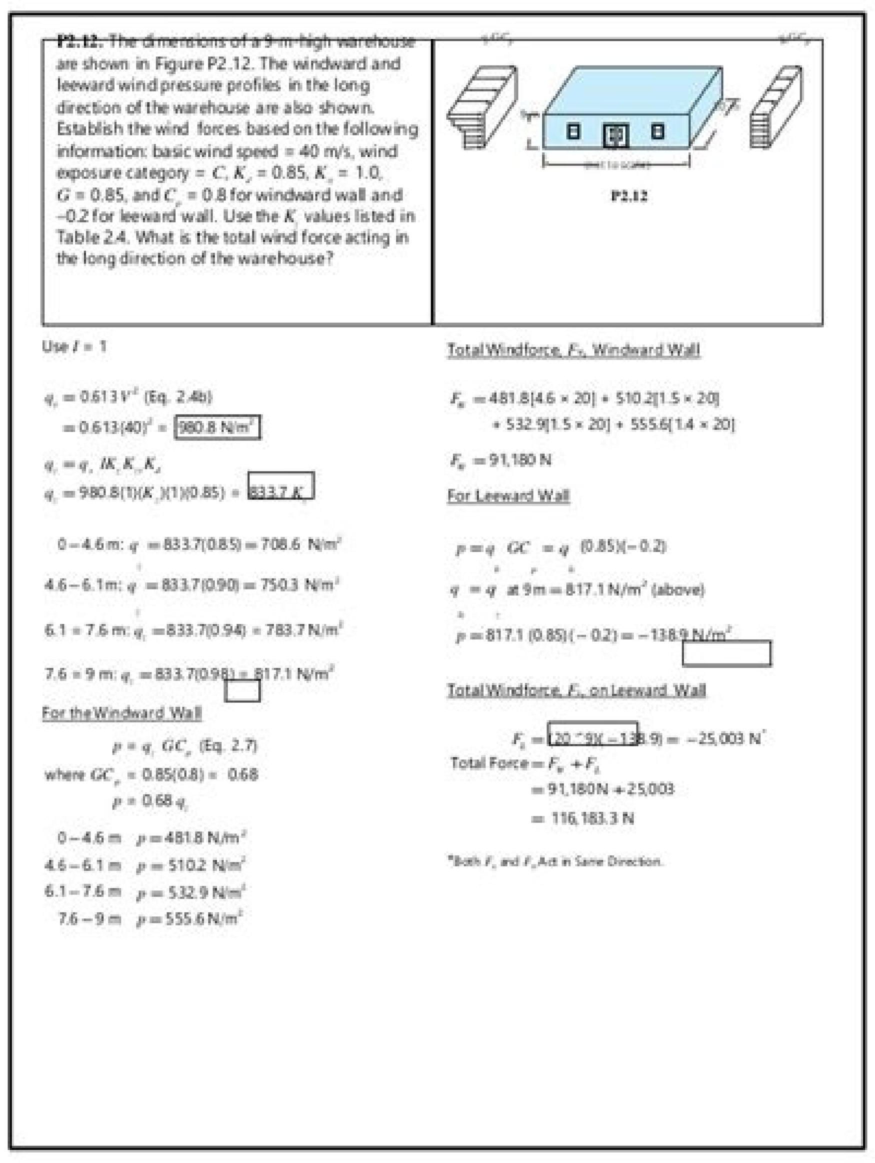
Fundamentals of structural analysis 5th edition solutions manual

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: θB 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 + θB) - 75 20 2 EI M BA = (2θB + 0) + 75 20 2 EI 392 M BC = (2θB + 0) 28 3 2 EI 392 MCB = (0 + θB) + 28 3 FEM AB = - FEM BA = 75 k · ft FEMCB = 392 k · ft 3 Shear Moment Equil. The horizontal reaction at support F is zero.
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 : ΣMC = 0 @ x = 24 ¢ - Ax (18) + Ay (24 ¢) = 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 ΣM A = çç ÷÷ (4)7 + 7(60) - 8 -12C y = 0 çè 2 ÷ø C y = 42.5 kips ΣFy = Ay - 4(7) - 60 + 42.5 = 0 Ay = 45.5 kips ΣFx = Ax + 12 = 0 Ax = 12 kips Cut of segment AB ΣFy = 45.5 - 4 x - V = 0 V = 45.5 - 4 x kips æxö ΣM = M + çç ÷÷· 4 x - x (45.5) = 0 çè 2 ÷ø M x = 45.5 x - 2 x 2 kip · ft Cut segment CB æ 169 ö÷ ÷÷· C = 46.04 kips C y ¢ = ççç y çè 12 ÷÷ø ΣFy = V + 46.04 = 0 V = -46.04 kips ΣMC = 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: Δ AO = -1.2 ¢¢ Δ AO + δAA X A = 0 δAA : From Table A.3 δAA = Pa 2 1(24)2 (L + a) = (16 + 24) 3EI 3EI Select RA as the Redundant 7680(1728) = 1.525 in/k (29,000)(300) Compatibility Equation: = -1.2 ¢¢ + (1.525) X 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.


P2.12. The dimensions of a 9-m-high warehouse 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, K_d = 0.85, K_{zt} = 1.0, G = 0.85, and C_p = 0.8 for windward wall and −0.2 for leeward wall. Use the K_z 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

$q_s = 0.613V^2$ (Eq. 2.4b)
$= 0.613(40)^2 = \boxed{980.8\ N/m^2}$

$q_z = q_s\ IK_z K_{zt} K_d$
$q_z = 980.8(1)(K_z)(1)(0.85) = \boxed{833.7\ K_z}$

0 – 4.6 m: q_z = 833.7(0.85) = 708.6 N/m²
4.6 – 6.1 m: q_z = 833.7(0.90) = 750.3 N/m²
6.1 – 7.6 m: q_z = 833.7(0.94) = 783.7 N/m²
7.6 – 9 m: q_z = 833.7(0.98) = 817.1 N/m²

For the Windward Wall
$p = q_z\ GC_p$ (Eq. 2.7)
where GC_p = 0.85(0.8) = 0.68
$p = 0.68\ q_z$
0 – 4.6 m: p = 481.8 N/m²
4.6 – 6.1 m: p = 510.2 N/m²
6.1 – 7.6 m: p = 532.9 N/m²
7.6 – 9 m: p = 555.6 N/m²

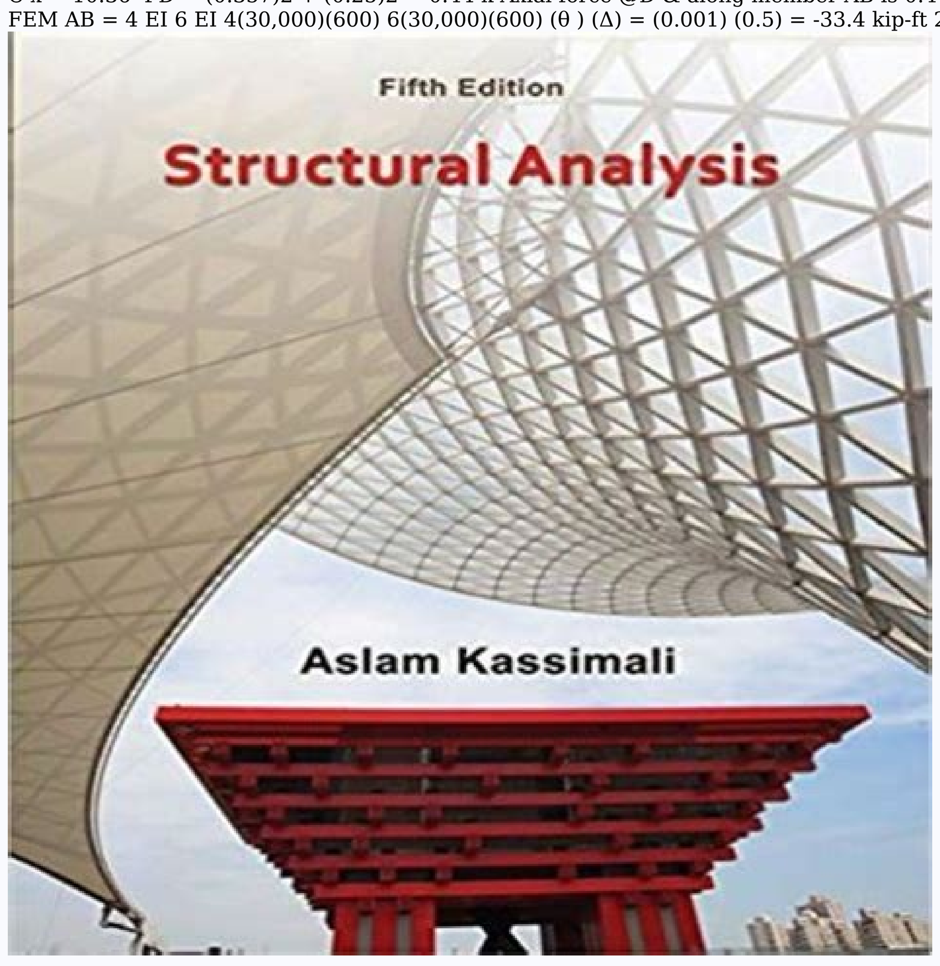
Total Windforce, F_w, Windward Wall
F_w = 481.8[4.6 × 20] + 510.2[1.5 × 20] + 532.9[1.5 × 20] + 555.6[1.4 × 20]
F_w = 91,180 N

