0 M = -8 x N \cdot k \Sigma Fy = 0; 5' By = 34 kips (a) A-B Origin at "A;" 0 f x f 4 + C 4' RB = 34 kips (b) SFy = 0; 3 kips/ft A - 8 + 34 - 3(x - 4) - V = 0 V = -3x + 38 (EQ. 10 k P16.6. Using the second se$ stiffness method, analyze the frame in Figure P16.6 and draw the shear and moment curves for the members. P6.24. The average weights of the 2 2 floor and roof are 90 lb/ft and 70 lb/ft, respectively. The wall consists of 8-in. A P4.30. Determine the horizontal and vertical deflection of joint C of the truss in Figure P8.11. w = 2 kips/ft P10.12. To reduce the differential vertical displacements between the interior and exterior of the building, a bonnet truss has been added at the top of the building, 0.016 rad B 12' I = 300 in.4 I = 75 in.4 A 0.48" 24' P11.23 6 EI Δ 2 EI α + L L2 6(29000)(300)(0.48) 2(29000)(300)(0.016) = + 24 (12 (24 (12)) = 1268.8 kip-in = +105.7 kip · ft 6 EI Δ 4 EI α = 2 + L L 6(29000)(300)(0.48) 4(29000)(300)(0.016) = + 24 (12) (24 (12)) = 2235.4 kip-in = +186.3 kip · ft 1 75 1 = = . DFAB = 12 12 3 I 300 2 = . DFAB = 12 12 3 I 300 2 = . DFAB = 12 12 3 I 300 2 = . DFAB = 12 12 3 I 300 2 = . DFAB = 12 12 3 I 300 2 = . DFAB Assume the truck can travel in either direction. w = 2 kips/ft A B hinge 6 kips C 6' 3' D 3' 2' P5.30 FBD of segment BCD Σ M B = 3(6 · 6) - 3C y + 8(6) = 0 12 kips/ft A B hinge 6 kips C 6' 3' D 3' 2' P5.30 FBD of segment BCD Σ M B = 3(6 · 6) - 3C y + 8(6) = 0 12 kips/ft A B hinge 6 kips C 6' 3' D 3' 2' P5.30 FBD of segment ABC Σ M A = MA + $3(12 \cdot 6) - 6(10) = 0 - 10$ MA = -156 kip · ft $\Sigma Fy = Ay - 12 \cdot 6 + 10 = 0 - 28$ 4.2 Moment (kip-ft) Ay = 62 kips 0.84' -57 - 152 Deflected Shape 5-32 Copyright © 2018 McGraw-Hill Education. (a) The three-hinged arch shown in Figure P12.22 has a parabolic profile. (a) Construct the influence lines for the reactions RA and MA at support A. 2 1 1 43235m 3m P15.1 Determine Δx and $\Delta y @$ Joint 1 Bar 1 cos fx = 1 Unit horizontal displacement of joint 1 sin fx = 0 Bar 2 cos fx = sin fx = Unit vertical displacement of joint 1 3 5 4 5 Equilibrium Equations $\Sigma Fx = 0$; K11D x + K12 D y = -48 $\Sigma Fy = 0$; K 21D x + K 22 D y = -64 (1) (2) Evaluate Stiffness Cofficients; Using Figure 15.2a and Figure 15.2a and Figure 15.2b 15.2b in Text AE AE $AE AE aE 3\ddot{o} \div c \div cos2 fx = (1) + L L L ce 5 \# 4 \ddot{o} AE AE aE 3 \ddot{o} \# c \div c \div cos fx sin fx = (0) + K 21 = \Sigma \div e \# 5 \div e \# 5 + e \# 6 + c \div c \div cos fx sin fx = (0) + K 21 = \Sigma + cos fx sin fx = (0) + K 21 = \Sigma + e \# 6 + c \div c \div c + cos fx sin fx cos fx = (0) + K 21 = \Sigma + cos fx sin fx cos fx = (0) + K 21 = \Sigma + e \# 6 + c \div c \div c + cos fx sin fx cos fx = (0) + K 21 = \Sigma + cos fx sin fx cos fx = (0) + K 21 = \Sigma + e \# 6 + c \div c + cos fx sin fx cos fx = (0) + K 12 = \Sigma L L L ce 5 + \# 6 + cos fx sin fx cos fx = (0) + L L L ce 5 + \# 6 + cos fx sin fx cos fx = (0) + K 21 = \Sigma + e \# 6 + cos fx sin fx cos fx = (0) + K 21 = \Sigma + e \# 6 + cos fx sin fx cos fx = (0) + K 21 = \Sigma + cos fx sin fx cos fx = (0) + K 21 = \Sigma + e \# 6 + cos fx sin fx cos fx = (0) + K 21 = \Sigma + cos fx sin fx cos fx = (0) + K 21 = \Sigma + cos fx sin fx cos fx sin fx cos fx = (0) + K 21 = \Sigma + cos fx sin fx cos fx = (0) + K 21 = \Sigma + cos fx sin fx cos fx cos$ = 25 L 16 AE K 22 = 25 L K11 = 15-2 Copyright © 2018 McGraw-Hill Education. The cable reaction may be assumed to be applied to the underside of the roadway. I H G 20' F A B 30 kips C D E 60 kips 5 @ 15' BJ, CJ, CI, HG, and DI P4.38 Entire Structure + $\Sigma M A = 0$; 30 (1) + 60 (3) - Fy (5) = 0 Fy = 42 k Ay - 30 - 60 + 42 = 0 Ay = 48k $\Sigma Fy = 0$; + $\Sigma Fx = 0$; + Ax = 0 Isolate Joint B $\Sigma Fy = 0$; + FBJ - 30 = 0 FBJ = 30 k T æ4ö 48 - 30 - FCJ cc $\div \div ce^{5} \div gFCJ = 22.5k$ T 40 - 30 - FCJ = 0; $+ Isolate Joint H \Sigma Fy = 0$; $+ Cut Section (2) \Sigma Fy = 0$; $+ Cut Section (3) + \Sigma M D = 0$; FHG = 0; + FBJ - 30 = 0 FHG = 63k C 4-39 Copyright @ 2018 McGraw-Hill Education. 6' A B 4' C 4' P12.9 12-10 Copyright © 2018 McGraw-Hill Education. C A B 7' 7' P9.4 Selecting By as the redundant, the compatibility equation is: 0 = D BO + δBB X B Using known relations for the resulting released structure: wL4 300.125 L3 14.292 D BO = =, δBB = =, ΔB = 0.384 EI EI 192 EI EI Compatibility equation: 3 kips/ft MA' MC' A C $\Delta BO 0 = \Delta BO + \delta BB X B = 0.0 = \delta BBXB = By - 300.125 14.292 XB + EI EI X B = By = 21 kips$ From symmetry: Ay = Cy $\Sigma Fy = 0 = -3(14) + 21 + 2 Ay Ay = Cy \Sigma Fy = 0 = -3(14) + 2 Ay Ay = Cy \Sigma Fy = 0 = -3(14) + 2 Ay Ay = Cy \Sigma Fy = 0 = -3(14) + 2 Ay Ay = Cy \Sigma Fy = 0 = -3(14) + 2 Ay Ay = Cy \Sigma Fy = 0 = -3(14) + 2 Ay Ay = Cy \Sigma Fy = 0 = -3(14) + 2 Ay Ay = Cy \Sigma Fy = 0 = -3(14) + 2 Ay Ay = Cy \Sigma Fy = 0 = -3(14) + 2 Ay Ay = Cy \Sigma Fy = 0 = -3(14) + 2 Ay Ay = Cy \Sigma Fy = 0 = -3(14) + 2 Ay Ay = Cy \Sigma Fy = 0 = -3(14) + 2 Ay Ay = Cy \Sigma Fy = 0 = -3(14) + 2 Ay Ay = Cy \Sigma Fy = 0 = -3(14) + 2 Ay Ay = Cy \Sigma Fy = 0 = -3(14) + 2 Ay Ay = Cy \Sigma Fy = 0 = -3(14) + 2 Ay Ay = Cy \Sigma Fy = 0 = -3(14) + 2 Ay Ay = Cy \Sigma Fy = 0 = -3(14) + 2 Ay Ay = Cy \Sigma Fy = 0 = -3(14) + 2 Ay Ay = Cy \Sigma Fy = 0 = -3(14) + 2 Ay Ay = Cy$ -12.25 kip ft C By =21 kips MC = 12.25 kip ft Ay =10.5 kips 9-5 Copyright © 2018 McGraw-Hill Education. Draw the influence lines for forces in bars FE and CE. REL = 0 Use Shear at Hinge as Redundant \triangle BO + 26BVB = 0 WL4 EL3 [V] = 0 + 8EI 3E B 3WL VB = 16 - Load Applied to Released Structure [VB] Unit Value of Redundant Applied to Released Structure 9-15 Copyright © 2018 McGraw-Hill Education. (i) Evaluate the moment at section 1. grade 50 kips B C 12' A E D 6' 12' P8.5 FP L AE k -1 (-150)(12 ¢ 12%) $\Sigma Q \cdot \delta P = \Sigma FQ 1k \cdot \delta By = 5$ in 2 (29000 k/in 2) = 0.149 ¢ 1k \cdot \delta Bx = -2 k (-150 k)(12 ¢ 12%) 5 in 2 (29,000 k/in 2) = 0.298¢ 8-6 Copyright © 2018 McGraw-Hill Education. B C 15 A 30' P9.40 \triangle AO = θ B 15¢ (See Fig 9.40) wL -37.5 2(30)3 = E 24 EI 24 E100 3 \triangle AO = Select RAX as Redundant δ AA = \diamond 15 0 2 X DX EIC + \diamond 30 0 é X3 ù é + \diamond 0 EIC EI G æ X \ddot{o} + $\dot{c}c$ + DX $\dot{e}20$ EI G 2 = \diamond 15 0 2 X DX EIC + \diamond 30 0 é X3 ù é + \dot{e} ú = \hat{e} \ddot{e} 3E 600 úû 0 \ddot{e} 230 M D -M a2 Dx a X + \diamond 0 EIC EI G æ X \ddot{o} + $\dot{c}c$ + DX $\dot{e}20$ EI G 2 = \diamond 15 0 2 X DX EIC + \diamond 30 0 é X3 ù é + \hat{e} ú = \hat{e} \ddot{e} 3E 600 úû 0 \ddot{e} 2375 27,000 = + 1800 E 10,800 E 15 $\delta AA = 30$ Released Structure 1.875 2.5 4.375 + = E E Compatability Equation. and E is 30,000 ksi. A P6.13. Also the diagonal cables reduce the lateral displacement of the arch, producing a more uniform distribution of stress on the arch. building when positive pressure acts on the windward roof. P11.3. Analyze each structure by moment distribution. Absolute Max Shear RA = 40.2 kips = Vmax 12-48 Copyright © 2018 McGraw-Hill Education of distance x along the longitudinal axis of the beam in Figure P5.5 for (a) origin of x at A and (b) origin of x at B. X A = RA = 0.787k Remaining Reactions by Statics: + $\Sigma M B = 0$; + 0.787(24) - RC (16) = 0 $\Sigma Fy = 0 RC = 1.18k$ RB = 1.967k Released Structure w/Unit Value of RA (b.) Compatibility Equations: $\Delta AO1 + \Delta AO2 + \delta AA X A = 0$ (Load) (b.) (Settlemt) θB : From Figure 9.3 WL3 (1)(16)3144 = 2.825(10-3) Rad. Use the following data: basic wind speed = 100 mi/h, wind exposure category = B, Kd = 0.85, Kzt = 1.0, G = 0.85, and Cp = 0.8 for windward wall and 0.2 for leeward wall., I = 240 in. If the maximum tensile force in the cable cannot exceed 600 kips, determine the sag hB at midspan. Draw the shear and moment curves for the beam.

Factor AD = 0.65 FEM = 11-15 Copyright © 2018 McGraw-Hill Education. D C 12' 24 kips B E 12' A 8' 8' P5.43 + Σ M D = 0; Ay 16 - 24 '8 - 24 '6 = 0 Ay = 21 kips + Σ Fx = 0 = 24 - Dx \ Dx = 24 kips + Σ Fx = 0 = 24 - Dx \ Dx = 24 kips + Σ Fy = 0 = -24 + 21 + Dy \ Dy = 3k 5-45 Copyright © 2018 McGraw-Hill Education. Compute the support reactions for the arch in Figure P6.25. Determine the reactions and all bar forces for the truss if member AB is fabricated 0.25 in. -1 Condition (M = 0 at hinge) Indeterminate 1° (b) Remove Restraints -2 at B, 1 at C 3, Cut at D \ Indet 6° (c) Unstable: Member CD Cannot carry A Lateral load, but Frame ABC is Stable Remove Restraints: (d) Member BE is a Two Force Element 1 Horiz. 30 kips C B D 11.25' h = 15' 11.25' A E 10' 10' 10' P6.23 Reactions Ay = E y (Symmetry) = 30 kips + Σ MC = 0; (Freebody CE) 0 = +30 '10 - 30 ' 20 + H15 H = 20 kips ¬ Compute Forces on a Normal Section at "B", Freebody of "AB": + Σ Fx = 20 - Bx = 0 Bx = 20 kips + Σ Fy ; 0 = 30 - 15 - By By = 15 kips + Σ M B = 0; 30 k (10 ¢) - 20 k (11.25¢) - 15k (5¢) + M B = 0 - 300 + 75 + 225 = M B MB = 0 EQ of Parabola: y = kx 2 at x = 20 ¢, y = 15¢ 15 = k (20)2 15 3 = k = 400 80 3x 2 y = 80 dy 3x = dx 40 6-25 Copyright © 2018 McGraw-Hill Education. Each concrete curb weighs 240 lb/ft and each rail 120 lb/ft. Consider the structures in Figure P13.22, respectively.

Given: 2 E = 200 GPa and A = 1000 mm for all bars. A P6.6. What value of θ is associated with the minimum volume of cable material required to support the 100-kip load in Figure P6.6? The 2 allowable stress in the cable is 150 kips/in.

For all bars, area = 2 in. C P4.31. 4 1 Q3 = 60 kips 3 1 2 2 Q2 = 48 kips 3 16' 9' P15.4 Member 1: 1 2 3 4 \acute{e} 0.64 0.48 -0.64 -0.48 \acute{e} 0.48 0.36 -0.48 \acute{e} 0.48 0.36 0.48 0.36 0.48 0.36 0.48 0.36 0.48 0.36 0.48 0.36 0.48 0.36 0.48 0.36 \acute{e} 0.48 0.36 -0.48 \acute{e} 0.48 0.36 0.48 0.36 0.48 0.36 \acute{e} 0.48 0.36 0.48 0.36 \acute{e} 0.48 0.36 0.48 0.36 0.48 0.36 \acute{e} 0.48 0.36 0.48 0.36 \acute{e} 0.48 0.36 -0.48 \acute{e} 0.48 0.36 0.48 0.36 \acute{e} 0.48 0.36 0.48 0.36 \acute{e} 0.48 0.36 0.48 0.36 \acute{e} 0.48 0.64 \acute{e} 0.48 0.

settles 0.75 in. Draw the shear and moment curves. This will eliminate the appearance of a sagging roadway. 3 P16.7. For the frame shown in Figure P16.7, write the stiffness matrix in terms of the three degrees of freedom indicated. $\Delta D() = \Delta DO() - \delta DO Dy Dy 1840 196 Dy = K EI EI 1840 Dy = + 196 EI K EI 200' 40 And = = 200 40 K 1840 Dy = + 4.65 kN 200 + 196 \Sigma M A = 0 : RB = 24.84 kN <math>\Delta D = \Sigma Fy = 0$: Select Dy as the Redundant. The arch shown in Figure P6.21 has a pin 25 kips support at A and a roller at C. Indeterminate, stable Geometrically unstable: concurrent reactions b = 10 r = 4 n = 6 b = 38 r = 4 n = 21 (f) (b + r = 14) > (2n = 12) (b + r = 42) > (2n = 42) 2 deg. to prevent damage to the exterior walls, is the building frame sufficiently stiff to satisfy this requirement? The maximum height permitted at any point along the arch, hzmax, is 20 m. 2 kips/ft P9.40. Draw the influence lines for the reactions at A and the bending moment on Section 1 located 1 ft from the centerline of column AB. A B C 20' 30' P16.1 Restraining Moments: Joint B: M1 - 66.67 + 150 = 0 M1 = -83.33 ft \cdot k joint C: M2 - 150 = 0 M2 = 150 ft \cdot k é 83.33 ft \cdot k ú ú Force Vector F = êê ú 150 ft k · ëê ûú Stiffness Matrix: Unit Rotation @ B: + Joint "B": $\Sigma M B = 0$; K11 - 0.2 EI - 0.133EI = 0 K11 = 0.333EI + Joint "C": $\Sigma MC = 0$; K 21 - 0.067 EI = 0 Unit Rotation $\theta = 0$; K 21 = 0.067 EI Unit Rotation $\theta = 0$; K 22 = 0.133EI e 0 K 22

15' P4.54. F D 3m 2m A C B 30 kN 60 kN 4m 4m 4m P4.49 Entire Structure + $\Sigma M D = 0$; $\Sigma Fy = 0$; + C y (4) - 60(8) - 30(12) = 0 C y = 210 kN + $\Sigma Fx = 0$; Ax = 0 Isolate Joint A + $\Sigma Fx = 0$; Ax = 0 Isolate Joint A + $\Sigma Fx = 0$; Ax = 0; Ax = 0 Isolate Joint B $\Sigma Fy = 0$; + C y (4) - 60(8) - 30(12) = 0 C y = 210 kN + $\Sigma Fx = 0$; Ax = 0 Isolate Joint A + $\Sigma Fx = 0$; Ax = 0 Isolate Joint B $\Sigma Fy = 0$; + C y (4) - 60(8) - 30(12) = 0 C y = 210 kN + $\Sigma Fx = 0$; Ax = 0 Isolate Joint A + $\Sigma Fy = 0$; + C y (4) - 60(8) - 30(12) = 0 C y = 210 kN + $\Sigma Fx = 0$; Ax = 0 Isolate Joint A + $\Sigma Fx = 0$; Ax = 0 Isolate Joint B $\Sigma Fy = 0$; + C y (4) - 60(8) - 30(12) = 0 C y = 210 kN + $\Sigma Fx = 0$; Ax = 0 Isolate Joint A + $\Sigma Fx = 0$; Ax = 0 Isolate Joint B $\Sigma Fy = 0$; + C y (4) - 60(8) - 30(12) = 0 C y = 210 kN + $\Sigma Fx = 0$; Ax = 0 Isolate Joint A + $\Sigma Fx = 0$; Ax = 0 Isolate Joint B $\Sigma Fy = 0$; + C y (4) - 60(8) - 30(12) = 0 C y = 210 kN + $\Sigma Fx = 0$; Ax = 0 Isolate Joint A + $\Sigma Fx = 0$; Ax = 0 Isolate Joint B $\Sigma Fy = 0$; + C y (4) - 60(8) - 30(12) = 0 C y = 210 kN + $\Sigma Fx = 0$; Ax = 0 Isolate Joint A + $\Sigma Fx = 0$; Ax = 0 Isolate Joint B $\Sigma Fy = 0$; + C y (4) - 60(8) - 30(12) = 0 C y = 210 kN + $\Sigma Fx = 0$; Ax = 0; Ax = 0 Isolate Joint B $\Sigma Fy = 0$; Ax = 0; Ax = 0; Ax = 0; Ax = 0, $\Delta Fy = 0$; Ax = 0; Ax = 0

w = 3 kips/ft A B 4' C 6' D 5' P5.5 Freebody Diagram Segment Origin Range BC 4 ¢ f x1 £ 10 ¢ A V Equation M Equation + V1 = 8 - 3 x1 Σ MO = 0; 4x1 - 4 ¢) = 0 + BC B 0 f x2 £ 6 ¢ + 4k - V2 - 3x2 = 0 V2 = $4 - 3x2 - 4k(x2 + 4) - M2 - 3x2 æ x2 \ddot{o} \div cc$; z = 0 $c\dot{c} = 0$ $c\dot$

Compute the reactions and draw the shear and moment curves for the beam in Figure P9.19. Tension in FCE æ 30 kN \ddot{o} \div Tmax = (0.3125 + 0.1042) cc \div = 6.25 kN $c\dot{e}$ 2 \div # 12-41 Copyright @ 2018 McGraw-Hill Education. Using the conjugate beam or moment distribution method, calculate the ordinate of the influence line at section 8. P11.23. B 1 kip If the hoist load is 60 kips, determine reactions @C and moment @B: C y = 1(60 k) MB A Moving Load = 60 k MC = -15(60 k) = 900 ft \cdot k C MB 1 kip Cy 0 0 MC 0 0 -15 kip-ft MB 0 0 -15 kip-ft MB 0 0 -15 kip-ft 12-9 Copyright @ 2018 McGraw-Hill Education. 20 kN P8.29. 18 kN P4.28.

Continued 15 kips = Dy 2.4 kips/ft 28.8 kips = Dx Final Forces D 12 ft C 173 kip-ft 15 kips 30 kips 173 kip-ft 15 kips 24 ft 173 kip-ft 8 15 kips C 15 kips 24 ft 173 kip-ft 2.4 kips/ft B 12 ft A 28.8 kips = Ax 15 kips = Ay D -28.8 Shear (kips) Deflected Shape Moment (kip-ft) D 352.8 15 172.8 B B C C 172.8 -15 5.30 in. Analyze the frame in Figure P10.29. Constant shear occurs when no distributed load is present. The bridge is to be designed for a uniform live load of 700 lb/ft acting over the entire length of the bridge., columns A = 2 4 2 13.1 in. 20 kN P6.29. Each hanger can be assumed to provide a simple support for the suspended beam. The combined footing shown in Figure P5.51 is designed as a narrow reinforced concrete beam. E = 200 GPa, A = 25 × 10 mm, and 6 4 I = 240 × 10 mm. Moment Maximum Shear + $\Sigma M B = 0 = RA 30 - 20 2 - 20 30 - 20 2 - 20 30 - 20 2 - 20 30 - 20 2 - 20 30 - 20 2 - 20 30 - 20 2 - 20 30 - 20 2 - 20 2 - 20 30 - 20 2 -$

P8.20. E = 29,000 ksi and 2 members AB, BC, AD, and DC have A = 3 in. (Ignore dead load of the arch.) The allowable tensile capacity of the rod is 32 ksi.

Draw the influence lines for the bar forces in members DE, DI, EI, and IJ if the live load in Figure P12.29 is applied through the lower chord panel points.

B C a 1 D E 15 kips b 15 kips 9' F A H 10' c 1 12' G 10' P3.30 + Σ Mc = 0; -Fcd 9 ¢ + 9k -12 ¢ = 0 Fa = Fcd = 12 kips compr. A P12.12. Compare the results of Problem P13.21 (a) and (c). (b) Determine the vertical displacement at A produced by a vertical load of 24 kips directed downward. A B D E C 5m 5m 5m 5m 5m 5m P10.14 I = 100 × 106 mm4 F = 200 GPA, 2 EI (20A + 0B - 3\PsiAB) L = 4800 (e2 (-0.00125) - (3 (-0.0024)) u M AB = 0 = 0.005 Red M AB = -76.56 kN · m MCB 2 EI (20C + 0B - 3\PsiCB) L = 4800(0 + 0.00125) + 3 (-0.0024) MCB = = 40.56 kN · m Kinematic Unknown: $\theta B \theta A = -0.005$, $\theta C = 0$ M BC = 4800(20B + $\theta C - 3\Psi CB$) 24 = 0.0024 rad 10 10 '1000 = $\Psi CB = -0.0024$ red $\Psi AB = \Psi BA = \Psi BC - \Delta B$ M BC = -2 EI 2 '200 '106 (120 '10-6) = = 4800 kN · m L 10 2 EI M BA = (1) (20B + 0A - 3\Psi BA) L 2 EI M BC = (20B - 3\Psi CB) (2) L Equilibrium at B = 4800 (2 '0.00125 + 0 - 3 '(-0.0024)) = 46.56 kN · m + \Sigma M A = 0 = 10VBA - 46.56 - 76.56 VBA = 12.312 kN \Sigma Fy = 0 : VAB - VBA = 12.312 kN \Sigma Fy = 0 : VAB - VBA = 12.312 kN \Sigma Fy = 0 : VAB - VBA = 12.312 kN \Sigma Fy = 0 : VAB - VBA = 12.312 kN \Sigma Fy = 0 : VAB - VBA = 12.312 kN \Sigma Fy = 0 : VAB - 3\Psi BA - 3\Psi

1 in.2 AE 5 = 2 6 D 1 in.2 FQ2 L -1 (C) $\delta BB = \Sigma Q$ -System (kips) -2/3 (C) C 3 in.2 0.5 1 E P-System (kips) = 0 X B = By = 18.6 kips D 0 0 A B 0 -0.25 in. (OK) Deflected Shape 7-58 Copyright © 2018 McGraw-Hill Education. The moving load shown has to be increased by an impact factor listed in Table 2.3. (a) Position the moving load to compute the maximum moment.

K P4.39. VD = 0.035(3k) = 0.11k M D = -10.7(3k) = -32.1 ft · k 12-27 Copyright © 2018 McGraw-Hill Education. 16 kips P13.4. Assuming the location of the point of inflection in the girder in Figure P13.4, estimate the moment at B. P9.44. M = 60 kip ft P9.5. Compute the reactions, draw the shear and moment curves, and locate the point of maximum deflection for the beam in Figure P9.5. Repeat the computation if I is constant over the entire length.

Determine the weight of the tension ring required to balance the vertical components of the cable forces. Load moves along girder BC. 6 kips 6' D 9' 12 kips B A 12' 12' P4.31 + Σ M A = 0; - 6 k (15¢) + 12 k (12 ¢) - By (24 ¢) = 0 By = 2.25k + Σ Fy = 0; Ay + 2.25k - 12 k = 0 Ay = 9.75k Freebody Right of Section(1) Compute Force in Bar DB. 6 21.5 % Red., 2 and E = 29,000 kips/in.

For the computer analysis, replace the tapered members by 3-ft-long segments of constant depth whose properties are based on each segment's midspan dimensions; that is, there will be 9 members and 10 joints. Draw the shear and moment curves for each member of the frame in Figure P5.25. Determine the forces in the cable and the post, and determine the reactions at A and C. P11.32. The right end of the truss rests on an elastomeric pad at G. P4.52. K J 3' I L H 6' 12' A B D C 30 kips E 90 kips F 30 kips 6 @ 15' = 90' BL, KJ, JD, and LC P4.36 Compute Forces in BL, KJ, JD, and LC P4.36. W = 2 kips/ft P9.14. P8.33.

= +13.2 k êë úû 2 1 + RC max. To avoid ponding, beams may be cambered upward so rain water cannot accumulate at the center regions of the roof.

Determine the reactions and all bar forces for the truss if support A settles by 20 mm. D 9' 5 kips P11.8. Analyze by moment distribution. 100 kN P4.20. If the maximum compression in members AB and BC will be 750 kips, what size diameter tension rod is required? Evaluate the ordinates at points A, B, C, and D. The magnitude of joint deflections is listed in the table on page 6.32A. The supporting hangers are closely spaced, generating a smooth curved cable. Also assume that the roadway beams and the columns do not restrain the arch. E P12.33.

30 kips P11.1. Analyze each structure by moment distribution.

Use support B as the redundant.

Since equilibrium not possible, structure is unstable. Compression, but in Tension, but in Tension, but in tension FAR, BB, MC, VC, and shear to the left of support D He or the beam in Figure P12.54. B 60 kips C 36 kips D E F 16 kips 24 kips 12⁴ A G + 9⁴ 9⁴ 9⁴ 9⁴ 9⁴ 4.2 Ka C + 9⁴ 0 C = 0 S

(NG) Relative Displacement between 2nd and 3rd floors = 15.58 - 5.81 = 9.77 in. 8" concrete masonry partition 9.5' concrete floor slab piping mechanical duct wide flange steel beam with fireproofing ceiling tile and suspension hangers Section P2.3 Uniform Dead Load WDL Acting on the Wide Flange Beam: Wall Load: 9.5¢(0.09 ksf) = 0.855 klf Floor Slab: 10 ¢(0.05 ksf) = 0.50 klf Steel Frmg, Fireproof'g, Arch'l Features, Floor Fluishes, & Ceiling: 10 ¢(0.024 ksf) = 0.24 klf Mech'l, Piping & Electrical Systems: 10 ¢(0.006 ksf) = 0.06 klf Total WDL = 1.66 klf 2-4 Copyright © 2018 McGraw-Hill Education.

Axis at middepth. Load P was removed temporarily and tension rod AC was added.