For Leeward Wall
$p = q_h\ GC_p = q_h\ (0.85)(-0.2)$
$q_h = q_z$ at 9 m = 817.1 N/m² (above)
p = 817.1 (0.85)(− 0.2) = −138.9 N/m²

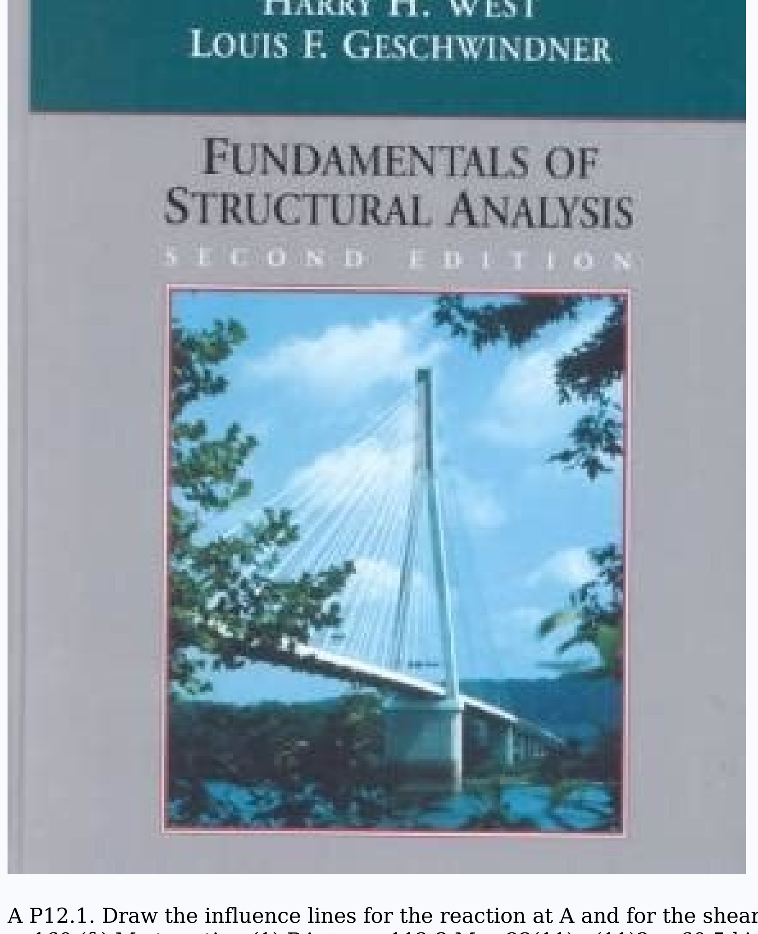
Total Windforce, F_L, on Leeward Wall
F_L = [20 × 9] × −138.9 = −25,003 N
Total Force = $F_w + F_L$
= 91,180N + 25,003
= 116,183.3 N

*Both F_w and F_L Act in Same Direction.

ki ps ki ps ki ps ki ps ki ps 9′ A A 10 kips 10 @ 12′ = 120′ ΣP 5k ′ 9 + 2.5k ′ 2 = = 0.417 kips/ ft L 120 ft I of truss considering flanqes only: I = 10 ′(54)2 ′ 2 = 58,320 in 4 Δ£ = bottom chord A = 10 in.2 Section A-A Treat truss as a uniformly loaded beam I = Ad 2 ′ 2 Where 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 l 1.2l 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 ΣK = 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 BC 40 (8)16 wL2 PB 2 82 5 (12) + = + = 95.55 kN · m 2 2 12 12 L (12) 2 FEMCB = Moment Distribution A 10/19 B 9/10 –33.75 +33.75 –131.11 +51.24 +46.12 84.99 84.99 +25.62 ← –8.13 C → 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 = 84.90 kN 11-3 Copyright © 2018 McGraw-Hill Education. 4-59 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 I A P10.4 θ A = θC = 0 ü 2 EI 24 ′10 ï ï M BA = (2θB + 0) ï ï 10 8 ï 2 EI 24 ′10 ï ï (0 + θB) ï ï 10 8 ï 2 EI 24 ′10 ï ï ï M BA = (2θB + 0) ï ï 10 8 2 ý 2 EI 12(14) ï ï M BC = (2θB + 0) ï 14 12 ï ï 2ï 2 EI 12 ′(14) ï ï MCB = (0 + θB) + ï ï 14 12 ï þ Equilibrium at Joint B M AB = (1) M BA + M BC = 0 4 EI 4 EI θ + 30 + (2θB) -196 = 0 10 B 14 242 θB = (2) EI 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: ΔD = 0.272 in. A w = 2 kips/ft 12′ 2 kips D B C 8′ E 16′ 4′ P10.33 + Unknowns: Δ, θB , θC , θE , (B) Joint C: ΣMC = 0 Δ Δ ΨAB = - , ΨCE = 12 8 ΨAB -Δ/12 = ΨCE Δ/8 2 ΨAB = - ΨCE 3 wL2 2(16)2 = = 42.66 kip · ft FEM BC = 12 12 (1) Express end moments in terms of displacements 2 EI M AB = (θ - 3ΨAB) 12 B 2 EI M BA = (2θB - 3ΨAB) 12 2 EI M BC = (2θB + θC) - 42.66 16 2 EI MCB = (2θC + θB) + 42.66 16 2 EI MCE = (2θC + θE - 3ΨCE) 8 2 EI M EC = [2θE + θC - 3ΨCE] 8 (A) Joint B: ΣM B = 0 Joint B Joint C (C) Joint E: ΣM E = 0 Shear Equation Where V1 = (D) 2 - (2) Equilibrium Equations M BA + M BC = 0 MCB + MCE -16 = 0 M EC = 0 + ΣFx = 0 M AB + M BA 12 & V2 = 2 k - V1 + V2 = 0 MCE 8 M AB + M BA MCE + =0 12 8 (3) Complete solution by substituting Values in Step1 into Equil EQ's and Solving for the Unknown Displacements. Given: I = 360 in. C P9.26. (a) (e) (b) (f) pinned joint (c) (g) (d) (h) P4.1 (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 ΔBO = 0 (a) Use RB as Redundant 0 = Δ BO + δBB [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 (a): ΣM A = -12 Bx + 9 By = 0 By = 1.33Bx Figure (b): 19 (19) = 0 2 180.5 = 10 (1.33Bx) + 12 Bx ΣMd = 10 By + 12 Bx - Bx = 7.125 kips By = 9.5 kips Ax = Bx = 7.125 kips Ay = By = 9.5 kips Figure (b): ΣFx = Bx + Dx = 0 ΣFy = -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 Δy æ 43 AE ö÷ ÷ D = 51.96 ççç è 25 L ÷ø x æ 32 AE ö÷ çç ÷ D = 30.00 èç 25 L ÷ø y 30.2 L = 0.081¢¢ AE 23.44 L Dy = = 0.063¢¢ AE Dx = AE AE AE æ 30.2 L ö÷ (cos fx) D x + (sin fx) D y = (1) çç è AE ø÷÷ L L L 30.2 L ÷ö AE æ 4 ÷öæ 23.44 L ÷ö AE æ 3 öæ ç ÷÷ç ç ÷÷ç F2 = ÷÷ ÷ ÷ ç ç è øè L 5 AE ÷ø L çè 5 ÷øèç AE ÷ø 23.44 L ÷ö AE æ 3 ö÷ 30.2 L AE æ 4 öæ çç- ÷ çç ÷÷ç F3 = ÷ ÷ ÷ ÷ è ø è øè 5 AE ÷ø F1 = F1 = 30.2 k Joint 1 F2 = 36.87k F3 = 0.63k 15-5 Copyright © 2018 McGraw-Hill Education. EI is constant for all members. 2 and E = 29,000 kips/in. B A hinge r = 20 m θ = 45° C P6.19 Determine Reactions: Equation (1): + Σ MC = 0; Ax (14.14) + Ay (34.14) - 56.56(27.07) - 80(10) = 0 Equation (2): ΣMB for segment AB, + Σ M B = 0; Ay (14.14) - Ax (20 -14.14) - 56.56(7.07) = 0 Solve EQ'S (1) and (2) Ax = 48.3 kN Ay = 48.3 kN ΣFx = 0; C x = 48.3 kN ¬ ΣFy = 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 B: M BA + M BC = 0 at Joint C: MCB + MCD = 0 at Joint A: M AB = 0 Shear Equation: V1 - V2 = 0 Where M AB + M BA 10 MCD + M DC V2 = 10 V1 = 10-41 Copyright © 2018 McGraw-Hill Education. 43.5 kN • 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 P11.4. Δ AD + RA δAA = 0 980 88.2 + RA =0 AE AE RA = 11.11 kN - P-System Relesed Structure Q-System for Δ AD P & Q System for δAA Bar Force and Reactions 9-40 Copyright © 2018 McGraw-Hill Education. A B C I = 600 in.4 I = 200 in.4 D 15′ 20′ P11.26 Analysis for temperature forces. M AB = M ED = -82(0.711) = M AB = M ED = -58.3 kip-ft M BA = M DE = -55.3(0.711) = M BA = M DE = -39.3 kip-ft M BC = M DC = 55.3(0.711) = M BC = M DC = 39.3 kip-ft MCB = -MCD = -50.8(0.711) = MCB = -MCD = -36.3 kip-ft MCF = -101.3(0.711) = MCF = -72 kip-ft Ax = E x = -11.4(0.711) = Ax = E x = -8.1 kips ¬ Ay = -E y = -11.4(0.711) = Ay = -E y = -4.7 kips Fx = -19.4(0.711) = Fx = -13.8 kips ¬ 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 + ΣM 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: + ΣM A = 0; 30 k (10 ¢) - 6 k (20 ¢) - E y (20 ¢) = 0 E y = 9k + ΣFy = 0; Ay - 30 + 9 = 0 k + k ΣMC = 0; E x (20 ¢) - 9k (10) = 0 E x = 4.5k ¬ Ay = 21k Entire Structure: + ΣFx = 0; - 6 k - 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′ 4′ 12′ 2′ 2′ P3.35 Freebody Timber Member + ΣFx = 0; 20 k - BFx = 0; BFx = 20 k ¬ BFy /11 = BFx /10 ¢ BFy = 22 k BF = BFx 2 + BFy 2 = (20)2 + (22 k)2 BF = 29.73k Compression in the link + Σ M e = 0; - BFy (4 ¢) - CG (8 ¢) = 0 - 22 k (4 ¢) - CG (8 ¢) = 0 CG = 11k Compression in the link Σ 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 + ΣM A = 0; 20 k (22 ¢) - 3 k/ft (24 ¢) (16 ¢) + M A = 0 M A = 7121 · k + —— ΣFx = 0; 20 k - Ax = 0 Ax = 20 k ¬ ΣFy = 0; + - 3 k/ft (24 ¢) + Ay = 0 Ay = 72 k 3-36 Copyright © 2018 McGraw-Hill 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 © 2018 McGraw-Hill Education. A B D C 5m 5m P10.14 Since the structure and the applied load are symmetrical with respect to a vertical axis thru support C, θC = 0 and the joint at C can be treated as a fixed EUD. , α = 6.5 × 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. + ΣMC = 0; FDBX (6 ¢) - 2.25(12 ¢) = 0 FDBX = 4.5k FDBX F = DBY ¢ 12 9¢ FDBY = 3.375k (tension) Freebody Joint B + ΣFy = 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 Reactions and Member end Moments + 9 ΣM B = 0; 54 kN ′ - RD 9 - 5.5 = 0 2 RD = 26.39 kN + ΣFy = 0; VBD = 54 - 26.39 = 27.61 kN + Member BD: Shear & Moment Diagrams ΣM 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 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 ΣFy = 0; 160(2) + 156.25 - K = 0 K = 476.25k / ¢¢ (b) Displacement due to 24k F = ΔK 24 k = Δ(476.25) Δ = 0.0504 ¢¢ (c) Bar Forces due to 24k @ A (156.25) = 7.88k 1¢¢ (200) = 0.0504 ¢¢ = 10.08k 1¢¢ FAC = 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 + ΣM A = 0; + ΣFy = 0; æ 20 ö 20(5m) + 40 çç ÷÷ - 5 RB + 20(2m) = 0 çè 3 ÷ø RB = 81.33 kN - 60 kN + 81.33 - RAY = 0 RAY = 21.33 kN 5-30 Copyright © 2018 McGraw-Hill Education. and I = 8.22 in. Compute the reactions and draw the shear and moment curves for the beam in Figure P10.14. FBD "EC" (Ax = (0.357) + 10 k - C x = 0 C x = 10.36 FD = (0.357)2 + (0.25)2 = 0.44 k Axial force @D & along member AB is 0.44k compression. EI æ 442.16÷ ç ÷ = -178.2 5 çè EI ø÷ 14-14 Copyright © 2018 McGraw-Hill Education. (a) Draw a qualitative influence line for the reaction at support A for the beam in Figure P12.57. , and E = 30,000 2 kips/in. A 0.001 rad B C 1/2" 24′ 8′ P11.22 FEM AB = 4 EI 6 EI 4(30,000)(600) 6(30,000)(600) (θ) (Δ) = (0.001) (0.5) = -33.4 kip-ft 2 2 L 12 (24(12)) L 12 (24(12)) FEM BA = 2 EI 6 EI 2(30,000)(600) 6(30,000)(600) (θ) (Δ) = (0.001) (0.5) = -43.8 kip-ft 2 2 L 12 24(12) ( ) L 12 (24(12)) FEM BC = -16(8) = -128 Ay = 7.5 kips 16 kips 0.001 rad 0.5 in. E is 2 4 constant and equals 30,000 kips/in.

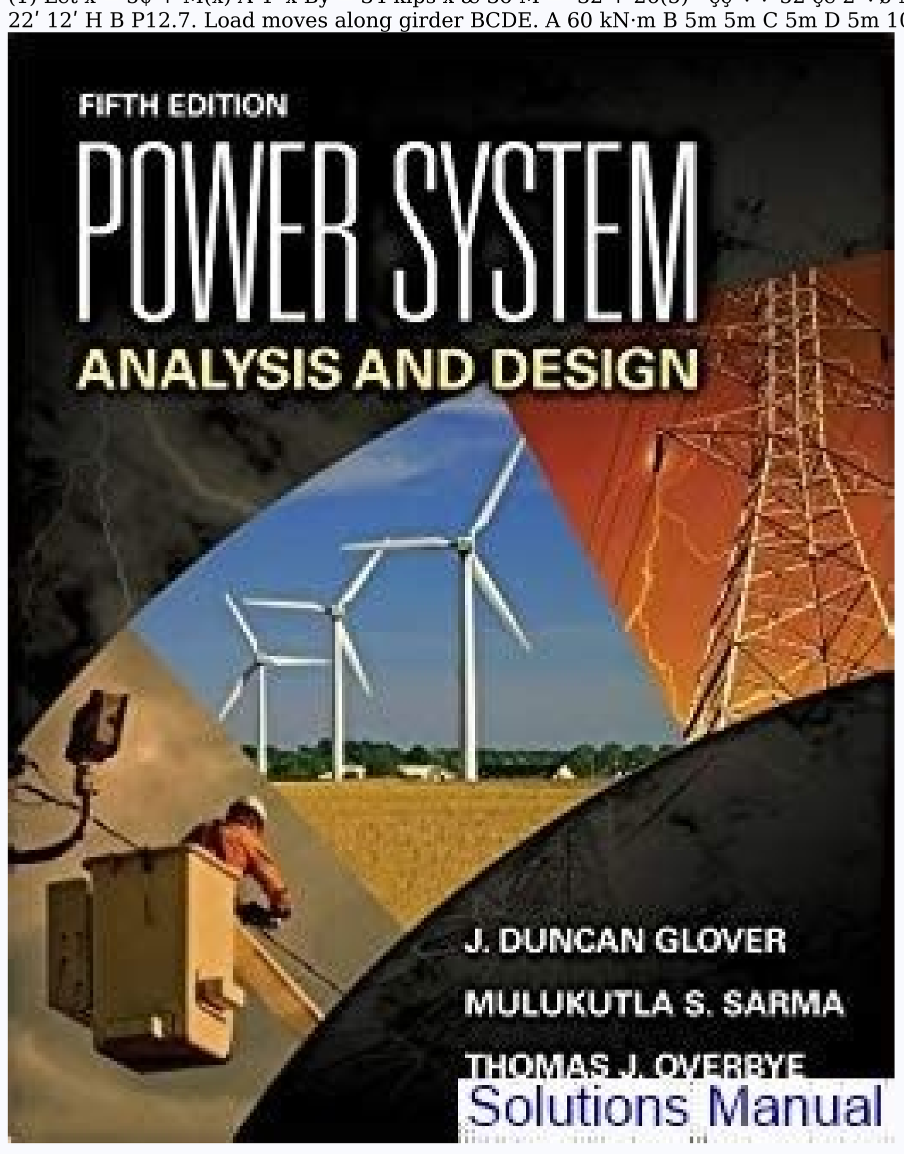


P6.34. Using the influence lines, determine the maximum live load force (consider both tension and compression) produced by the 54-kip truck as it transverses the bridge, which consists of two trusses.
x 2 æç x ö÷ ç ÷- M = 0 4 çè 3 ÷ø æxö M = 12 + x - R çç ÷÷÷ èç 3 ø 1(x) + 12 - M = 12 + x - Check@ x = 6: V and M Diagrams 1 1kN - ´W ¢ - V = 0 2 1 æxö V = 1 - ´çç ÷÷ 2 çè 2 ø÷ x3 12 Ans. 2 2 n =3 AE AE éê æ 3 ö÷ æ 3 ö÷ ùú 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 öæ 4 ö æ 3 öæ 4 ö AE ê 0 + çç ÷÷÷çç ÷÷÷ + çç- ÷÷÷çç ÷÷÷ú K 21 = Σ K 21 = 0 cos fx sin fx = 1 è 5 øè 5 ø è 5 øè 5 øúû L L êë æ 3 öæ 4 ö æ 4 öæ 3 öù AE AE é ê 0 + çç ÷÷÷çç ÷÷÷ + çç ÷÷÷çç- ÷÷÷ú sin fx cos fx = è 5 øè 5 ø è 5 øè 5 øúû L L êë 2 2 n =3 AE æ4ö æ4ö ù AE éê K 22 = Σ sin 2 fx = 0 + çç ÷÷÷ + çç ÷÷÷ ú 1 è 5 ø è 5 ø ûú L L ëê n =3 K12 = Σ 1 K12 = 0 K 22 = 32 AE 25 L 15-4 Copyright © 2018 McGraw-Hill Education. P11.12. The bar areas are shown in the figure. Draw the influence lines for the vertical and horizontal reactions, AX and AY, at support A and the bar forces in members AD, CD, and BC. (a) Compute the reactions in Figure P9.42. B 4m C 2m D 4m E 2m RA P12.15 Unit Load at: Reaction on Girder A B C Ay 1 0 –0.5 Mid-span BE –0.2 By 0 1 1.5 Ey 0 0 Fy 0 0 D E F 0 0 0 0.7 0 0 0 0 0.7 1.5 1 0 0 –0.2 –0.5 0 1 Moving Load 1 kip A C B Ay E D F Ey By A Fy 0.25 B 4m 2m 2m 2m 2m 4m 1.0 0.75 0.625 0.5 0.375 Ay (kips) 4.5 3 2.8 1.5 1 MB (kip-ft) 0 1.125 0.75 0.70 0.375 VAB (kips) 0 0 0.25 12-16 Copyright © 2018 McGraw-Hill Education. + ΣM P . w = 2 kips/ft 1′ 8 kips C B 6" 4′ 6 kips 12" 5′ A 6′ P3.28 Compute Reactions at Support A ΣFx = 0; 6 - RAx = 0 \ RAx = 6 kips ¬ ΣFy = 0; RAy - 2 kips/ ft (6.5¢) - 8 = 0 + + \ RAy = 21 kips Σ Moments about & of base at "A" + ΣM A = 0; æ 6.5 ö 8 ´6 + 13k ´çç6 - ÷÷÷ + 6 ´ 5 - M A = 0 2 ø èç \ M A = 113.75 kip · ft Freebody Diagrams Member "BC" & JT. 20′ A P12.20. = (+8)(14 ¢)1.2 k / ft. 6 kips/ft P3.19.

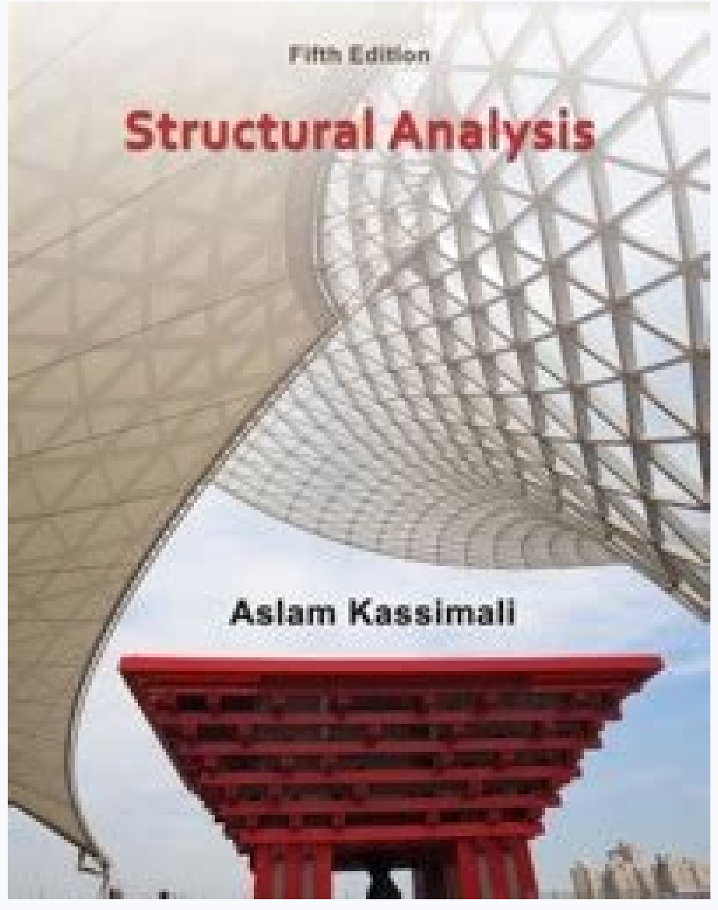


A P12.1. Draw the influence lines for the reaction at A and for the shear and moment at points B and C. Continued 3x (e) B–C + ΣFy = 0; M(x) V - 3 x + 22 = 0 V = -22 + 3 x z x V(x) + ΣM z = 0; A–B + ΣFy = 0; Cy = 22 kips x M + 3 x · - 22 x = 0 2 1 M = 22 x - x 2 2 8 kips -8 k - V = 0 V(x) M(x) V = -8 k A 20 – x + ΣM 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 · ft 2 (g) Mmax, Set V = 0; -22 + 3 x = 0 22 = 7.332 3 3 = 22 ´ 7.33 - (7.33)2 2 = 161.26 - 