It would appear at first that the moment at joint B would have a larger magnitude in the taller frame in P13.22, however the knee brace brings the column height to 10 ft reducing the moment arm. Computer study of arch with a continuous floor girder. Wind Pressure on Side Walls I = 1.15 for hospitals p = qz GC p = 49.05(1.52)(0.85)(-0.7) K zt = 1; K d = 0.85 p = -44.36 psf Kz, Read in Table 2.4 Elev. If bar AE is fabricated 85 in too long, how far to the right must the roller at B be displaced horizontally so that no vertical deflection occurs at joint D? Beam ABC is supported by a three-bar truss at point C and at A by an elastomeric pad that is equivalent to a roller. B 30' D A E 20' 20' P9.24 Select RC as the Redundant Released Structure W/Applied Load All Bars E = 30,000 Ksi & A = 15 in 2 Geometry $\delta CX = 0 \ \Delta CO = 0.25 \ \delta CC = 0.25 \ \phi = 0.25$

Also E = 29,000 kips/in. Use E = 29,000 kips/in. 2. 2 3 100 kips 4 1 5 3 4 1 20' P4.54 Case 1: Determine the reactions and the forces in members 4 and 5 if the area of all bars is 2 10 in. 10 kips P5.7. Write the equations for shear and moment using the origins shown in the figure. F w = 4 kN/m P3.17. J P4.38. 4 2 Given: I = 100 in. Indicate if each link is in compression or tension. 50' h=? The cable is supported at point D by a rigid member DF. C D E RD 12' A 5' 10' RA P12.7 12-8 Copyright © 2018 McGraw-Hill Education.

Continued Case B 46.4 ... -9 18 -34.2 68.4 0 Sidesway S 1 in. 9 kips/ft P3.10. E = 29,000 kips/in.2, I = 2000 in.4. A 30 kips B C D elastomeric pad 10' 10' P8.19 Compute δ f (Midspan) Diagram Segment Orgin Range MP MQ AB (= CD) A $0 \le x1 < 10 \ 30x1 \ x1/2 \ \delta$ center line B $0 \le x2 \le 5 \ 300 \ 5 + x1/2 \ WQ = UQ \ dx \ EI \ x = 10 \ x = 5 \ x \ dx \ xo \ dx \ w = 2 \ \delta$ (30 x) + 2 δ ccct cr line B $0 \le x2 \le 5 \ 300 \ 5 + x1/2 \ WQ = UQ \ dx \ EI \ x = 10 \ x = 5 \ x \ dx \ xo \ dx \ w = 2 \ \delta$ (30 x) + 2 δ ccct cr line δ f = 0.86 in 8-22 Copyright © 2018 McGraw-Hill Education.

10800 $\Delta C = EI \Delta C + t C = 120D =$ Member ABCD 9-51 Copyright © 2018 McGraw-Hill Education. 9-6 Copyright © 2018 McGraw-Hill Education. Assuming that the ends of the girder are 25 percent of the fixed-ended moments, compute the reactions and draw the moment curve for the girder. Using a finite summation, compute the initial deflection at midspan for the beam in 2 Figure P8.42. B C 24' D A 48' P9.47 Coef of Temp $\alpha = 6.5^{10-6} E = 30,000$ ksi For $\delta DD = 2 \circ 24$ M E2 0 æ x 3 $\ddot{o} \div \div \delta DD = 2 \circ 24$ M E2 0 æ x 3 \ddot{o}

For the frame in Figure P5.47, draw the shear and moment curves for all members. w = 6 kips/ft A B C D E 40' 8' 24' 8' P3.37 æ 24 ¢ ö FBD "CD" Force in Hinges CY = DY = 6 k/1 çc \div = 72 k $ce^2 \div e$ FBD "ABC" + $\Sigma M A = 0$; $\Sigma Fy = 0$; + 2 6 k/ft (48¢) + 72 k (48¢) - By (40¢) = 0 2 By = 259.2 k Ay - 6 k/ft (48¢) + By - 72 k = 0 Ay = 100.8 k FBD "DEFG" + $\Sigma M E = 0$; -72 k (8¢) + 318k (18.5¢) - Fy (40¢) = 0 Fy = 132.7 k $\Sigma Fy = 0$; + F G shear plate connections -72 k - 318k + E y + 132.7 k = 0 E y = 257.3 k 3-38 Copyright © 2018 McGraw-Hill Education. Released structure w/unit value for redundant X1 = FAC = 1 k Q-system for $\Delta 10$, $\delta 11$ P-system for $\delta 11$, $\delta 21$ Compatibility Equations: $\Delta 1$, AC REL = 0 = $\Delta 10 + \delta 11 X1 + \delta 12 X2 \Delta BX = 0 = \Delta 20 + \delta 21 X1 + \delta 22 X 2 9-42$ Copyright © 2018 McGraw-Hill Education. Determine the reactions at A and C and the tension in rod AC. A C D B K = 40 kN/m 5m 4m P9.20 Compatibility EQ.

9-35 Copyright © 2018 McGraw-Hill Education. G G P4.37. bonnet truss 10' F D E 10' 10' 10° F 10' 10' 10° F 70° F 360' A B 20' C 20' P9.49 9-56 Copyright © 2018 McGraw-Hill Education. 4.5 kip • ft w = 4 kips/ft A B 1.69 kips 8' C 6' 15.19 kips P5.33 Δ M AB = -1.69'8 = -13.52 kip · ft 4.5 -13.52 = -9.02 kip · ft 4.5 -13.52 = -9.02 kip · ft Locate Point of Inflection 13.5 3.375 = y x 4x = y Δ M = Area V - Diagram 1 1 13.76 = yx = (4 x) x 2 2 13.76 2 x = 6.88 2 x = 2.62 ¢ 5-35 Copyright © 2018 McGraw-Hill Education. Show all forces acting on a free-body diagram of joint C. (Ignore the 4-kip load.) (b) If a hoist is also attached to the lower chord at the midpoint of the end panel on the right (labeled joint 6*) to raise a concentrated load of 4 kips, determine the forces and moments in the lower chord (members 5 and 6). Continued Virtual Work: $\Delta 10 : \Sigma Q \cdot \delta P = \Sigma F Q$ FP LAE / $\delta 21 : 1k \cdot \delta 21 = \delta 11 = 0.0032 ¢ (\neg (-0.8k) 60 k (20 '12) (2) 2(30,000) 1k \cdot \Delta 10 = (-0.6)45 (15'12) 1(-75)(25'12) + 2(30,000) 4(30,000) k + \Delta 10 = -0.6525 ¢ cap Opens \delta 12 : 1k \cdot \delta 12 = \delta 22 : 1k \cdot \delta 22 = -1(60)(20 '12) 2(30,000) \Delta 20 = -0.24 ¢ substitute displacements into compatibility equations: X1 = 37.62 k (-0.8) (20 '12) (2) (20,000) 622 = 0.004 ¢ (\neg / \delta 11 : FQ = FP; Q \cdot \delta 11 = FQ2 LAE 1k \cdot \delta 11 = (-0.8)(-1)(20 '12) 2(30,000) \delta 12 = 0.0032 ¢ (\neg \Delta 20 : 1k \cdot \Delta 20 = 2 (-1)(-0.8)(20 '12) 2(30,000) \delta 12 = 0.0032 ¢ (\neg \Delta 20 : 1k \cdot \Delta 20 = 2 (-1)(-0.8)(20 '12) 2(30,000) \delta 22 = 0.004 ¢ (\neg / \delta 11 : FQ = FP; Q \cdot \delta 11 = FQ2 LAE 1k \cdot \delta 11 = (-0.8)(-1)(20 '12) 2(30,000) \delta 12 = 0.0032 ¢ (\neg \Delta 20 : 1k \cdot \Delta 20 = 2 (-1)(-0.8)(20 '12) 2(30,000) \delta 12 = 0.0032 ¢ (\neg \Delta 20 : 1k \cdot \Delta 20 = 2 (-1)(-0.8)(20 '12) 2(30,000) \delta 22 = 0.004 ¢ (\neg / \delta 11 : FQ = FP; Q \cdot \delta 11 = FQ2 LAE 1k \cdot \delta 11 = (-0.8)(-1)(20 '12) 2(30,000) \delta 12 = 0.0032 ¢ (\neg \Delta 20 : 1k \cdot \Delta 20 = 2 (-1)(-0.8)(20 '12) 2(30,000) \delta 12 = 0.0032 ¢ (\neg \Delta 20 : 1k \cdot \Delta 20 = 2 (-1)(-0.8)(20 '12) 2(30,000) \delta 12 = 0.0032 ¢ (\neg \Delta 20 : 1k \cdot \Delta 20 = 2 (-1)(-0.8)(20 '12) 2(30,000) \delta 12 = 0.0032 ¢ (\neg \Delta 20 : 1k \cdot \Delta 20 = 2$

Concrete Frame h = 30 ¢ Building Height Vbase = Vmax. Analysis Exact Analysis by Moment Distribution PL 20(3) FEMs AB: = 7.5 kN · m 8 8 wL2 5(6)2 FEMs BC: = 15 kN · m 12 12 D.F . Fx P3.23. Continued (b) The lateral displacements are significantly increased if shear connections are used. for exterior columns. Use symmetry to simplify the analysis.

Analyze the frame in Figure P11.17 by moment distribution. and E = 29,000 kips/in. w = 1 kip/ft P9.16. (b) Assuming that all 4 columns are 12 in square (I = 1728 in. Express the slope in degrees and the deflection in inches. cable D C cable 10' A B P6.7 Compute Jacking Force T: ft · k 135 5ft H = 27 kips Hh = M ; H = Hh1 = 75ft · k ft · k 75 27k h1 = 2.78ft tan $\theta = h1 10$ ft. The columns and girder of the indeterminate rigid frame in Figure P5.57a are fabricated from a W18 × 130 wide flange steel section: 2 4 A = 38.2 in. Continued Deflected Shape: Case 2 From the computer analysis, the horizontal deflection at Joint 4 is 0.702 in.

In addition to the applied load, support A rotates clockwise by 0.005 rad. A two-lane highway bridge, supported on two deck trusses that span 64 ft, consists of an 8-in. B A C post cable D 15' 15' P6.9 Before Post & Cable are Added: Determine Post Force P: M B due to post force = Ppost = $4(112.5ft \cdot k)$ 30 ¢ Ay = C y = 7.5k PL set = $112.5ft \cdot k = 15k$ PC set = 15k - 7.5k = 7.5k 6.10 Copyright © 2018 McGraw-Hill Education. Compr in bar CG. Check your answer using RISA-2D. Pab 2 PL -9(20)(40)(2) 9(6) = 8 8 L2 (60)(2) = 147.5 ft \cdot k FEM AC = 9(40)(202, 9(60) + 8 (61)(2) = 80, 000) = 8 (200, 000) = 12 (200, 000) = 12 (200, 000) = 12 (200, 000) = 12 (200, 000) = 12 (200, 000) = 12 (200, 000) = 12 (200

The cross section of the rectangular ring is 12 in. Determine the load P such that all the members in the three-hinged arch in Figure P6.29 are in pure compression. 1/2 1/2 PL/2 PL/8 Equilibrium to the frame: B 2M A = 0 = 16(10) - 6(20) + C x (15) C 10 ft 10 ft POI C x - 2.67 kb j x - 2 Kb y - 0 x +

P11.20.

Determine the reactions and the moments induced in the members of the frame in Figure P11.27. by Beam Analogy: FC = 13-17 Copyright © 2018 McGraw-Hill Education. 30 kips P14.5. Analyze the steel rigid frame in Figure P14.5. After member end moments are evaluated, compute all reactions and the moment diagram for beam BC. B x D h = 150' 100' A C y L = 175' P12.24 y = 4 hx 2 8(150) x 2 = 0.0392 x L 21752 @ yD = -50 ¢, x= yL2 = 50.25 ¢ hdy = 0.0392 x L 21752 @ yD = -50 ¢ hdy = 0.0392 x L 2175 @ yD = 0.0392 x L 2175 @ yD = -50 ¢ hdy = 0.0392 x L 2175 @ yD = -50 ¢ hdy = 0.0392 k L 2 4 k

Since box symmetrical about horiz and vertical £'s can adjust stiffness of all sides by 12 & eliminate c.o.m. wL2 0.50(13)2 = 7.04 k · ft 12 12 I 1 K AB = $i = 0.0714 I 7 2 \Sigma K S c = .1099 I .0385 I Distr. = -7.2 ft · k 2 12-5 Copyright © 2018 McGraw-Hill Education. P13.17. In addition to the 16-kip load, support A also rotates clockwise by 0.001 rad and support B settles 12 in. (b) Repeat the computation if the 10-kip load is also located at joints C and D, respectively. E w = 1 kip/ft C F G 12' A H 6' 20' 6' P5.41 By Symmetry Ay = H y = 1k / ft (6 c) = 6 k Ax = 0 FBD: Cantilever Beams "BC" & "FG" Opp.$

2 3 6 @ 6.67' = 40' C4 A B2 G1 B3 B4 2 @ 10' = 20' C2 B G4 G3 G2 B1 C 5 @ 8' = 40' C3 C1 40' 20' P2.4 (a) AT = 8(40) = 320 ft , K LL = 2, AT K LL = 640 > 400 w 2 æ çè 15 ö ÷ L = 60 çç0.25 + 60 ÷ ÷ = 50.6 psf > , ok 2 640 ø B1 and B2 w = 8(50.6) = 404.8 lb/ft = 0.40 kips/ft (b) AT = 6.67 2 w = (20) = 66.7 ft , K LL = 2, AT K LL = 133.4 < 400, No Reduction 2 6.67 (60) = 200.1 lb/ft = 0.20 kips/ft (c) AT = 6.67 2 w = (20) + 10(10) = 166.7 ft , K LL = 2, AT K LL = 33.4 < 400, No Reduction 2 6.67 (60) = 200.1 lb/ft = 0.20 kips/ft (c) AT = 6.67 2 w = (20) + 10(10) = 166.7 ft , K LL = 2, AT K LL = 33.4 < 400, No Reduction 2 6.67 P (60) = 200.1 lb/ft = 0.20 kips/ft w 2 P = q (Wtrib) (L beam) 2 = 60(10)(20) = 6000 lbs = 6 kips 2 G1 æ 40.20 ö ÷ 2 çè 2 + 2 ÷ ø 36 = 1080 ft , K LL = 2, AT K LL = 2, AT K LL = 233.4 < 400, No Reduction 2 6.67 P (60) = 200.1 lb/ft = 0.20 kips/ft w 2 P = q (Wtrib) (L beam) 2 = 60(10)(20) = 6000 lbs = 6 kips 2 G1 æ 40.20 ö ÷ 2 çè 2 + 2 ÷ a 36 = 1080 ft , K LL = 2, AT K LL = 2,

P14.7. Analyze the reinforced concrete frame in Figure P14.7. Determine all reactions. B 30' D A E 20' 20' P9.25 Area all Bars = 5 in 2 E = 30,000 kips/in 2 Select RC as Redundant. Consider the building in Figure P2.22, which has a width into the page of 35 ft. P13.13.