80.59 V= Mmax Mmax = 80.67 kip · ft + (h) ΣFy = 0; -8 + 34 - 3 x - V = 0 V = 26 - 3 x 3x 8 kips 3x · x ΣM 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¢ + M(x) A 4' x By = 34 kips x æ 3õ M = -32 + 26(5) - çç ÷ ÷ 52 çè 2 ÷ø M = 60.5 kip · 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 ´δ = Σ ò MQ M P M 2 dx = EI 100 EI ò P 1.5 é 4 ù 1 ê 4 M 2 dx + 2 M dx ú δ= P P ò ò ê ú 100 EI ëê 0 -1.5 ûú Use Q-System = 2 EI (2θ - 3Ψ) L 2 EI M EF = -55.55 = (θ - 3Ψ) L 11.11L Solving for θ = 2 EI -66.66 L Ψ= 3´ 2 EI Δ -66 L Ψ= = L 3´ 2 EI M FE = -44.44 = Δ= Member AB Member AF 1.5 é 4 ù ê (-44.5 + 25 x)2 dx + 2 (29.67 x 2)dx ú ò ò ê ú ëê 0 ûú -1.5 1 é 4(3454.3) + 2(1380.7)ùû = 100 EI ë 17, 779 17, 779 = = 100 EI 100(200 ´106)(250 ´10-6) δ = 0.00356 = 3.56 mm 1 = 100 EI -66 L2 -66(4)2 = 2 EI 6 ´ 200 ´ 250 Δ = 0,00356 m = 3.56 mm 11-46 Copyright © 2018 McGraw-Hill Education. P = 10 kips w = 2 kips/ft P5.39. 120 kN B C 2I I 5m I D A 5m 5m P14.10 FEM23 = -FEM32 = -2 PL -2(120)15 = = -400 kN·m 9 9 θ2 = 1 rad M 23JD M32JD 2 3 From a unit rotation of Joint 2 (and 3): M 21JD 2 EI 2 EI (2(0) + 1) = 5 5 2 EI 4 EI JD M21 = (2(1) + 0) = 5 5 2 EI (2 I) 4 EI JD M23 = (2(1) + 0) = 15 15 16 EI JD JD K 2 = M 21 + M23 = 15 M12JD = Symmetric 12 ß [θ2] M12JD 1 3 2 Equilibrium of Joint 2: -120 ΣM2 = 0 = -400 + K 2θ 2 θ 2 = 400 15 375 = 16 EI EI -90 Shear (kN) 90 4 1 Ax = 90 kN Member end moments are given by: M12 = 0 + θ 2 M12JD = 375 æç 4 EI ö÷ + = M21 = -M34 = 300 kN-m ç EI çè 5 ø÷ 375 æç 4 EI ö÷ M23 = -400 + ÷ = M23 = -M32 = -300 kN-m ç EI çè 15 ø÷ M21 = Ay = 120 kN Dy = 120 kN 300 375 æç 2 EI ö÷ ÷ = M12 = -M 43 = 150 kN-m ç EI çè 5 ø÷ Dx = 90 kN 3 2 300 -300 -300 -300 Moment (kN-m) 150 1 14-13 Copyright © 2018 McGraw-Hill Education. P8.19. P7.4. Derive the equations for slope and deflection for the beam in Figure P7.4. If E = 4 29,000 ksi, I = 50 in. I 3' D 100 kips B 5' 2I I A 8' P10.27 2 Unknowns θB ,Δ x @ D θ A = θC = θD = 0 Δ /12 ¢¢/ ft ΨAB = x = 0.01667Δ x 5¢ -Δ x /12 ¢¢/ ft ΨCB = = -0.02778Δ x Counterclockwise 3¢ 29,000 ksi (100 in 4) EI = = 20,138.9 ft 2 · k 144 in 2 / ft 2 2 EI (2θA + θB - 3(0.01667Δx)) = (0.4 θB - 0.02Δx)EI 5¢ 2 EI M BA = (2θB + θA -3(0.01667Δx)) = (0.8 θB - 0.02Δ x)EI 5¢ 2 EI M BC = (2θB + θC -3(-0.02778Δ x)) = (1.33 θB + 0.0556Δ x)EI 3¢ 2 EI MCB = (2θC + θB - 3(-0.02778Δ x)) = (0.667 θB + 0.0556Δ x)EI 3¢ 2 E (2 I) = θB EI M BD = (2θB + θD -0) 8¢ 2 E (2 I) = 0.5 θB EI M DB = (2θD +θB - 0) 8¢ M AB = JT. Leet, Chia-Ming Uang, Joel T.