9-53 Copyright © 2018 McGraw-Hill Education. smaller). J 40 kips I 40 kips H G F 10' A B C D E 4 @ 15' = 60' P13.16 Assume (1) P.I. @ Midspan and (2) Shear Divide Equally $\Sigma MO = 0$; 60(7.5) - F1(10) = 0 F1 = 45k C F2 = F1 F2 = 45k T FBD Member AB $\Sigma MO = 0$; 60(22.5) - 40(7.5) - F3(10) = 0 F3 = 105k C F4 = F3 = 105k T FBD Member BC FBD Member BI FBD Member HC: Note: No moment or shear only axial force 13-23 Copyright © 2018 McGraw-Hill Education.

42.2 11-37 Copyright © 2018 McGraw-Hill Education. 150 kips B C 12' A E D 6' 12' P8.5 FP L AE -1(-210 k) (12 ¢ 12%) k 1 · δ By = 5 in 2 (29000 k/in 2) Σ Q · δ P = Σ FQ = 0.209 ¢¢ 1k · δ Bx = (+2.236 k (67.1k)(13.4 ¢ 12%)) + ((-2 k)(-210)(12 ¢ 12%)) 5 in 2 (29000 k/in 2) δ BX = 0.584 ¢¢ 8-7 Copyright © 2018 McGraw-Hill Education. (a) Assuming that no loads act in Figure P9.16, compute the reactions if support B is constructed 0.48 in. -46 P6.12. P2.11. Draw the influence lines for both the horizontal and vertical reactions at A and the moment at D.

(b) With the dead load on both spans and the live load on the left span ABCD, determine the shear, moment, and axial load diagrams for both spans, the axial load in the vertical cable tower. Continued Summary of the final member end moments and reactions. and I = 240 in. Modify stiffness as discussed in Section 11.5. E is constant. B C D 3' A rocker support hinge P = 12 kips 4' 20' P5.20 Shear @ A: Entire Structure: k V 12 = A ¢ 5 4¢ @ B x = 5¢ VA = 9.6 k + 8k = 17.6 k Axial Force @ A: A 12 k = A ¢ 5 3¢ A@ A = 7.2 k 2 k/1 (5¢) (3¢) = 6 k 5 \ A = 7.2 k + 6 k = 13.2 k @ B, x = 5¢ : FBD "BC" + 2MC = 0; - 68ft k - 22 k (20 ¢) - 2 k/1 (20)2 + By (20 ¢) = 0 2 By = 45.4 k \SigmaFy = 0; + -22 k - 2 k/1 (20 ¢) + 45.4 k + C y = 0 C y = 16.6 k FBD "CD" \SigmaFy = 0; + + \Sigma M D = 0; -16.6 k (20 ¢) + M D = 0 2 M D = 732 ft k 5-22 Copyright © 2018 McGraw-Hill Education. (in.3) 5010 - %Inc.

Compute the reactions and draw the shear and moment curves for the beam if support A rotates clockwise by 0.005 rad. In Bar FAL (2 Trusses) F 3 3 1 FKJ = 10 k + 24 k + 20 k 8 4 2 = 3.75 + 18 + 10 = 31.75 kips 212-42 Copyright © 2018 McGraw-Hill Education. w = 3 kips/ft E B 1.5I I C 1.5I I A 1.5I G 30' F 30' F 30' P11.20 Symmetric structure with summetric loads; evaluate half the structure: $\theta \in C - D = 0$ Joint "B": Distribution Factors EI DFBA = 0.5 20 c 1.5 EI K BC = DFBC = 0.40 30 c EI KCH = DFCH = 0.40 20 c 1 æ1.5 EI ö ÷ KCD = c ÷ DFCD = 0.20 2 c è 30 ÷ ø \Sigma K @ C = 0.125 EI 1.0 KCB = FEM = 20' I I H 30' D wL2 3k / ft (30)2 = 225ft k 12 12 11-25 Copyright © 2018 McGraw-Hill Education. E B D 12' C H 9 kips 30' F G 9 kips 9 kips 6 @ 15' = 90' P6.13 H · h = M H · 12 c = 945 ft · k H = 78.75 k Sag at D (By Symmetry hB = hD) H · hD = M hD = 540 l · k = 6.86 ft 78.75k Max Tension at "A" or "B" Tmax = 78.752 + 182 = 80.78 kips 6-14 Copyright © 2018 McGraw-Hill Education. Analysis 13-2 Copyright © 2018 McGraw-Hill Education.

FAH = X AB = 16.67 kN 4-21 Copyright © 2018 McGraw-Hill Education. This moment rotates the B-end of member BC in the counterclockwise directions to shift to the right. P13.2. Continued CASE 2 L1 = 12 m In this case, because of the longer length of span AB, a much larger conterclock wise moment is applied member AB to the left end of member BC.

4' 4' 8' 4w H = 200 kN Shear (kips) General Cable Theorem at $\frac{1}{2}$ span: -4w M = Hhz 40 w = 200(3) 8100 40w w = 15 kN/m Moment (kip-ft) 8(15) 2 Ay = By = 60 kN Ay = 15 kN/m Moment (kip-ft) 8(15) 2 Ay = By = 60 kN Ay = By = 60 kN Ay = 15 kN/m Moment (kip-ft) 8(15) 2 Ay = By = 60 kN Determine the forces in all bars of the trusses. (a) Mmax 20 kips A 20 kips B 8' 40' P12.49 P = 1.2520 (20 k) = 25k Mmax = 405 ft · k 107, 400,000 \Delta max = @ 2.4 ¢ Right of "C" EI (b) Loads Centered on Beam $\Delta mac = 108, 749,000$ @£ EI Max. Analysis Compare Closely. (b) Using the influence line in part (a), determine the maximum value of the reaction RB produced by two concentrated 20-kip wheel loads spaced 8 ft apart. Repeat part (b) in problem P6.33 if a continuous girder with 2 4 A = 102.5 in and I = 40,087 in. (a) Compute the vertical deflection of joint D produced by the 30-kip load in Figure 2 P8.13. Continued Case B To Account for Force of 2.91k, Multiply Case BX 2.91 = 0.458 6.34 Round to 0.46 Final Results Deflected Shape 11-40 Copyright © 2018 McGraw-Hill Education. (b) Repeat computations given that support C moves upward a distance of 288/(EI) when the load is applied. A B C 9' 6' P9.1 36 kips Selecting Cy as the redundant, the compatibility equation is: C A (Δ CS + Δ CO) + δ CC XC = Δ C 153 1125 = EI 3EI = tCA = xAM/EI Δ C B δ CC = Δ CO Δ CO δ CC A C 1 kip 1 324 17, 496 = (6 + 6) 9 = 2 EI EI A B C 4 Therefore, with E = 30,000 ksi and I = 320 in. In a determinate structure, for a given set of loads, only one load path is available to transmit the loads into the supports, whereas in an indeterminate structure, multiple load path sexist (Section 3.10). The actual point of inflection in the columns was at 0.625h vs the assumed P.I. location at 0.6h. (b) Determine the maximum axial force in the arch in Case B. 3 kips/ft P8.21. (a) hinge (b) hinge (c) fixed base (d) hinge hinge (e) (f) P5.53 5-55 Copyright © 2018 McGraw-Hill Education. and 2 E = 200 GPa. For the cable A = 6000 mm and E = 150 GPa. 6m B A C 6m 2m 280 kN P8.35 Virtual Work dx L + Σ FQ FP EI AE 6 2 dx dx æ 528(1.88)6 2 \ddot{o} ÷ + 1kN · $\delta CV = \dot{o}$ (0.33 x1) (93.3 x1) + \dot{o} (x2)(280 x2) + ccc ÷ EI EI cè AE ø ÷ 0.0 2217.6 746.67 8422.35 = + + EI EI AE Note : Axial Deformation is Ignored in the Beam O · $\delta CV = \chi \dot{o}$ MO M P $\delta CV = 76.3$ mm P-System for YCV 8-41 Copyright © 2018 McGraw-Hill Education. A jump in moment diagram arises from an applied concentrated moment. 10 kips D E F G H 5' C L M N O P 5' B J pinned connections typical 10' pinned base typical A K 6 @ 5' = 30' P13.22 Unlike a typical horizontal load to the column @ "B" and "J", causing a moment in columns "ABC" & "KJI" The structure is symmetrical and half the lateral load goes to each column: $Ax = Kx = 5k \neg$ Entire Structure: MTotal = 10 k (20 ¢) = 200 ft · k + $\Sigma M K = 0$; 10 k (20 ¢) = 0 Ay = 6.67k FBD Column "AB" + $\Sigma M B = 0$; - M BA + 5k (10 ¢) = 0 M BA = 50 ft · k FBD Column "ABC" MBA = 50 ft · k FBD C component BLx of the force in the knee brace. D 3m B 70 kN C 3m A 5m P10.31 Member end moments, recognizing $\Psi AB = \Psi = \Delta$, $\Psi BC = -\Psi 3 2 EI 2 EI M BC = 2 \theta B + \theta C$), MCB = ((2 $\theta C + 3\Psi$), M DC = 3 ($\theta C + 3\Psi$), M DC = 3 ($\theta C + 3\Psi$) 3 2 EI 2 EI M BC = 2 $\theta B + \theta C$), MCB = ((2 $\theta C + 3\Psi)$, M DC = 3 ($\theta C + 3\Psi$) 3 2 EI 2 EI M BC = 2 $\theta B + \theta C$), MCB = ((2 $\theta C + 3\Psi)$) 3 2 EI 2 EI M BC = 2 $\theta B + \theta C$), MCB = ((2 $\theta C + 3\Psi)$) 3 2 EI 2 EI M BC = 2 $\theta B + \theta C$), MCB = ((2 $\theta C + 3\Psi)$) 3 2 EI 2 EI M BC = 2 $\theta B + \theta C$), MCB = ((2 $\theta C + 3\Psi)$) 3 2 EI 2 EI M BC = 2 $\theta B + \theta C$), MCB = ((2 $\theta C + 3\Psi)$) 3 2 EI 2 EI M BC = 2 $\theta B + \theta C$), MCB = ((2 $\theta C + 3\Psi)$) 3 2 EI 2 EI M BC = 2 $\theta B + \theta C$), MCB = ((2 $\theta C + 3\Psi)$) 3 2 EI 2 EI M BC = 2 $\theta B + \theta C$), MCB = ((2 $\theta C + 3\Psi)$) 3 2 EI 2 EI M BC = 2 $\theta B + \theta C$), MCB = ((2 $\theta C + 3\Psi)$) 3 2 EI 2 EI M BC = 2 $\theta B + \theta C$), MCB = ((2 $\theta C + 3\Psi)$) 3 2 EI 2 EI M BC = 2 $\theta B + \theta C$), MCB = ((2 $\theta C + 3\Psi)$) 3 2 EI 2 EI M BC = 2 $\theta B + \theta C$), MCB = ((2 $\theta C + 3\Psi)$) 3 2 EI 2 EI M BC = 2 $\theta B + \theta C$), MCB = ((2 $\theta C + 3\Psi)$) 3 2 EI 2 EI M BC = 2 $\theta B + \theta C$), MCB = ((2 $\theta C + 3\Psi)$) 3 2 EI 2 EI M BC = 2 $\theta B + \theta C$), MCB = ((2 $\theta C + 3\Psi)$) 3 2 EI 2 EI M BC = $0, 0 = \theta B + \theta C - \Psi 155161$ Joint C: MCB + MCD = $0, 0 = \theta C + \theta B + \Psi 15513$ Therefore: $-\theta B = \theta C$ and $\Psi = \theta B 15$ Shear Equilibrium: 70 kN B $\Sigma Fx = 0 = 70 - V1 - V2 \psi CD 70$ kN M AB = $70 = -M AB - M BA + 71.59 EI - 71.59 \theta C = EI 62.04 \Psi = EI \theta B = Dx = 35$ kips -3535 Shear (kips) V2 -76.4 Ax = 35 kips D C MCD + M DC M AB = -76.4 kip-ft Δ ψ AB V1 3 4 EI 2 EI 4 EI - 2 EI 70 = $\Psi + \Psi \theta B + \theta C + 3 3 3$ Solving: 3 Δ 28.6 28.6 B 28.6 Moment (kip-ft) M BA = -28.6 kip-ft MCB = -76.4 kip-ft A -76.4 Deflected Shape 10-38 Copyright © 2018 McGraw-Hill Education. 6-38 Copyright © 2018 McGraw-Hill Education. and E = 30,000 2 kips/in. 5 kips/ft C B E hinge D 20 kips 8' A 4' 6' 6' P3.10 Freebody Diagram Right of Hinge D: + $\Sigma Md = 0$; 30 k (3¢) - Re 6 ¢ = 0 Re = 15 kips + $\Sigma Fy = 0$; Dy - 30 k + Re -15k = 0 Dy = 15 kips + \Sigma Fy = 0; Dy - 30 k + Re -15k = 0 Dy = 15 kips + $\Sigma Fy = 0$; Dy - 30 k + Re -15k = 0 Dy = 15 kips + \Sigma Fy = 0; Dy - 30 k + Re -15k = 0 Dy = 15 kips + \Sigma Fy = 0; Dy - 30 k + Re -15k = 0 Dy = 15 kips + \Sigma Fy = 0; Dy - 30 k + Re -15k = 0 Dy = 15 kips + \Sigma Fy = 0; Dy - 30 k + Re -15k = 0 Dy = 15 kips + \Sigma Fy = 0; Dy - 30 k + Re -15k = 0 Dy = 15 kips + \Sigma Fy = 0; Dy - 30 k + Re -15k = 0 Dy = 15 kips + \Sigma Fy = 0; Dy - 30 k + Re -15k = 0 Dy = 15 kips + \Sigma Fy = 0; Dy - 30 + $\Sigma Fy = 0$; Ay - 45k - 15k - 20 k = 0 Ay = 80 kips 3-11 Copyright © 2018 McGraw-Hill Education. Compute the horizontal deflection of joint B. The cable-supported roof for a summer theater, shown in Figure P6.15, is composed of 24 equally spaced cables that span from a tension ring at the center to a compression ring on the perimeter. 6-40 Copyright © 2018 McGraw-Hill Education. All members in the Howe Truss are in tension, except for the top chord, which requires less cross-sectional area. The analysis approach is the same for both problems P13.21 and P13.22. The distance between the centroids of the top and bottom chords equals 9 ft. P15.3. Continued Compute Force in Bar 1, Use Equation 15.51 to Determine Elongation é Si ù = é1 0 0 0ù é 0 ù ê ú ê ú ê ú ê ú ê ú ê ú ê ú ê é 0.192 ú ê ú ê -0.865ú êë úû Solve Gives: Si = 0 S j = 0.192 From Equation 15.53 F1 = AE é 2(30,000) S - Si ùûú = [0.192 - 0] = 48k L ëê j 20 '12 Similiarly Forces in other Bars Gives: F2 = -60 k F3 = -36 k 15-8 Copyright © 2018 McGraw-Hill Education. = $c \div \div cc \div \div (3)1.2 k / ft$. Evaluate V and M at point C using the equations. and I = 2460 in. Analyze the frame in Figure P11.14 by the moment distribution method. B P8.45. (see top figure for results). B C 8" 6' 8" 8" A D 4 kips/ft 3' 3' 10 kips 12' P11.15 I 1æ I ö 4 1 K AB = , K BC = cc \div \div , DFBA = , DFBC = 6 2 cè12 ø 5 5 FEM BC = -22.5k ft (by superposition) FEMCB = 22.5k ft wL2 4(6)2 = = -7.2 k ft 20 20 wL2 4(6)2 = = -4.8k ft 30 30 FEM AB = -4.8k ft FEM BA 11-18 Copyright © 2018 McGraw-Hill Education. A B C D E 5 @ 5 m = 25 m P12.21 A B C D E F 5 @ 5 m P12.21 A B C D E F 5 @ 5 m P12.21 A B C D E F 5 @ 5 m P12.21 A B C D E F 5 @ 5 m P12.21 A B C D E F 5 @ 5 m P12.21 A B C D E F 5 @ 5 m P12.21 A B C D E F 5 @ 5 m P12.21 A B 2018 McGraw-Hill Education. 10' C B 15' A 20' P13.4 P Assume the point of inflection (POI) is less than the fixed-fixed beam under the same loading, say 0.2L = 4 ft. 2-26 Copyright © 2018 McGraw-Hill Education. D 12' 3 kips A B C 8' 8' 3' P8.28 Ignore axial deformations, E = 29000 ksi I = 120 in.4 & DX Segment Orgin Range MQ1 MQ2 Mp AE A 0 £ $(-x)(-3x) dx EI \circ 0 \ 0 \ 0 \ 0 \ 1k \cdot \delta DX = \delta DX dx EI \ 6c \ 3 \ 1728 \ (-0.25x \ 2) \ dx + \delta \ (1.25x \ -3)(4x \ -18) \ dx + \delta \ 18x \ dx EI \ \delta 0 \ 0 \ 0 \ 6 \ 6 = -0.0833 \ x \ 3 + (1.67x \ 3 \ -17.25x \ 2 + 54x \) + 9x \ 2 \ 3 \ 1728 \ (-0.25x \ 2 \) \ dx + \delta \ 18x \ dx \ EI \ \delta 0 \ 0 \ 0 \ 6 \ 6 = -0.0833 \ x \ 3 + (1.67x \ 3 \ -17.25x \ 2 + 54x \) + 9x \ 2 \ 3 \ 1728 \ (-0.25x \ 2 \) \ dx + \delta \ 18x \ dx \ EI \ \delta 0 \ 0 \ 0 \ 6 \ 6 = -0.0833 \ x \ 3 + (1.67x \ 3 \ -17.25x \ 2 \ + 54x \) + 9x \ 2 \ 3 \ 1728 \ (-0.25x \ 2 \) \ dx + \delta \ 18x \ dx \ EI \ \delta 0 \ 0 \ 0 \ 6 \ 6 = -0.0833 \ x \ 3 \ + (1.67x \ 3 \ -17.25x \ 2 \ + 54x \) + 9x \ 2 \ 3 \ 1728 \ (-0.25x \ 2 \) \ dx \ + \delta \ 18x \ dx \ EI \ \delta 0 \ 0 \ 0 \ 6 \ 6 = -0.0833 \ x \ 3 \ + (1.67x \ 3 \ -17.25x \ 2 \ + 54x \) + 9x \ 2 \ 3 \ 1728 \ (-0.25x \ 2 \) \ dx \ + \delta \ 18x \ dx \ EI \ \delta 0 \ 0 \ 0 \ 6 \ 6 \ = -0.0833 \ x \ 3 \ + (1.67x \ 3 \ -17.25x \ 2 \) \ dx \ + \delta \ 18x \ dx \ EI \ \delta 0 \ 0 \ 0 \ 6 \ 6 \ = -0.0833 \ x \ 3 \ + (1.67x \ 3 \ -17.25x \ 2 \ + 54x \) \ + 9x \ 2 \ 3 \ 1728 \ (-0.25x \ 2 \) \ dx \ + \delta \ 18x \ dx \ EI \ \delta 0 \ 0 \ 0 \ 6 \ 6 \ = -0.0833 \ x \ 3 \ + (1.67x \ 3 \ -17.25x \ 2 \ + 54x \) \ + 9x \ 2 \ 3 \ 1728 \ (-1.67x \ -17.25x \ -17.25$ 3¢¢ δAH = ? Practical application: A one-lane bridge consists of a 10-in.-thick, 16-ft-wide reinforced concrete slab supported on two steel girders spaced 10 ft apart. 5m 36 kips P4.6. Determine the forces in all bars of the trusses. , area of beam = 16 in. Details of the Barracks Bridge over the Mississippi River-a 909 ft-long, 280 ft high arch-continuous beam system uses a 12'-5" deep continuous I-shaped plate girder supported by 17 hangers. 8' 12' P8.42 20 kips 24 in 12 in 1 2 3 4 157.5 5 180 6 7 172.5 150 157.5 112.5 8 112.5 6 5.25 3.75 3.75 2.25 0.75 MQ (kip-ft) 0.75 Segment Depth (in.) 0.5IG (in.4) 1 2 3 4 5 6 7 8 14 18 22 24 24 22 18 14 1372 2916 5324 6912 6912 5324 2916 1372 MQ (kip-ft) 0.75 2.25 3.75 5.25 5.25 3.75 5.25 5.25 3.75 5.25 5.25 3.75 2.25 0.75 MP (kips-ft) 22.5 67.5 112.5 157.5 112.5 157.5 112.5 37.5 Σ MP MQ(144)/0.5IG (kip2/in.2) 1.771 7.500 11.410 17.227 18.867 15.975 12.500 2.952 MQ M P I 88.20 Deflection at mid-span: δC M Q M P Δx n EI 88.20(3)(12) 3000 δC 1.06 in. While the axial force, and the direct stresses it produces, will shorten the arch, the changes in length of the arch axis and the vertical deflection of points along the arch axis will typically be small if the magnitude. 6-30 Copyright © 2018 McGraw-Hill Education. Q R T S 12' P O M N 12' L I H J K G F B C 12' E 15' A 20' 24' D 20' P13.15 (a) Estimate Axial Load and Moment in Column AH. (b) If bars AB 3 and BD are fabricated 4 in. 3/8 in. C 60 kips 9' B D 9' A G 12' 24' P8.12 $\Sigma Q \cdot \delta P = \Sigma F Q$ FP E F L AE é ù 1 ú · é(+1)(106.7)(12 ¢ + 24 ¢)12% 1k · $\delta Gy = \hat{e} \hat{e} 5$ in 2 (29000) ú ë ë û + (0.33) (106.7k)(12 '12 %) + (-1.25)(-133.3k)(15 '12 %) + (-0.417)(-133.3k)(15 '12 %) + (-0.33k)(-80 k)(24 '12 %))Analyze the frame in Figure P10.31. A C B 5m 5m P9.3 Select RC as the Redundant $\triangle CO$ Use Moment-Area Method æ 30 ö 1125 $\triangle CO = tC/A = 5$ cc \div (7.5) = () cè EI ϕ \div ci (7.5) = () cè EI ϕ \div ci (7.5) = () ce EI ϕ ϕ \div ce EI ϕ ϕ \bullet ce EI ϕ \bullet ce EI EI 3EI C $\Sigma Fy = 0$: RA + RC = 0 \ RC = -3.375 kN () \ RA = 3.375 kN () \ RA = 0 : M A - 30 - 10 RC = 0 (+) \ M A = -3.75 kN·m () Shear Moment 9-4 Copyright © 2018 McGraw-Hill Education. (b) Re-position the moving load symmetrically on the span and compute the maximum moment and the maximum deflection. EI is constant for 6 4 all members, E = 200 GPa, I = 1800 × 10 mm. Classify the structures in Figure P5.54. 3m P4.16., and AAD = 8 in. Also evaluate the forces in bars a, b, c, and d. EQs EI is Constant Joint B Σ M B = 0; M BA + M BC = 0 (1) Joint C Σ MC = 0; M CB + M CD = 0 (2) Shear EQ Σ Fx = 0: M AB + M BA MCD + M DC + 8 6 + 60 = 0 (3) æ EI ö EI 6 cc θ B - Ψ AB \div \div + (2 θ B $+ \theta C$) = 0 (1a) 8 èc 2 ø 5 $\Delta \Psi CD \Delta \Delta 4$ = 6 = ΨAB = , $\Psi CD = \Delta 3 8 6 \Psi AB 8 4$ and $\Psi CD = \Psi AB 3 2 EI (\theta B - 3\Psi AB) 8 2 EI M BA = (2\theta B - 3\Psi AB) 8 2 EI M BA = (2\theta B - 3\Psi AB) 8 2 EI M BC = (2\theta B + \theta C) 10 2 EI (2\theta C - 4\Psi AB) MCD = 6 2 EI (\theta C - 4\Psi AB) MCD = 6 2 EI (\theta C - 4\Psi AB) MCD = 6 2 EI (\theta C - 4\Psi AB) MCD = 6 2 EI (\theta C - 4\Psi AB) MCD = 6 2 EI (\theta C - 4\Psi AB) MCD = 6 M AB = -93.72 kN · m EI EI (2 \theta C + \theta B) + (2 \theta C - 4\Psi AB) = 0 (2a) (EQ12.12) 5 3 2EI 2$ EI ($\theta B - 3\Psi AB$) + ($2\theta B - 3\Psi AB$) 8 8 EI EI ($2\theta C - 4\Psi AB$) 60 3 3 C + + =0 6 EI θ 6 12 8 60 $\theta - \Psi + C - \Psi AB = (3a)$ 64 B 64 AB 6 18 EI = -71.29 kN · m = -147.58 kN · C E D F 10 kN 20 kN A J I 20 kN H G 20 kN 5 @ 5 m = 25 m P4.5 + Σ M A = 0; 20 10 + 20 15 + 20 25 + 10 5 - RGY 20 = 0 RGY = 52.5 kN + Σ Fx = 0; AX = 10 kN \neg 4-6 Copyright © 2018 McGraw-Hill Education. P11.26. Continued Member Section Force Member Label 1 2 3 4 5 6 7 Axia Shear Moment (k) (k) (k-ft) 1 -6.667 4.893 0 2 -6.667 4.893 0 2 -6.667 4.893 12.231 3 -6.667 4.893 24.463 4 -6.667 4.893 24.463 4 8.011 -9.785 24.463 4 8.011 -9.785 0 1 6.667 5.107 0 2 6.667 5.107 12.769 3 6.677 5.107 12.769 3 6.677 5.107 12.769 3 6.677 5.107 12.769 3 6.677 5.107 12.769 3 6.677 5.107 12.769 3 6.677 5.107 1 1 -8.656 -10.215 51.075 2 -8.656 -10.215 38.306 3 -8.656 -10.215 25.537 4 -8.656 -10.215 12.769 5 -8.656 -10.215 0 1 -20.757 0 0 2 -20.757 0 0 4 -20.757 0 0 5 -20.757 0 0 5 -20.757 0 0 5 -20.757 0 0 5 21.669 0 0 5 21.669 0 0 5 21.669 0 0 1 1.774 0 0 2 1.774 0 0 3 1.774 0 0 4 1.774 0 0 5 1.774 McGraw-Hill Education. 9' 60 kips B D 4.5' 12' C P9.39 $\Delta A = 0 \Delta AD + \delta AA X A = 0$ (1) Compute ΔAO (Use Moment-Area) -48600 270 (12)(6 + 9) = EI EI Compute δAA (Use Table A.3) $\Delta AD = t AC = PL3$ 1(21)3 3087 = 3EI 3EI EI Substitute (2) & (3) into (1) -48600 3087 é ù X = 0 X A = 15.74 k + EI EI ë A û $\delta AA = (2)$ (3) 9-44 Copyright © 2018 McGraw-Hill Education

+ ΣE x = 0 = 24 k - FFEX \ FFEX = 24 k T Complete by Method of Joints See Top Figure 4-43 Copyright © 2018 McGraw-Hill Education.

For each beam, draw the shear and moment curves label the maximum values of shear and moment locate points of inflection, and sketch the deflected shape. A B C D E F hinge 4' 8' 4' 4' P12.5 Reactions produced by W = 1.2 kips/ft æ1 3ö RB = WA = 1.2 kips/ft çç 12 ¢ $\div \div = 10.8$ kips $c^2 2 \# 61 \# 30$ $u^2 1 \# 10$ RD = 1.2 $c^2 + 4 + 4 + 1.2$ $c^2 + 1.$

w = 12 kips/ft P5.30. Analyze the frame in Figure P10.26.

P14.9. The cantilever beam in Figure P14.9 is connected to a bar at joint 2 by a pin. PL 2 EI Reaction at end of beam due to mid-span point load: $-\Delta B + X B \delta BB = 0 3 10$ kips 3 5PL PL + XB = 0 48EI 3EI 15P XB = 48 - 1 in. (b) Repeat the computations if, in addition to the applied loads, support B settles 0.5 in. Cy = 14 kips C P9.24. The rocker at A is equivalent to a roller. 4m 4m 8m H 4m C B A 4m I A E F G 24 kN 30 kN P4.21 + $\Sigma M A = 0$; 24 $^{\prime}8 + 30$ $^{\prime}16 - RD 24 = 0$ RD = 28 kN Zero bars: BH, CI, CE, CF Analyse by method of joints 4-22 Copyright © 2018 McGraw-Hill Education. Consider the beam shown in Figure P12.46 without the 80 kN load.