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 (1.4)(42 ¢) + (-0.27)(30 ¢) + (0.1)(30 ¢) - (0.053)(12)ú êë 2 ûú 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 ûú 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 ûú 2 k E y = 62 é1 ù Gy = 2 k / ft ê (1.55)(34 ¢)ú êë 2 ûú k Gy = 52.7k é1 ù 1 1 MC = 2 k / ft ê (-8)(30 ¢) + (2.91)(30 ¢) - (1.59)(12 ¢)ú êë 2 ûú 2 2 k·ft MC = 171.8 ft · k é1 ù MG = 2 k / ft ê (-8)(30 ¢) + (4.36)(12)ú êë 2 ûú 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) + δAA X A = 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) δAA = 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/ft 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 + 3 24 (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 ly = 18 kips (a) Only tension diagonals effective. C 20 kips 15' A F D E 10 kips 15' 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(42) = = -10.67 N·m 12 12 1 3 æ 2I ö 1 = , K BD = çç ÷÷ = = 4 4 4 èç 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 C FEM 3.56 3.56 0 -7.11 3.56 D -10.67 FEM MCB 1 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 MC = -7.11 kN·m FEM D MBD 1/3 M BA = 3.56 kN·m MCB = 12.45 kN·m M BD = 3.56 kN·m D Ax = 0.3 kN·m ¬ C x = 0.3 kN ¬ Ay = 1.3 kN C y = 17.3 kN M A = 1.78 kN·m MC = 12.45 kN·m Dx = 0.6 kN Dy = 16 kN 11-16 Copyright © 2018 McGraw-Hill Education. And CT = 0.016 Reinf. (b) Replace the roller at support D in Figure P8.44 by a pin to prevent horizontal displacement of joint D, and repeat the analysis of the frame. too low. Evaluate the ordinates at 6-ft intervals of the influence line.

(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 P9.19.

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 δEV Area of Bars AB, BD, CD, 5 in2 Area all other Bars = 3 in2 E = 30,000 ksi Q2-System for δEH 8-16 Copyright © 2018 McGraw-Hill Education. T=? -wL2 Pab 2 -2(60)2 20(40)2 - 2 = 12 12 L (60)2 = -777.78 ft · k FEM AC = wL2 Pba 2 (60)2 20(40)(20)2 + 2 = + 12 12 L (60)2 = 688.89 ft · k FEMCA = + ΣMC = 0; RA 60 - 20(40) -120(30) + 538.89 RA = 64.35k M B = 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. Φ –125' 74.33' 167.38' 0.954P 0.025P –0.03P 0.043P I –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) = 0.876 k ¬ C y = 0.5(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 = 37. Draw the shear and moment curves and sketch the 2 deflected shape.

ΣFy = 12.5 -10 -10 + Dy = 0 Ax Ay Dy = 7.5 kips ΣM A = -7.5(40) + 10(20) + 10(40) + Dx (30) = 0 Dx = -10 kips ¬ 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 analysis. 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.

Continued Columns "ABC" & "KJI" Exact Analysis–Risa Softwear, (C.) Reactions & Deflected Shape Member Number & Joint Leters 13-43 Copyright © 2018 McGraw-Hill Education. B 10' 1 kip • ft A 12' P3.5 + ΣMA = 0; ΣFy = 0; + ΣFx = 0; 1ft · k - C y (12 ¢) = 0 C y = C y - Ay = 0 1 = 0.083k 12 Ay = 0.083k Ax = 0 3-6 Copyright © 2018 McGraw-Hill Education. Determine the axial forces, shear, and moment at D. P15.6. Continued Solve for Δ1 and Δ2 Δ1 = 0 é 0 ù é 2.08 ù é Δ1 ù ê ú ê ú ê ú = ê ú ê ú ê ú ëê 0 ûú ëê -80úû = K êë 0 .72úû êëΔ2 úû = - 80 K = -1.85 mm .72 Solve for Reactions Using Q3 = K 2 , ΔF Equation 15.34 é Q3 ù ê.64 -.48ú é 0 ù ê ú ê ú êQ ú ê -.48 -.36ú ê -80 ú ê 4 ú ê ú ê ú Kú êQ ú ê .48úú ê 0 .72 úú 5 ú ê ú = ê -.64 ú ê ú ê ú êQ6 ú ê -1 é ù ê ú ê Q8 ú ê ú ê ú 0 Q3 = 53.33 kN 40 Q4 = 40.00 kN 53.33 Q5 = -53.33 kN 0 53.33 80 Q6 = -40.00 kN Q7 = 0 Q8 = 0 40 15-14 Copyright © 2018 McGraw-Hill Education. Hint: If you have trouble computing bar forces, review K truss analysis in Example 4.6. A 3m 1 3m H G 3m L K J F 3m E 3m D 3m C B A 18 kN 60 kN P4.16 Establish Zero Bars (shown on sketch) IJ, HG, GF, KD, & LB + Compute Reactions: ΣM A = 0 60 ´ 9 - 60 ´6 - Bx 6 = 0 Bx = 30 kN + ΣFy = 0; Ay - 60 = 0 + ΣFx = 0; 30 + 60 - Ax = 0 Ay = 60 kN Ax = 90 kN ¬ Freebody below Section (1) + ΣM L = 0; - 30 ´3 - 60 ´3 + 60 ´ 9 - FEF 3 = 0 FEF = 30 kN Complete by Method of Joints Check Joint K ΣFx = 0 \ Joint in Equil. Determine (a) the absolute maximum values of shear and moment in the beam produced by the wheel loads and (b) the maximum value of moment when the middle wheel is positioned at the center of the beam in Figure P12.43. occurs w/hoist @ C = -11.5(60k) = 6901·k 12-30 Copyright © 2018 McGraw-Hill Education. Next compute the shear, axial load, and moment at points B and D, located at the quarter points. w = 4 kips/ft P5.44.

30 kips P8.19. FI = VCOLUMN MP 12.86 = = 2.14 k (Exact 2.2) æ COL ö÷ ç 6 çç L ÷ çè 2 ÷ø FBeam = 15k - 2.14 k = 12.86 M EXT = 15k (18¢) + 25k (6 ¢) = 420 k·ft M INT = 157.5V2 A = 420 V A = 2.67k 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 ¢) + 25k (20 ¢) + 25k (8¢) = 1180 ft · k M INT = 157.5 V3 A = 1180 First Floor Column Forces V3 A = 7.5k F1 = F4 = 3(7.5) = 22.5k (Exact 24.9k) F2 = F3 = 1.5(7.5) = 11.25k (Exact 0) Remaining member forces found by static w FBDs 13-29 Copyright © 2018 McGraw-Hill Education. 6 @ 10' = 60' h1 = 5' 27k H = cos θ 0.964 T = 28.02 kips 6-8 Copyright © 2018 McGraw-Hill Education. P4.34.