(a) (b) (c) (d) (f) (e) (g) P4.2 (a) (b) (c) (d) b = 17 r = 5 n = 10 b = 21 r = 3 n = 13 (e) (b + r = 22) > (2n = 20) (b + r = 24) > (2n = 26) 2 deg. Set up the equilibrium equations required to analyze the frame in Figure P10.32 by the slope deflection method. B 360 lb/ft 240 lb/ft 36' A C elastomeric pad 30' 15' 15' P3.23 + $\Sigma Fx = 0$; Ax -12.96 k - 8.64 k = 0 Ax = 21.6 kips + $\Sigma M A = 0$; -12.96 k (18) - 8.64(18) - C y 90 ¢ + 10.8(67.5) - 16.2(22.5) = 0 C y = -0.27 kips $\Sigma Fy = 0$; Ax -12.96 k - 8.64 k = 0 Ay = -5.13 kips 3-24 Copyright © 2018 McGraw-Hill Education.

 $8 \text{ kN/m A B 6m C 6m D 4m P3.15 FBD "AB" + <math>\Sigma M b = 0$; -24 kN (3m) - 6 kN (2m) + Ay (6m) = 0 Ay = 14 \text{ kN FBD "ABCD" + }\Sigma M d = 0; 14(16m) - 48(10m) - 24(8m) + C y (4m) = 0 C y = 112 \text{ kN Entire Structure }\Sigma Fy = 0; + 14 kN - 48kN - 24 kN - 40 kN + E y + 112 kN = 0 E y = 14 kN + $\Sigma M b = 0$; 14 kN (20 m) - 48(14) - 24(12) + 112(8) - 40(4) + Me = 0 Me = 56 \text{ kN} \cdot \text{m} 3-16 \text{ Copyright } \odot 2018 \text{ McGraw-Hill Education.} Assume all member properties are the same except the areas 2 of AE, ED, BD, and BC are 8 in.

(b) Write the equations for shear and moment in column AB. for practical applications. (a) *Note: If you wish to compute the forces or deflection at a particular point of a member, designate the point as a joint. A B 9' k = 5 kips/in. C D E 15 kN 20 kN 4m F A G 40 kN 3m 30 kN 3m 3m P4.3 æ4ö Node C : $\Sigma Fy = 50 - CF \ cc \ \div \div = 0 \ ce 5 \ o \ \Sigma M \ A = 4 Bx + 4(15) + 9(20) + 6(30) + 3(40) = 0 Bx = -135 kN \neg CF = 62.5 kN (T) \ \Sigma Fx = Bx + 15 + Ax = 0 \ æ \ 3ö \ \Sigma Fx = -67.5 + 62.5 \ cc \ \div \div + CD = 0 \ ce 5 \ o \ Ax = 120 kN \ \Sigma Fy = Ay - 40 - 30 - 20 = 0 \ CD = 30 kN (T) \ Ay = 90 kN \ Node \ D: \ \Sigma Fy = 90 + AB = 0 \ DF = 0 \ kN \ AB = -90 \ kN \ (C) \ \Sigma Fx = -30 + DE = 0 \ \Sigma Fx = 120 - AG = 0 \ DE = 30 \ kN \ (T) \ AG = -120 \ kN \ (C) \ \varpi 4ö \ \varpi 4ö \ Node \ F: \ \Sigma Fy = -30 + 62.5 \ cc \ \div \div + 0 + FE \ cc \ \div \div = 0 \ ce \ 5 \ o \ E = -25 \ kN \ (T) \ \varpi 3ö \ \Sigma Fx = -135 + 112.5 \ cc \ \div \div \div + 0 + FE \ cc \ \div \div \div = 0 \ ce \ 5 \ o \ E = -25 \ kN \ (T) \ \varpi 3ö \ \Sigma Fx = -135 + 112.5 \ cc \ \div \div \div + 0 + FE \ cc \ \div \div \div = 0 \ ce \ 5 \ o \ E = -25 \ kN \ (T) \ \varpi 3ö \ \Sigma Fx = -135 + 112.5 \ cc \ \div \div \div + 0 + FE \ cc \ \div \div \div + 0 + FE \ cc \ \div \div \div + 0 + FE \ cc \ \div \div \div = 0 \ ce \ 5 \ o \ E = -25 \ kN \ (T) \ \varpi 3ö \ \Sigma Fx = -135 + 112.5 \ cc \ \div \div \div + 0 + FE \ cc \ \div \div \div + 0 + FE \ cc \ \div \div \div + 0 + FE \ cc \ \div \div \div + 0 + FE \ cc \ \div \div \div + 0 + FE \ cc \ \div \div \div + 0 + FE \ cc \ \div \div \div + 0 + FE \ cc \ \div \div \div + 0 + FE \ cc \ \div \div \div + 0 + FE \ cc \ \div \div \div + 0 + FE \ cc \ \div \div \div + 0 + FE \ cc \ \div \div \div + 0 + FE \ cc \ \bullet \div \div + 0 + FE \ cc \ \div \div \div + 0 + FE \ cc \ \bullet \div \div + 0 + FE \ cc \ \bullet \div \div + 0 + FE \ cc \ \bullet \div \div \div + 0 + FE \ cc \ \bullet \div \div \div + 0 + FE \ cc \ \bullet \div \div \div + 0 + FE \ cc \ \bullet \div \div \div + 0 + FE \ cc \ \bullet \div \div \div + 0 + FE \ cc \ \bullet \div \div \div + 0 + FE \ cc \ \bullet \div \div \div + 0 + FE \ cc \ \bullet \div \div \div + 0 + FE \ cc \ \bullet \div \div \div + 0 + FE \ cc \ \bullet \div \div \div + 0 + FE \ cc \ \bullet \div \div \div + 0 + FE \ cc \ \bullet \div \div \div + 0 + FE \ cc \ \bullet \div \div \div + 0 + FE \ cc \ \bullet \div \div \div + 0 + FE \ cc \ \bullet \div \div \div + 0 + FE \ cc \ \bullet \div \div \div + 0 + FE \ cc \ \bullet \div \div \div + 0 + FE \ cc \ \bullet \div \div \div \div \div + 0 + FE \ cc \ \bullet \div \div \div$

= 0 cè 5 ø AB BG GC BG Ax A G AG 40 kN Ay E CD BC GC = -50 kN (C) GF = -52.5 kN (C) D DE CF CF DF F GF FE FE 30 kN 4-4 Copyright © 2018 McGraw-Hill Education.

The ASCE standard permits linear interpolation for the value of the inclined angle of roof. If member AB in Figure P10.25 is fabricated 43 in. 4' 1 kip/ft P3.12. 20 kips 9 kips/ft A 9 kips/ft B 10' 6' 6' P11.3 wL2 -9 (10) = = -75 k · ft 12 12 = 75 k · ft 12 12 = 75 k · ft 2 FEM AB = FEM BA 20 (12) PL == -30 k · ft 8 8 = 30 k · ft FEM BC = FEMCB D C FEMCD = -75 k · ft 12 12 = 75 k

Lanning, and Anne M. Compute the reactions and draw the shear and moment curves for the beam in Figure P9.20. 8 kips 12 kips B 12 kips B 12 kips B 12 kips W = 2.4 kips/ft C D F 10' 10' 32' 10' P11.10 Consider Symmetry: Analyze 3 I 4 (20) 1 I KCD = K DC = 2 (32) $\Sigma K S = 0.053125 I K BC = K DE = 1$ structure 2 Distribution factors at joint C and D KCB = 0.706 KCD = 0.294 PL = 30 k · ft 8 2 wL = 204.8¢¢ 12 FEM BC = FEMCD = FEMCD = FEMCD = FEMCD = FEMCD = FEMCD = C.2018 McGraw-Hill Education. 11-42 Copyright © 2018 McGraw-Hill Education.

Compute the deflection at midspan of the 6 4 beam in Figure P8.24. Given: AE is 2 constant for all bars, A = 10 in. Continued (b) Seismic Loads by Equivalent Lateral Force Procedure Given: W 90 psf Floor & Roof; SDS 0.27g, SD1 0.06g; R = 8, I 1.5 Base Shear Vbase = SD1 W T (R/I) W Total Building Dead Load Where Wroof = 90 psf (100 ¢)2 = 900 k W2nd = 90 psf (100 ¢)2 = 900 k Wtotal = 1800 k T = CT hnx = 0.342 sec.

B C 16' A D 800 lb/ft 2' 2' 14' P10.13 Member end moments, from symmetry: $\theta C = -\theta B$: Fixed-End Moments: -0.8(16) = -10.24 kip-ft 2 2 - 0.8(16) = -3.27 kip-ft $12 2 FEM AB = -FEMCD FEM BC = -FEMCB Equilibrium of joint B: M BA + M BC = 0 2 EI 2 EI 0 = (2\theta B + 0) + 6.83 + (2\theta B - \theta B) - 3.27$ 16 14 -9.06 $\theta B = EI VBC MBC B 0.2$ kips/ft MBC CD M AB = -11.37 kip-ft M BA = 4.57 kip-ft M BC = -4.56 kip-

The location of the P.I. in span CD is 3.9' from D rather than 5' assumed. Member EB is straight $\therefore \theta B = \theta E = 0 \therefore$ Joints B and E can treated as fixed supports. 6' 9 kips P12.53. Select RC as the Redundant $æ 2 \times \ddot{o} dx @ x \ddot{$

P12.39. P6.33. 30 kips P4.33. Compute the support reactions and the maximum value of w if the allowable tension force in the cable in Figure P6.10 is 200 kN. Notice that sidesway is possible because the load is unsymmetric. 10 kips P3.1. Determine the reactions for the structure. Draw the shear and moment curves for each member of the frame and draw the deflected shape. B C D E RA 8' L = 32' L = 32' P12.52 12-56 Copyright © 2018 McGraw-Hill Education. B A K = 40 kN/m C 10 m P9.21 Δ BO = () wL4 4(10)4 = = 0.4167 m 8EI 8(200)(60) PL3 1(10)3 = = 0.0278 m () 3EI 3(200)(60) 1 = 0.025 m () δ S = 40 Compatibility Equation δ BB = - Δ BO + δ BB RB = -0.025 RB - 0.1 Select RB as the Redundant. , area of bar 2 = 3 in. Label maximum values of shear and moment. 60°F Coef. Estimate the deflection at midspan of the truss in Figure P13.12, treating it as a beam of constant cross section.

(b) If support B settles 1 in. Axial forces exist in horis and vertical members. wind 16' 16' 80' 48' (a) qhGCp qhGCp θ h qzGCp qhGCp Section (b) P2.13 TABLE P2.13 Roof Pressure Coefficient Cp * θ defined in Figure P2.13 Windward Leeward Angle θ 10 15 20 25 30 35 45 \geq 60 Cp $-0.9 - 0.7 - 0.4 - 0.3 - 0.2 - 0.2 0.0 0.0 0.2 0.2 0.3 0.4 0.010* 10 15 - 0.5 - 0.5 - 0.5 \geq 20 - 0.6$ Consider Positive Windward Pressure on Roof, i.e. left side. 4-47 Copyright © 2018 McGraw-Hill Education.

 $+ \Sigma M A = 0; 5k (15¢) - M BA - 45ft + k = 0 M BA = 30 ft + k \Sigma M A = 0; 10 k (15¢) - Dy (30) = 0; Dy = 5k + \Sigma Fy = 0 Ay = 5k 13-39 Copyright © 2018 McGraw-Hill Education. 4 kN 2 kN + M A 3m B 3m C 4m D 6m P3.7 Freebody of Member "AB": + \Sigma Fx = 0; + RAx = 0 \Sigma M b = 0; M A - 4 kN 3m = 0 x M A = 12 kN + m + \Sigma Fy = 0; -4 kN + Fb = 0 Fb = 4 kN Freebody of Member "BD": + \Sigma M d = 0; -4 kN (10 m) + Rc 6 m - 2 kN + m = 0 Rc = + \Sigma Fy = 0; 42 = 7 kN 6 - 4 kN + Rc - Rd = 0 - 4 kN + 7kN = Rd 3 kN = Rd Entire Structure: 3-8 Copyright © 2018 McGraw-Hill Education. <math>e^{2} 2 S = sin f = 1, C = cos f = 0 e^{8.33'10-4} SC (-N + P) - QS ù \dot{u} e^{2} e^{0} 0 NS 2 + PC 2 - QC \dot{u} = 125,000 e^{2} \dot{u} e^{2} e^{0} e^{2} (kF) = 125,000 e^{0} 8.33'10-4 e^{0} 0 - 0.05 e^{0} S = sin f = 0, -0.05 \dot{u} \dot{u} \dot{u} 0 C = cos f = 1 0 \dot{u} \dot{u} \dot{u} - 0.05 \dot{u} \dot{u} e^{0} e^{-6250} - 6250 1,000,000 \dot{u} \hat{u} 16-13 Copyright © 2018 McGraw-Hill Education. 10' P12.14. w A B L P7.1 Analysis by Double Integration w(L - x) = M 2 2 d y M w = = (L2 - 2 xL + x 2) 2 EI 2 EI dx d2 y 2 EI 2 = -IL2 + 2wxL - wx 2 dx 0 dy x 2 wx 3 = -wL2 x + 2wL + C1¢ = 0 2 EI dx 8 3 dy = 0 \ C1 = 0 At x = 0, dx 2 2 0 wL x wLx 3 wx 4 + C2¢ = 0 2 EI y = 2 3 12 At x = 0, y = 0 \ C2 = 0 (1) (2) Compute DB; Set x = L in Eq(2) 1 æc wL4 wL4 wL4 \ddot{v} + \dot{v} +$

Factors $AB = 0.35 \cdot 1099 I Distr. F E D 24 kips C 15' B A 24 kips 3 @ 10' = 30' P4.10 \Sigma M A = 15Gx + 10(24) + 30(24) = 0 Node F: <math>\Sigma Fx = -64 + 48 - AF \cos(56.31) = 0 AF = -28.84 kips (C) \Sigma Fy = 28.84 sin (56.31) + BF = 0 Gx = -64 kips ¬ \Sigma Fy = Ay - 24 - 24 = 0 BF = 24 kips (T) Ay = 48 kips F Gx EF GF \Sigma Fx = Ax - 64 = 0 ED CD 24 kips FC AG D EC AF Ax = 64 kips ~ \Sigma Fy = -24 - CD sin (26.57) = 0 AfF = -44 kips (T) Ay = 48 kips F Gx EF GF \Sigma Fx = Ax - 64 = 0 ED CD 24 kips FC AG D EC AF Ax = 64 kips ~ \Sigma Fy = -24 - CD sin (26.57) = 0 AfF = -44 kips (T) Ay = 48 kips (C) \Sigma Fy = -24 - CD sin (26.57) = 0 AfF = -44 kips (T) Ay = 48 kips (C) \Sigma Fy = -24 - CD sin (26.57) = 0 AfF = -44 kips (T) Ay = 48 kips (C) \Sigma Fy = -24 - CD sin (26.57) = 0 AfF = -44 kips (T) Ay = 48 kips (C) \Sigma Fy = -24 - CD sin (26.57) = 0 AfF = -44 kips (T) Ay = 48 kips (T) Ay = 48 kips F Gx EF GF \Sigma Fx = Ax - 64 = 0 ED CD 24 kips FC AG D EC AF Ax = 64 kips (T) Ay = 48 kips$

Using the influence lines, determine the reactions at supports A and F if the dead load of the floor system can be approximated by a uniform load of 10 kN/m. z V(x) A P5.13. The area of all bars = 4 in. EQ at Joint B + $\Sigma M B = 0$; M BC + M BA + 16.8 = 0 $\Sigma M B = 0$; VBC - 67.2 + 37.1 = 0 (2) Substitute Eq's (1) into (2) 4 E (4 I) 4 EI θB -134.4 + $\theta + 50.4 = 0.24 12 B 84 EI \theta B = 84 \theta B = (3) EI Substitute (3) into Eq's (1) 4 E (4 I) é 84 ù ê ú -134.4 = -78.4 k · ft 24 ê EI úû 4 EI æç 84 ö ÷ = 28 ft · kips ç ÷ 12 çè EI ÷ø VBC = 30.1 kips Axial Force in BA FAB = 16.8 + 30.1 FAB = 46.9 kips Shear in Column AB + <math>\Sigma M A = 0.28 + 14 - V12 = 0 V = 3.5$ kips M BC = M BA 2 E (4 I) æç 84 ÷ö ç ÷ + 134.4 = 162.4 k · ft 24 çè EI ÷ø 2 EI 84 = 14 kip · ft 12 EI MCB = M AB 10-6 Copyright © 2018 McGraw-Hill Education. wind 16' 16' 80' 48' (a) qhGCp qh

All dimensions are measured from the centerlines of members. 80 kN A B 8m C 4m D 4m E 4m P12.46 12-50 Copyright © 2018 McGraw-Hill Education. of Temp.

E P3.31. B C moment P5.13 Constant negative shear results in a linear moment diagram with negative slope.

and displaces 1/2 in. The bolted web connection at B may be assumed to act as a hinge. Origins for each member are shown.

Given: area of bar 2 2 BD = 4 in. C A B 24' 20' P13.1 Assume P.I. 0.25 L to Left of Support $B + \Sigma M P$. P P12.24. "ABC" M BA = 75ft ·k Determine Bottom Chord Force FBL: F ab F (5¢)(15¢) M B = 75 ft ·k = BL = 20 k + \Sigma Fx = 0; - VCB 20 k - 5k = 0 VCB = 15k ¬ Truss Bar Forces by Method of Joints: Joint "C" Joint "L" + $\Sigma Fy = 0; - 6.67k + CLy = 0$ CLy = 6.67k C CLx = 6.67k ¬ C FCL = 9.43k C + $\Sigma Fx = 0$ 10 k + 15k - 6.67k + FCD = 0 FCD = 18.3k C 13-42 Copyright © 2018 McGraw-Hill Education. Draw the influence lines for bar forces in members CD, EL, and ML of the truss shown in Figure P12.34. 4m P9.31. File loading please wait... and an effective modulus of elasticity of 26,000 2 kips/in. D B C D' 10' 20' A 15' 30' P8.27 WQ = UQ k - 2 (1.5k) + 1k \delta BH = 0.3 \delta BH = 1k WQ = UQ k - 1 (1.5k) + 1k \delta BY = 0.2 k 3 \delta BY = 4 WQ = UQ 1k (1.5k) + 1 ft k \Delta \theta BC = 0.30 æ1ö 1.5 çç ÷÷ çè 30 ø \Delta \theta BC = 1.2 k \Delta \theta BC = 0.004167 Radians - 8-33 Copyright © 2018 McGraw-Hill Education. + $\Sigma Fx = 0; 15k - FAJ = 0$ FAJ = 15k compr.

 $P = 60 \text{ kips } P9.45. 9 \text{ kips/ft } P = 15 \text{ kips } 25 \text{ kips } 34 \text{ E B D A C 4}' 8' 8' 4' P3.11 + \Sigma M A = 0; 20(4 \text{ c}) + 54(16 \text{ c}) + 15(24 \text{ c}) - 16C \text{ y} = 0 \text{ C y} = 81.5\text{k} + \Sigma Fx = 0; 15 - Ax = 0 \text{ Ax} = 15\text{k} - \Sigma Fy = 0; + Ay - 20 - 54 + 81.5 - 15 = 0 \text{ Ay} = 7.5\text{k}$ Isolate Member Be + $\Sigma Mb = 0; 54(8) + 15(16) - \text{Dy}(12) = 0 \text{ Dy} = 56 \text{ k}$ $\Sigma Fy = 0; + Dy + 56 - 54 - 15 = 0 \text{ Dy} = 13\text{k} + \Sigma Fy = 0; Dx = 0 \text{ 3} - 12 \text{ Copyright } \mathbb{C}$ 2018 McGraw-Hill Education. See, for example, page 240 of the textbook; the highway bridge is composed of a truss composed of diagonal members and the girders of the roadway. = 0.45 16 80 3 I 6I K BC = 2 20 80 \text{ D.F}. 30 kips P11.7. Analyze each structure by moment distribution. G P4.24.

w = 2.4 kips/ft B C A D 18' 48' 48' P13.8 FEM = wL2 PL 2.4(96)2 10(96) + = + = 1963 ft · k 12 8 12 8 Girder end moments: 0.25(FEM) = 490.8 ft · k $\Sigma Fy = 0$; V1 + V2 -10 - 2.4(96) = 0; by symmetry V1 + V2 = 120.2 k + $\Sigma M A = 0$ 18VA - 490.8 = 0 VB = 27.27k 13-12 Copyright © 2018 McGraw-Hill Education. The load moves along BH of the truss. w = 0.4 kip/ft A variable B 24' P12.42 Envelope for Max. In practice, the deflection of the floor system due to the dead weight of the bridge can be eliminated by shortening the length of the vertical cables that support B is the required area of each power (joints 12 to 20). For the parabolic arch in Figure P12.26. 4 kips/ft 4'' B E D 6' 6' 3 kips/

A - 6 IAB = ICD = 1440 in. In the gabled roof structure shown in Figure P2.13, determine the sloped roof snow load Ps. The building is heated and is located in a windy area in Boston. Continued N D = 0.451C x + 0.893C y - 0.893P VD = 0.451P M D = C x (100) - C y (36.98) + P(x2) FBD " DC " P@ y @A @B a' x Cy = 0.0057 Pa' Cx = 0.0033 Pa' VD ND MD -150' -87.5' 0 0 0 0 0 -125' -79.88' 7.62' 0.0434P 0.025P -0.05P 0.033P -0.895P -100' -71.44' 16.06' 0.0915P 0.053P -0.167P 0.01P 3.1P -50' -50.52' 36.98' 0.211P 0.122P -0.244P 0.014P 4.4P -25' -35.72' 51.78' 0.295P 0.171P -0.34P 0.020P 6.19P -12. w = 2 kips/ft P5.43. and 2 E = 30,000 kips/in. Continued (b) Analysis by Cantilever Method Assume area of interior columns is twice that of the exterior columns 13 V A = 4.38 kN 8 1 F2 = F4 = VI 2 A = 5.39 kN (a) F1 = F5 = ycc + = 61.3 kN 8 cc 74.25 + Ø Note: Computations contain a round off error which results in a horizontal shear at the base 5% greater than the correct value. 40' P6.16 Continued (a) Shear (kips) Moment (kip-ft) Axial Force (kips) Deflection (Max. + + = 2 EI EI EI AE Since MQ,EH and MQ,BH are equal and no axial loads are developed in BCED due to applied loading, $\delta BH = \delta EH : \delta EH + 0.061$ in. C 3m 100 kN 3m A 4m P10.17 E = 200 GPa, I = 25 × 106 mm4 Joint B M BC + M BA = 0 θB ö EI 2 EI $\alpha c c 2 \theta - + + + (2 \theta B + 0.005) = -75 2 cc^{2} + 0 + (2 \theta B + 0.005) = -75 2 cc^{2} + 0 + (2 \theta B + 0.005) = -75 2 cc^{2} + 0 + (2 \theta B + 0.005) = -75 2 cc^{2} + 0 + (2 \theta B + 0.005) = -75 2 cc^{2} + 0 + (2 \theta B + 0.005) = -75 2 cc^{2} + 0 + (2 \theta B + 0.005) = -75 2 cc^{2} + 0 + (2 \theta B + 0.005) = -75 2 cc^{2} + 0 + (2 \theta B + 0.005) = -75 2 cc^{2} + 0 + (2 \theta B + 0.005) = -75 2 cc^{2} + 0 + (2 \theta B + 0.005) = -75 2 cc^{2} + 0 + (2 \theta B + 0.005) = -75 2 cc^{2} + 0 + 0 + (2 \theta B + 0.005) = -75 2 cc^{2} + 0 + 0 + (2 \theta B + 0.005) = -75 2 cc^{2} + 0 + (2 \theta B + 0.005) = -75 2 cc^{2$

ft \hat{e} 2 \hat{u} 3 = 115.94 k Check uning EQ (b.) \hat{w} 10 w \hat{c} \hat{c} \hat{v} \hat{v} \hat{c} \hat{v} \hat{c} \hat{v} \hat{v} \hat{v} \hat{c} \hat{v} \hat{v}

Also determine the horizontal 4 displacement of joint B. (b) Determine the cable sag at point B. (c) Determine the cable sag at point B. (c) Determine the cable sag at point C 62' P3.27 + Σ MA = 0; + Σ Fx = 0; 78¢ + 8 x 28¢ - Rdy 62 ¢ = 0 2 Rdy = 121.37 kips 187.2 k Rdy + Rdy -187.2 k = 0 RAy = 187.2 -121.37 = 65.83 K - RAx + 8 = 0 RAy = 121.37 kips 187.2 k = 0 RAy = 121.37 kips 187.2 kips 18 2018 McGraw-Hill Education. I 3 I 3 I = 30440120 I 3 I 5 I = = 18424120 8 5 8 Pab 2 1(12) 6 2 = = -2.88 L2 30 2 Pba 2 1(12) 6 2 = = -2.88 L2 40 Pba 2 1(12) 6 2 = = $\Sigma M B = 0 = RA 30 - 1'12 + 3.6 RA = 0.28$ kips 12-61 Copyright © 2018 McGraw-Hill Education. 20 kips P4.35. For the girder in Figure P12.17, draw the influence lines for the reaction at A, the moment at point C, and the shear between points B and C in girder AE. Following the ASCE standard, the wind pressure along the height on the windward side has been established as shown in Figure P2.10(c). 6" 6" 48" 24" 8" 12" 18" Section P2.1 Compute the weight/ft. P8.4. For the truss in Figure P8.4, compute (a) the vertical displacement of joint C, (b) the horizontal displacement of joint C, (b) the horizontal displacement of joint C, and (c) the vertical displacement of joint C, (b) the horizontal displacement of joint C, and (c) the horizontal displacement of joint C, and (c) the horizontal displacement of joint C, (b) the horizontal displacement of joint C, and (c) the horizontal displacement of joint C, and (c) the horizontal displacement of joint C, (b) the horizontal displacement of joint C, and (c) the horizontal displacement of joi end A for the beam in Figure P10.8, compute the required weight W that needs to be placed at midspan of CD. The 16-ft-long stringers frame into the floor beams, which in turn transfer the live and dead loads to the panel points of each truss. Computer Analysis of Truss Case 1 Member Axial Force (kips) Reaction (kips) Joint X 1 0 2 - 45 (C) 3 45 (T) 4 Y - 75 (C) 6 75 (T) Displacement (in.) X Y 1 - 45 - 60 1 0 0 3 - 45 60 2 .702 0 3 0 0 4 .702 0 5 .647 0 0 5 Joint 4-55 Copyright © 2018 McGraw-Hill Education. I G F 15' H L K 15' A B 30 kips C 60 kips D Education. 5' -25.26' 62.24' 0.355P 0.2854P -0.41P 0.023P 7.41P 0 0 87.5' 0.5P 0.292P -0.579P 0.035P 10.78P -0.667P 0.107P 3.24P Cx = 0.0033 Pa' -0.0067Px @D -12.5' 25.26' 112.76' 0.643P 0.2054P -25' 35.72' 123.22' 0.702P 0.171P -0.704P .0164P 8.86P -50' 50.52' 133.02' 0.787P 0.122P -0.758P 0.246P 16.9Px = 175 - a' -75' 61.87' 149.37' 0.851P 0.085P -0.095P 0.143P 2.66P -100' 71.44' 158.94' 0.906P 0.053P -0.06P 0.09P Appr. Analyze the frame in Figure P10.19. (b) Using the influence line for moment at point B. Draw the shear and moment curves for each member of the beam in Figure P5.30. then $\Delta CS = -0.5$ in. Continued O-System æ $Wx 2 \ddot{o} \div dx \div 1$ $\theta C = \dot{o} 1 ccc Px + c\dot{e} 2 \div \theta EIL 0 \acute{e} Px 2 Wx 3 \dot{u} \acute{e} \acute{u} + \dot{e} 2 6 \acute{u} 0 \ddot{e} PL2 wL3 + \theta C = 2 EI 6 EI 6(12) 2(12) 3720 + = \theta C = EI 2 EI 6 EI L 1 \theta C = EI 2 EI 6 EI 2 EI 6 EI L 1 \theta C = EI 2 EI 6 EI L 1 \theta C = EI 2 EI 6 EI 2 EI 6 EI L 1 \theta C = EI 2 EI 6 EI L 1 \theta C = EI 2 EI 6 EI L 1 \theta C = EI 2 EI 6 EI L 1 \theta C = EI 2 EI 6 EI L 1 \theta C = EI 2 EI 6 EI L 1 \theta C = EI 2 EI 6 EI L 1 \theta C = EI 2 EI 6 EI L 1 \theta C = EI 2 EI 6 EI 2 E$ $dx \, \delta B = \dot{o} \, \dot{c} \, \dot{c} \, 2 \, EI \, 0 \, \phi + 6 \, \ddot{o} \, 1 \, a c \, w \, 3 \, 2 \, 2 \, \delta B = c \, 2 \, c \, 2 \, \delta B = c \, 2 \, c \, 2$ 1 EI oM 1 EI o (1)ccce18w + 6wx + 2 x 0 6 Q · M P dx æ w 0 0B = 2 ö + 6 P + Px ÷ dx ÷ ø 576 EI 8-25 Copyright © 2018 McGraw-Hill Education. The effect of a deep stiff continuous girder is to spread the concentrated load longitudinally to adjacent vertical cables, thereby applying a more uniformly distributed load to the arch. P8.15. 9-34 Copyright © 2018 McGraw-Hill Education. (Shorten) 8-46 Copyright © 2018 McGraw-Hill Education. Complete the analysis by method of joints. Select the origin at A. Member end Forces: MAB = -61.2 ft k MBA = -72.6 ft k ft k MBC = 101.1 MCB = 119.9 ft k ft k MBD = -28.3 MDB = -14.2 Segment AB MBA = 26.72 k MBC = 73.4 k Segme $(8c) = 0.28.3 - 14.2 + VDB VDB = 5.3k + \Sigma Fy = 0; VBD = -VDB = 5.3k 10-31 Copyright © 2018 McGraw-Hill Education. 10 kips 10 kips 8 C D 4 kips 6' E A H G F 4 @ 8' = 32' P4.13 Compute RE : \Sigma M about G + \Sigma M G = 0; 10 k '8c - 4 k '6c - RE 16c = 0 RE = 3.5 kips + \Sigma Fx = 0; FHG - 4 = 0 \ FHG = 4 k Complete by Method of Joints.$ P15.2. Using the stiffness method, determine the horizontal and vertical components of displacement of joint 1 in Figure P15.2. Also compute the reactions, draw the shear and moment curves for the beam in Figure P9.8 if segment AB has 1.51. 40 kips 35 kips 30 kips I J B 35 kips H 40 kips G C F D 50' 38.46' 39 kips A E 4@30' = 120' 90 kips 90 kips 90 kips 90 kips P8.16 P-System Arch Support A Funicular Loading, Therefore MP = 0 FDE = 392 + 50 2 = 63.41 kips FBC = 152 + 392 = 41.79 kips M P = 0 All Members. For older bridges constructed at a time when highway loads were much smaller, the addition of cables is an economical way to increase the P2.15. FP L = $\Sigma FQ \Delta LP AE 2(68.57)(5.66) = -1(.00412) + 500' 200 = -0.00412 + 0.00541 1k \cdot \Delta AX = 0.00137 \text{ mm} \Delta AX = 0.00137 \text$ (compression forces in columns reduce cracking).