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 = 0.5kip/ft 25 kip • ft A 12' C 10' D 10' P5.1 ΣM A = -10(8) - 25(0.5 -10) · 25 + 20C y = 0 C y = 34 kips ΣFy = Ay - 8 - 0.5 -10 -15 + 34 = 0 Ay = -6 kips a) Cut beam with origin at A ΣFy = -6 - 8 - V = 0 V = -14 kips ΣM A = -M + (10) - 8 + x (-14) = 0 M = 80 -14 x kip · ft b) Cut beam with origin at D ΣFy = -0.5(10) -15 + V + 34 = 0 V = -14 kips ΣM D = M + (x2) · (-14) + 10(34) - 0.5(10)(5) + 25 = 0 M = 14x - 340 kip · ft 5-2 Copyright © 2018 McGraw-Hill Education. ΔCO + δCC 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 ´ 200 AE *Since bar AC is vertical and the force is known, there is no need to consider other bars. 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 "ABC" compression 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 C C D C 1 kip -10 E B D 1 kip -1 A -10 δEV = 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 1 58(12) 2 10 430 + 466 6 + (-10 + x)(-1.2x + 48x - 480) dx + 10(430) 6 + 2 EI EI EI 2 AE 10 0 = FQ,EH 17,000 25,800 26,880 696 61,180 696 (1728) + (12) = δEV = 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.

Δ AO = ΣFQ ΔLP = 2 A 4/3 (T) 1 in.2 AE 5 = 2 3 D B 3 in.2 ( ) 4/3 (T) 2 1 Δ AO = 2.578 in. EQ M AB = 2 K (θB - 3Ψ) - 2 KθB - 2 KΨ M BA = 2 K (2θB - 3Ψ) - 4 KθB - 6 KΨ M BC = 2(.6)K (2θB + θC) - 76.8 = 2.4 KθB + 1.2 KθC - 76.8 MCB = 2(.6)K (2θC + θB) + 19.2 = 2.4 KθC + 1.2 KθB + 19.2 MCD = 2 K (2θC - 3Ψ) = 4 KθC - 6 KΨ M DC = 2 K (θC - 3Ψ) = 2 KθC - 6 KΨ æ 3.13 ö÷ = 2K - 6 K çç ÷÷ + K èç 6 ø =0 12.2 kip · ft æ 3.13 ö÷ (15.49) = 4K - 6 K çç ÷÷ çèK 6 ø + K M AB = 43.18 kip · ft (2) Equilibrium Equations Joint B: M BA + M BC = 0 (a) Joint C: MCB + MCD = 0 (b) Shear EQ: M BA + M AB MCD + M DC + = 0 12 12 æ 3.13 ö÷ + (-2.97) - 6 K çç ÷ çè K ø÷ = -30.66 kip · ft æ 3.13 ö÷ + (-2.97) = 2K - 6 K çç ÷ ÷ çè K 6 ø M DC = 4 K DC = -24.72 kip · ft 10-32 Copyright © 2018 McGraw-Hill Education. 3' w P6.10. Use E = 200 GPa for all members; for beams and interior columns use 2 4 A = 10,000 mm and I = 50 × 106 mm ; for 2 exterior columns use A = 5000 mm and I = 25 × 106 mm . W = 3 kips/ft A C B 4' 8' 4' P10.8 Unknown: θC θ A = θB Unloaded cantilever Thus M AB = M BA = θD = 0 No vertical displacement @ A thus θB = 0 2 WL2 (3 k ft)(8¢) = = 16 ft · k 12 12 WL W (8¢) = = = W 8 8 FEM BC / CB = FEMCD / DC 2 EI (θC) -16 ft·k = 0.25θC EI -16 8 2 EI MCB = (2θC) + 16ft·k = 0.5θC EI + 16 8 2 EI MCD = (2θC) - W = 0.5θC EI - W 8 2 EI = 0.25θC EI + W MCD = (θC) + W 8 M BC = + ΣM B = 0; 0.25θC EI -16 = 0 θC = 64 EI + ΣMC = 0 MCB + MCD = 0 (0.5θC EI + 16) + (0.5θC EI - W) = 0 64 + 16 - W = 0 W = 80 k 10-9 Copyright © 2018 McGraw-Hill Education. 20 D kips C 20 kips 15' A B 20' P13.13 Assuming the 40 kips of shear is equally distributed to the diagonal truss members via axial force. 20 ft 20 kips D C 25 15 25 20 20 kips D 15 (T) 25 C 0 15 15 20 20 kips -15 (C) 15 ft 30 (T) By = 30 kips 13-19 Copyright © 2018 McGraw-Hill Education. , I = 7800 in. The area of all 2 bars = 2500 mm , and E = 200 GPa. C 240 kN B 8m A D 6m P8.3 A = 2500 mm2, E = 200 GPa Q-System for δCH δCV Q-System for δCV δCH BAR Fp FQ1 FQ 2 L FpFQL FpFQL AE 200 4 3 0 8 6400 3 0 0 BC 240 1 0 6 1440 0 CD 0 0 –1 8 0 0 AD 0 0 0 8 0 0 BD –400 0 10 20000 3 0 Σ = 10,240 Σ=0 - 5 3 δCH Σ10,240 ´103 AE 2500 ´10-6 (200 ´10 4) δCV = 20.48mm Q ΣFQ1 FP L = δCV ΣQ2 δCV ΣFQ2 FP L AE = 0 8-42 Copyright © 2018 McGraw-Hill Education. Given: EI is constant, 2 4 E = 30,000 kips/in.

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' 1 12" 9 2 10 Section 1-1 11 36 kips 36 kips 36 kips 36 kips 36 kips 36 kips 36 kips 36 kips 36 kips 18 @ 36' = 360' P6.32 (a) Uniformly distributed dead load only Reactions (kips), Deflections (in), and Deflected Shape 6-35 Copyright © 2018 McGraw-Hill Education. FQ2 FP P8.16. Compute the reactions at points A and D. L.L. Tension Bar KJ (2 Trusses) 2 = 41.01 kips 2 41.01 = 20.51 kips Each Truss = 2 FAL = 24 k · 2 + 10 k · G 20' Influence Lines Max. 1 36 kips P15.3. Continued Member 3-0-0 15 = cos f = sin f = 1 15 15 AE 2(30,000) = = 333.3 L 15(12) 3 4 é0 0 é0 1 é é0 0 é0 -1 é0 5 6 3 4 é0 0 1 0 0 ú é0 0 -1ú ú 4 1.67 é0 200 éê 0 0úú 5 0 é0 ú 6 0 1ú é0 éê 0 -1.67 ú 5 6 0 0ú 3 ú 0 -1.67úú 4 0ú 5 0 1.67úú 6 ú Structure Stiffness Matrix Q - KΔ 1 4 3 4 5 6 é0 ú 6 é1.89 0.48 -0.64 -0.48 -1.25 0ú 1 é Δ1 ù é ú ê é-.36ú é 0.48 ú é Δ3 ú é 0.36 0.48 0.36 0 0 é ú é é ú é 2ú é 0 é0.64 0.48 0 0ú ú 3 éê 0 0 ú é é é1.67úú 4 éê 0 é é5 é é-1.67 0 -1.67ú 0.1.25 0ú 1.67úú 6 é éú Rearramge Matrix to Permit Solution by Equation 15.30 1 2 4 3 5 6 é1.89 1é 0 ú é0.48 -0.48 -0.64 -1.25 0ú é Δ1 ú 0 ú é -.36ú é ú é ... 0.48 -0.48 -0.36 2.03 0 -1.67 ... 0.48 0.64 ... ú Solve for D1, D2, D4, Using QF = K21DF (Equation 15.33) Solution Gives: Δ1 = 0.192 k Δ2 = -0.865 = Δ x1 k Δ4 = -0.108 = Δ y1 k Solve for Support Reactions Using Equation 15.34 Q3 = K21DF Q3 = 200[0.24] = 48k Q5 = 200[-0.24] = -48k ¬ Q6 = 200[0.18] = 36 k 15-7 Copyright © 2018 McGraw-Hill Education. 30 kips P10.11. Gilbert SOLUTIONS MANUAL CHAPTER 9: ANALYSIS OF INDETERMINATE STRUCTURES BY THE FLEXIBILITY METHOD 9-1 Copyright © 2018 McGraw-Hill Education.