(e) What other conclusions can you draw from the study? 15' 100' 15' 100' P2.18 (a) Wind Loads Using Simplified Procedure: Design Wind Pressure Ps Kzt IPS30 1.66 Table 2.8, Mean Roof Height 30' PS = 1.66(1)1.15PS 30 = 1.909 PS 30 Zones PS 30 A C 12.8 psf 8.5 psf 24.44 psf 16.22 psf Resultant Force at Each Level; Where Distance a 0.1(100) 10; 0.4(30) 12; 3 a = 10 ¢ Controls & 2a = 20 ¢ Region (A) 15¢ 24.44 psf 2 15¢ Zone (C): 16.3psf 2 Froof : Zone (A): (¢ k ¢ k 20 = 3.67) 1000 80 = 9.78) 1000 Froof Resultant = 13.45k (F2nd: Zone (C): 15¢ 16.3 psf ¢ 20 = 7.33) 1000 ¢ k 80 = 19.56) 1000 k F2nd Resultant = 26.89k Base Sheak Vbase = Froof + F2nd = 40.34 k Overturning Moment MO.T. = Σ Fi hi MO.T. = 13.45 (30 ¢) + 26.89k (15¢) = 806.9 ft . 5 5 P13.12. How far should support A be moved? = -0.9k çè 2 ÷ø c 2 ÷ø - RC max. E is constant, but I varies as noted. Members have been detailed such that the support at D acts as a simple support for both the tower and the roadway girders. 2 The area of all bars = 1.8 in. Assume that vehicles move along the center of the roadway so one-half the load is carried by each truss. If yes, compute the horizontal displacement of joint B. However, to satisfy architectural requirements, the depth of the columns will be as small as possible.

Continued Forces in member AF Compute V in AB & FE + $\Sigma M A = 0 = 4V - 60 - 75 135 V = 33.75 kN 4$ Since V = 25 kN reduce all forces 25 = 0.740741 by Ratio of 33.75 60(.74071) = 44.44 + 44.44 + V 3 V = 29.63 kN Member and Moments (kN·m) Axial Forces in all Members in kN. P5.3. Write the equations for shear and moment between points A and B. 2. (frame is not stiff enough) $\Delta D = 0.834 in. 20 kN 40 k$

= 0 0 = 54.15(49) -165(27.5) + RC 19 RC = 99.17 + Σ Fy = 0 = 54.15 + 99.17 -165 + VP. On the windward side, evaluate the magnitude of the wind pressure every 35 ft in the vertical direction. B 30 kN P4.8. Determine the forces in all bars of the trusses. , 2 2 area of column = 6 in. P5.36. (c) Compare the results with the exact analysis using a computer software. too long, determine the moments and reactions created in the frame when it is erected. B C D E F H I 20' G 24' 12' 12' 24' P12.19 12-20 Copyright © 2018 McGraw-Hill Education.

Recompute the reactions for the beam in Figure 9.4 if a spring with K = 235 kips/in. $\times 5.5$ in. E P9.48. (Hint: You will need two moment equations: Consider the left or right of the hinge at B.) 20 kN 30 kN B 18 kN hinge 8m C 12 m A 5 @ 8 m = 40 m P6.25 + $\Sigma M A = 0 = 10$ '8 + 20 '16 + 30 '32 - 18' 20 - C y 40 - C x 12 (Eqn1) + $\Sigma M B$ (FBD to right of hinge) = 0 0 = 30 kN '8m - C y 16m + C x 8m (Eqn2) + + Solving Eq's 1& 2 gives C y = 21.25 kN C x = 12.5 kN $\neg \Sigma Fx = 0 = Ax \cdot 18 \cdot 12.5 \setminus Ax = 30.5 kN + \Sigma F = 0 = 21.25 \cdot 60 + A \setminus A = 38.75 kN y y y 6-28$ Copyright © 2018 McGraw-Hill Education. P12.28. Bottom chord in tension top chord in cohpression some truss action. P11.14. Moment (kip-ft) x -109.2 Δ max at 10.37 ft 9-12 Copyright © 2018 McGraw-Hill Education. Continued Axial Force Diagram (kips) Moment Diagram (kip-ft) The structure is composed of a deep, stiff, continuous girder, which is supported by pin supports at each end as well as by vertical cables that transmit roadway loads to the supporting arch.

, A = 4 in.

Using moment distribution, calculate the ordinate of the influence line at section 4. All reactions are given. Δ HC = 30 = 0.711 in.

3-31 Copyright © 2018 McGraw-Hill Education. 12-39 Copyright © 2018 McGraw-Hill Education. 12.5 ft 30 kips C D 1 POI POI 1 POI 6 ft E 12.5 ft (a) Portal Method 6 ft B FBD from Section 1: POI 30 kips C M1 6 ft M1 A V1 1 25 ft V1 F1 F1 Σ Fx = 0 = 30 - 2V1 V = 15 kips 1 VCD 30 kips C M1 FCB MCD = M1 = 90 kip-ft C FCD 12.5 ft V1 = 15 kips 2018 McGraw-Hill Education for D FCD = 15 kips 12.5 ft VDC = -7.2 D MDE = 90 kip-ft V1 = 15 kips Equilibrium of Beam CD Σ Fx = 0 = 30 - V - FCD Σ M POI = 0 = MCD - VCD (12.5) (left half of CD) FCD = 15 kips VCD = 7.2 kip Σ MC = 0 = M1 - MCD 6) Σ Fy = 0 = -7.2 + VDC MCD = 90 kip-ft V1 = 15 kips Equilibrium of Beam CD Σ Fx = 0 = 30 - V - FCD Σ M POI = 0 = MCD - VCD (12.5) (left half of CD) FCD = 15 kips VCD = 7.2 kip Σ Sector 1 and (a) control (b) Σ Fy = 0 = -7.2 + VDC MCD = 90 kip-ft V1 = 15 kips (CD = 7.2 kip Σ Sector 1 and (a) control (b) Σ Fy = 0 = -7.2 kip Σ Sector 1 and (c) control (c) Σ Fy = 0 = -7.2 kip Σ

2018 McGraw-Hill Education. (f) hinge A P3.34. 120 kN P8.3. For the truss in Figure P8.3, compute the horizontal and vertical components of the displacement of joint C. Draw the shear and moment curves for the beam in Figure P5.26. Loads are transmitted into the truss through the lower chord panel points. w = 3 kips/ft 20 kip ft C B A 4' 8' D 8' P10.7 Unknowns: θB , θC FEMS 3(8) 2 WL2 == -16 k·ft, 12 Member end Moments FEMCB = 16 k·ft FEM BC = - M AB = -20 k·ft 2 EI (2 $\theta B + \theta C$) -16, 8 2 EI MCD = (2 θC), 8 Equil. Hint: Start with the method of sections. Given: E = 29,000 ksi and 4 I = 100 in. FBD "BCD" + $\Sigma M d = 0$; -20 kN (5m) + 60 kN + Dy (15m) = 0 $\Sigma Fy = 0$; + - 20 + Cy - 2.67 = 0 Dy = 2.67 kN Cy = 22.67 kN Entire Structure 3-9 Copyright © 2018 McGraw-Hill Education. spring compression, XB = 10 kips 15K S 48 K S = 32 kips/in 10 = 55.15 kips/in 1 in. Given: EI is constant. Continued Reacations Joint Label X Force (k) Y Force (k) Moment (k-ft) A -4 921 -6.667 0 K - 5.079 6.667 0 Totals: -10 0 Joint Displacement Joint X Translation Rotation Label (in) (in) (radians) A 0 0 -1.036e-2 B 1.391 .003 -2.459e-3 L 1.401 -.018 0 M 1.407 -.021 0 N 1.411 -.014 0 O 1.407 -.021 0 N 1.411 -.014 0 G 1.413 -.006 0 H 1.414 0 0 I 1.418 -.004 4.326e-4 K 0 0 -1.044e-2 Moment Diagram Arial Forces (See Label Diagram w/Table of MBR Forces) 13-44 Copyright © 2018 McGraw-Hill Education. $\Sigma M A = 0 = 12'3 - FBCX FGCY = 9$ kN tension 9 = FGCY 4 = FBCX = 27 kN compr. Similar analysis of ther joints will show all interior base scept AH and HG are zero. Note: The interior columns must be designed for both the floor loads and the compression force created by the temperature differential. J I H G 3m A B 12 kN C 16 kN D E 12 kN G 4 k J = 0.2 (5 k) - Ay = 0 Ay = 1.37.5k 2.5(5 k) - Ay = 0 Ay = 1.37.5k 2.5(5 k) - Ay = 0 Ay = 1.37.5k 2.5(5 k) - Ay = 0 Ay = 1.37.5k 2.5(5 k) - Ay = 0 Ay = 1.37.5k 2.5(5 k) - Ay = 0 Ay = 1.37.5k 2.5(5 k) - Ay =

Section 2-2 Forces in Top and Bottom Chords See Section 1-1 Above + 2M B = 0; $0 = 20^{-4}$ - FJ1 6 FJ1 = 13.33 kN compr. Reactions at support A are given. Find the vertical displacement of joint D. E P/2 D 2 15 P C 15 P B 18 A P 3.46 2F y = 0; + 4 = 0; 2.5(55k) - Ay = 0 Ay = 137.5k 2.5(55k) (0.835k) - E x (50 k) = 0 E x = 2.29k - 4 = 0; 2.5(55k) - Ay = 0 Ay = 137.5k 2.5(55k) (0.835k) - E x (50 k) = 0 E x = 2.29k - 4 = 0; 2.5(55k) - Ay = 0 Ay = 137.5k 2.5(55k) (0.835k) - E x (50 k) = 0 E x = 2.29k - 4 = 0; 2.5(55k) - Ay = 0 Ay = 0 Ay = 137.5k 2.5(55k) (0.835k) - E x (50 k) = 0 E x = 2.29k - 4 = 0; 2.5(55k) - Ay = 0 Ay = 0 Ay = 137.5k 2.5(55k) (0.835k) - E x (50 k) = 0 E x = 2.29k - 4 = 0; 2.5(55k) - Ay = 0 Ay = 0 Ay = 0; 2.5(55k) - Ay = 0 Ay = 0 Ay = 137.5k 2.5(55k) (0.835k) - E x (50 k) = 0 E x = 2.29k - 4 = 0; 2.5(55k) - Ay = 0 Ay = 0 Ay = 137.5k 2.5(55k) (0.835k) - E x (50 k) = 0 E x = 2.29k - 4 = 0; 2.5(55k) - Ay = 0 Ay = 0 Ay = 137.5k 2.5(55k) (0.835k) - E x (50 k) = 0 E x = 2.29k - 4 = 0; 2.5(55k) - Ay = 0 Ay = 0 Ay = 137.5k 2.5(55k) - Ay = 0 Ay = 0 Ay = 137.5k 2.5(55k) - Ay = 0 Ay = 0 Ay = 137.5k 2.5(55k) - Ay

EI $\Delta CO = tCA = Therefore$, closing the gap due to a unit load at C: 60 kips A C B t CA = ΔCO ($\Delta CS + \Delta CO$) + $\delta CC XC = \Delta C$ (0 - 276.48) + 1.642 X C D 60 kips = 0 1500/EI XC = FCable = 168.4 kips 300/EI 20/3 (2)20/3 20+20/3 20+(2)20/3 3900/EI 168.4 kips MA = 327kip-ft A 143.1 kips NA = 143.1 kips 60 kips B 89.3 kips C D 60 kips Ay = 30.8 kips 9-55 Copyright © 2018 McGraw-Hill Education. Continued Total Closing of Gap by Unit Load $\delta FA = 0.004966 \delta BB = 0.007094 0.002412 0.014472 = 168$ in Truss = X = 116.09 kips Force in Exterior Columns 116.09 kips (Tension) Force in Interior Columns 232.18 kips (Compression) Final Difference in Length 1.1k - 0.82 = 0.28¢¢ Force in Bonnet Truss 9-58 Copyright © 2018 McGraw-Hill Education.