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 + ΣFx = 0; + ΣM E = 0; + ΣFy = 0; E x - 30 = 0 E x = 30 kN 30 (40) + Dy (15) = 0 Dy = 80 k 80 - 40 - E y = 0 E y = 40 k Isolate Top Truss + ΣMC = 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 FBD at Section 1 M1 6 ft ΣM C = 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 σ1 1 V1 Centroidal Axis of Columns 0 = 30(6) - F1 (12.5) - F2 (12.5) F1 = A1σ1 = F2 180 F1 = F2 = (12.5) F1 = FCB = 7.2 kips F2 = FDE = 7.2 kips C M1 F1 σ2 12.5 ft Column Areas FBD at Section 2 Axial Stresses Through Section 2 ΣM Z = 0 = 30(28) - F1 (12.5) - F2 (12.5) 840 F1 = A1σ1 = F2 = (2(12.5) F1 = FAB = 33.6 kips F2 = FEF = 33.6 kips 30 kips VCD V1 = 15 kips FCB ΣM POI = 0 = MCD - VCD (12.5) ΣM D VDE = 7.2 kip = 0 = 90 - 7.2(25) + M DC (top half) ΣFy = 0 = 30 - FCD -15 FCD = 15 kips ΣM B = 0 = -90 + 15(12) - M BC (whole col.) ΣFy = 0 = FBC - FBC + VBE M BC = 90 kip-ft M DC = 90 kip-ft From Joint C: VCB = 15 kip POI 6 ft VDC = -7.2 VCD = -7.2 ΣFy = 0 = 7.2 + VDC ΣM POI = 0 = -90 + VCB (6) 6 ft FDC = 15 kips 12.5 ft Equilibrium of Column BC: VCB = 15 kips MBC = 90 kip-ft Equilibrium of CD (whole beam) MCD = 90 kip-ft MCB = 90 kip-ft VBE = 26.4 kips VBC = 15 kips MCB = 90 kip-ft Equilibrium of BE: FBC = 7.2 kips ΣM POI = 0 = M BE - 26.4(12.5) FBC = 7.2 kips MBC = 90 kip-ft V1 = 15 kips MBE = 330 kip-ft B MEB = 330 kip-ft E ΣM E = 0 = 330 - M EB - 26.4(25) (whole beam) M EB = 330 kip-ft B 12.5 ft MBA = 240 VBE = 26.4 kips (left half) M BE = 330 kip-ft POI A1 σ2 POI 12.5 ft M1 ΣFy = 0 = VCD - FCD C MCD = M1 = 90 kip-ft C FCD C Equilibrium of CD (left half): FCB = 7.2 kips F2 12.5 ft Equilibrium of Joint C: VCD = 7.5 kips A1 σ1 V2 12.5 ft ΣFy = 0 = -26.4 + VEB VEB = 26.4 V2 = 15 kips VEB = 26.4 kips FBA = 33.6 kips 13-26 Copyright © 2018 McGraw-Hill Education. P6.26. What is the percent increase in total volume of material used? (b) Assuming that the dead load of the arch and floor system can be represented by a uniform load of 4.8 kip/ft, determine the forces in bars CM and ML produced by the dead load. 20 kN 20 kN B 5m D 5m G H A 24 kN F E 4@5m P4.32 Freebody Left of Section (2) + Freebody Left of Section (1) Compute FAB + ΣM" = 0; 32 kN(3m) - FABX (20m) = 0 æ 20 ö ΣMG = 0; 32 kN (10 m) - 20(5 m) - FBCX çç ÷÷ = 0 çè 3 ø÷ FBCX = 33 kN compression & \ FBCY = 33 kN Final Results, Symmetrical about Centerline : FABX = 48 kN compress. Between B and D Evaluate Moments at 4 m Intervals 12-49 Copyright © 2018 McGraw-Hill Education. Slab + Stringers art 6 ñ R = (3k / ft)çç ÷÷ + 1.5 = 24 k èç 2 ø VCD = 0 12-19 Copyright © 2018 McGraw-Hill Education. tubes with 2 4 A = 2.59 in.

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 arch. (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: + ΣM 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 + ΣFx = 0 = 30.4 - Ax \ Ax = 30.4 kips + ΣFy = 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 " : ΣFy = 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 ΣM D = 0; - 30 k (36 ¢) -18k (24 ¢) -12 k (9 ¢) + FEI (12 ¢) = 0 FEI = 135k compression Bar "EF" : + ΣM I = 0; -30 k (24 ¢) -18k (12 ¢) -12 k (9 ¢) + FEDY (12 ¢) = 0 FEDY = 87k FEDX 87k = 4¢ 3¢ + ΣM H = 0; FED = 145k tension Thus FEF (9¢) - 30 k (12 ¢) = 0 FEF = 40 k tension FEDX = 116 k Freebody Joint I: ΣFy = 0; + - FEI + FIJ = 0 FIJ = 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 2.4 kips/ft 73.3 kip-ft B 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 = (2θB + θC) ú 4 ú 2 EI ú MCB = (2 θC + θB) ú 4 ú (3) (4) EI (-2 ´ 0.011767 + 0.005884) 2 200 ´ 25 (-0.019456) = -44.125 kN · m = 2 M BA = 44.125 kN · m M BC = M AB = M AB = -77.94 kN.m Analysis of members + ΣM B = 0 = C y 4 - 44.125 C y = 11.031 kN VBE = 11.031 kN + ΣM A = 0 = 44.125 + 100 ´3 - 77.94 - VBA 6 = 0 VBA = 44.364 kN ¬ Joint Equilibrium, EQs. Joint C 2 EI (2 θC + θB) 4 θ θC = - B 2 MCB = 0 = (2) ΣFx = 0 -VBA + 100 - VAB = 0 VAB = 100 - 44.364 55.636 kN ¬ Reactions and Member end Moments 10-19 Copyright © 2018 McGraw-Hill Education. (f) hinge P3.33. P5.56. 6Dx = 0.172 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 E P9.35. Express your answer in terms of E, I, L, and M. 4 ft 1 F2.6. The uniformly distributed live load on the 2 floor plan in Figure P2.4 is 60 lb/ft . P14.8. The structure in Figure P14.8 is composed of a beam supported by two struts at the cantilever end.

G 5' P12.55 (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 L K 30 kN 30 kN 30 kN 15 kN J C I D H E B 3@8m 2@6m P6.31 Reactions: ΣM A = 0 = 30(8 + 16 + 24 + 30) + 15´36 - Fy 36 - 4 Fx (1) + ΣM D = 0 = 30 ´6 + 15´12 + Fx 8 - Fy 12 = 0 Solving Eg ¢s 1& 2 8m F A + G (2) Fx = 64.3 kN , Fy = 72.86 kN ΣFx = 0; Ax = Fx = 64.3 kN ΣFy = 0; Ay = 77.14 kN At Joint E: + ΣM E = 0; -(72.86 -15)6 + 64.3 yE = 0 yE = 5.4m Joint B: + ΣM B = 0; 77.14(8) - 64.3(yB) -15(8) = 0 yB = 7.73m Joint C: + ΣMC = 0; (77.14 -15)16 - (64.3) yC - 30(8) = 0 yC = 11.7m 6-34 Copyright © 2018 McGraw-Hill Education. For all bars the 2 area = 2 in. Using an approximate analysis of the Vierendeel truss in Figure P13.16, determine the moments and axial forces acting on free bodies of members AB, BC, IB, and HC. Determine all reactions for the frame in 4 Figure P9.40 given IAB = 600 in. For the arch rib to be funicular for the dead loads shown, establish the elevation of the lower chord joints B, C, and E. Determine the reactions at supports A and E in Figure P9.48. P16.1. Continued é 0.33 0.067ù é ... 0.133 -0.067ù ... 83.33ù ... ú -150ú ... 0.33ú éë-150ú ... -1388.9 ... 0.067 ... ú Span "BC" end Moments: ¢ + M BC ¢¢ = -150 - 22.2 M BC = M BC ¢ -172 ft · k Where ¢ = -150 ft · k M BC and ¢¢ = (θ1 (0.133EI) + θ2 (0.067 EI) = -22.2 ft · k M BC ¢ + MCB ¢¢ = +150 + (θ1 (0.067 EI) + θ2 (0.133EI)) = 0 MCB = MCB Span "AB" end Moments: ¢ + M AB ¢¢ = (-66.67) + (θ1 (0.1EI) + θ2 (0)) M AB = M AB = -13.9 ft · k ¢ + M BA ¢¢ = (66.67) + (θ1 (0.2 EI) + θ2 (0)) M BA = M BA = 172.2 ft · k FBD: Span "BC" FBD: Span "AB" + ΣM A = 0; -13.9 + 172.2 - VBA (20) + 2 (20)(10) = 0 VBA = 27.92 k & VAB = 12.08K + ΣM B = 0; -172.2 + 2 (30)(15) - VCB (30) = 0 VCB = 24.26 K & VBC = 35.74 K Joint B: By = 27.92 + 35.74 = 63.66 K 16-3 Copyright © 2018 McGraw-Hill Education. The midspan girder CD is simply supported on the cantilevered ends of the left and right girders. unloaded frame (b) P5.57 X (x) 26.395 -31.395 -5 Y (k) 77.75 82.25 160 MZ (k-ft) 0 0 0 Deflected Shape (inverted) Vertical deflection at girder midspan = 1.79 in. Shear Diagram (kips); Moment Diagram (kip-ft); (b) Vertical deflection at midspan of girder = 1.179 in. w = 1.5 kips/ft P6.23. For the arch and loading in Figure P6.18, compute the reactions and determine the height of each point. 0.003 rad w = 4 kips/ft A 0.3" 8' C B 2' P9.8 Using Moment-Area determine Δ BO RA + RB - 4 ´10 = 0 RA = 40 - 21.15 = 18.85 M A = 4 ´10 ´5 - 21.15´8 M A = 30.8 kip · ft 10 200 ´(7.2 - 2) 3 EI 1æ 80 ö 200 ´ = ´ (12)3 + 4 3664 3664 (1728) = = ´ = 0.0873 ft ´ 30,000 240 EI Δ BO = t BA = 12 = 240 in 4 , E = 30,000 kips / in 2 Select RB as the Redundant P3.1 Δ B3 = 512(1728) = = 3EI 3EI 3´39000 ´ 240 = 0.041k δBB = 0.8555 δBB δROT = 0.003(8 ´12) = 0.288¢¢ Gap: Opens = .879 + 0.288 = 1.167k Support Move = -.3 Gap = 0.867k RB = 0.867 = 21.15 kips δBB = .041 9-9 Copyright © 2018 McGraw-Hill Education. Given: E = 29,000 ksi and I = 100 in. 40 kips P6.30. "E": + ΣFx = 0; - 35 kN + 55 - FCX = 0 FCX = 20 kN C FCY = FCX (2) = 8 kN 5 LT P. 2.4 kips P4.46. GILBERT SOLUTIONS MANUAL CHAPTER 15: MATRIX ANALYSIS OF TRUSSES BY THE DIRECT STIFFNESS METHOD 15-1 Copyright © 2018 McGraw-Hill Education. Compare your results with those in part (a), and briefly discuss differences in behavior with respect to the magnitude of deflections and moments. 5-40 Copyright © 2018 McGraw-Hill Education. E = 200 GPa, I = 40 × 10 mm . E4.42. Use moment at support A as the 2 redundant. 10' A 10' C B 8' D E 6' 6' P8.39 P-and Q-System L dx + Σ ò MQ M P AE EI 10 2 é -20)2 (10 ´12) ù é (12)2 (12 ´12) ù ú+ê ú + 2 (32 x) dx = ΣFP FQ 64 δBV 96000 20736 2048 êë x 3 ùú (1728) + + E EI EI êë 3 úû 0 10 = δBV = 0.59 in.

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 the computations using an origin at point D. A P12.56. + ΣFx = 0; Ax - 6.25 = 0 Ax = 6.25 kips + ΣFy = 0; Ay - 40 + 20 = 0 Ay = 20 kips + ΣMA = 0; 40 ´10 - 20 ´20 - MA = 0 MA = 0 kip · ft 3-26 Copyright © 2018 McGraw-Hill Education. 5-15 Copyright © 2018 McGraw-Hill Education. 2 3 6 @ 6.67' = 40' C4 A B2 G1 B4 B3 2 @ 10' = 20' C2 B G4 G3 G2 B1 C 5 @ 8' = 40' C3 40' C1 20' P2.4 5 ft (a) Method 1: AT 20 AT 200 ft 8 ft Method 2: AT 200 4 2 5 2 B3 6.67 ft 6.66 ft 6.67 ft 2 B4 (b) Method 1: AT 6.67 20 AT 133.4 ft Method 2: AT 5 ft 2 (f) AT 10 10 ; AT 100 ft 2 36 ft G4 2 AT,C4 B4 2 (e) AT 30 20 ; 3.33 ft 2 2 Method 2: AT 493.4 2 AT 488.5 ft 3.33 ft Right Side 2 (d) Method 1: AT 4 40 33.33(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 ... 0.07899 EI K 21 = 2 = 0.16667EI 6 ... EI K 12 = 2 = 0.16667EI K 22 = + + EI 6 12 6 2 EI -6 EI K 23 = 2 = 0.16667 EI 13 13 K 23 = -0.09375EI 12 8 FEM23 = -FEM32 = K11 K13 B C 30 kip-ft 30 kip-ft -6 EI K 33 = ... = 0.83333EI 12 8 K 31 = K23 K21 K12 K20 kips K31 K32 K22 é 30.417 -5.834 é 0.07899 0.16667 -0.09375ú 4.589é-1.é ù 0.16667 1.06667ù , inverting yields: K = ê-5.834 2.154 -1.087úú é-0.09375 0.16667 é 4.589 -1.087 0.83333úû 1.934 ... 16-7 Copyright © 2018 McGraw-Hill Education.

(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-ft 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 K ME (kip-ft) 19.2 14.4 9.6 4.8 ME (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 2º, 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 θ = θE + ΔθE - D 60 60 (3) (1.5) + EI EI 2 EI 180 θ=EI Use Minus Sign. Ay = 45.15 kips Deflection at B by: 10 kips Reactions: Ay = 0.272(45.15) = Ay = 12.3 kips 12.3 kips Ay = ΔKTOT 15 = Δ(55.15) By = 0.272(10) = By = 2.7 kips Δ = 0.272 in.

Investigate Joint I. C B 4m A 3m 4m P8.31 P-System E = 200 GPC I = 80 ´106 mm 4 Spatial structure DCx 1kN ·DCx = Σ ò MQ M P dx EI 4 DCx = ò (0.5714 x)(25.71x) 0 dx EI 5 + ò (0.4571x)(20.571x) 0 4 dy EI 5 dx dy + ò 9.404 x 2 EI EI 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 m = 44.1 mm = 0.0441 m = 14.691x 2 1kN · m θB = Σ ò MQ M P Compute θB dx EI 4 θB = ò (-0.1429 x)(25.71x) 0 dx EI 5 + ò (0.0857 x)(20.574 x) 0 4 dx EI 5 dx dx + ò (1.763x 2) EI 0 -0.67 -78.379 73.458 = + EI EI -4.921· kN 2 m 3 θB = 200 ´106 ´80 ´106 10-12 = -0.00031 Radians 8-37 Copyright © 2018 McGraw-Hill Education. wL2 3.6(20)2 = = 120 ft · k 12 12 wL2 3.6(24)2 = = = 172 ft · k 12 12 FEM HG = FEMGF Joint H 1728(1.5) = 216 12 1728 K HA = = 115.2 15 12000 K HG = = 600 20 ΣK I = 931.2 K IH = D.F.IH = 0.232 D.F .HA = 0.124 D.F .HG = 0.644 Joint G 1728 = 144 12 1728 KGB = = 115.2 15 12000 KGH = = 600 20 1 12000 KGF = = 250 2 24 ΣK I = 1109.2 K IG = D.F.IG = 0.13 D.F.GB = 0.10 D.F .GH = 0.54 D.F.GF = 0.23 13-22 Copyright © 2018 McGraw-Hill Education. Analyze the beam in Figure P11.24 by moment distribution. E 1 (T) L (ft.) Cy 1 1 0 0 0 0 0 0 6 6 10 10 8 8 10 6 8 6P = ΣFQ ΣQδP = ΣFQ D B FQ (kips) Cx 0.75 0.75 –1.25 –1.25 1 1 0 0 0 C 1 (T) 0 D B A C 1 kip Cx Q-System (kips) 0.75 (T) 24 kips 1 kip P-System (kips) 36 (T) 48 kips ΣQδP = ΣFQ FP L AE 0 0 A FQ FP L F (in.) AE Cx 0.034 0.034 0.155 0.194 0.099 0.099 0 0 Cy 0.045 0.045 0 0 0 0 0 0 0.615 0.089 FP L AE (δCy) · (1)(0) ·(0)(6x) = 0.089 δCy = 0.089 in. B 3V D A E 20' 20' P9.24 Selecting Cx as the redundant, the compatibility equation is: (ΔCS + ΔCO) + δCC XC = ΔC δCC = Σ FQ2 L AE Q-System (kips) 25(12) 3 30(12) + 2( ) (45) 5(30,000) 4 5(30,000) 2 2 =2 - B δCC = 0.0076 in. (a) Draw the influence lines for the bar forces in members HC, HG, and CD of the truss shown in Figure P12.32. P10.7. Analyze the beam in Figure P10.7. Draw the shear and moment curves. 12 kN/m P13.3. Assume values for member end moments and compute all reactions in Figure P13.3 based on your assumption. C 10 kN D E B 6m A 3m 3m 6m P5.24 Reactions + ΣFx = 0 : RA y = 0 + ΣM A = 0 : 10 ´3 + 24(12) - R D 12 = 0 RD = 26.5 kN + ΣFy = 0 : RAY - 34 + 26.5 = 0 RAY = 7.5kN 5-26 Copyright © 2018 McGraw-Hill Education. Classify the structures in Figure P3.33. Then the uniform load on all girders extends to the centerlines of the columns. A timber member supported by three steel links to a concrete frame has to carry the loads shown in Figure P3.35. Restraint Remove Support F \ Indet 4º (e) 5 Reactions 3 Eq. of Equil. What moments are created in the frame in Figure P11.26 by a temperature change of +80°F in girder ABC?

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

For all 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. + ΣM D = 0; - 20 k (15¢) + FABY (30 ¢) = 0 FABY = 10 compression æ15 ö FABX = çç ÷÷ FABY = 15k çè10 ÷ø k Freebody Left of Section (2): ΣFy = 0; + FDJY - 20 k tension 15 ö FDJX = çç ÷÷ FDJY = 15k ç è 20 ÷ø FCD = FAB = (10)2 + (15)2 = 18k compr. P6.31.

E P4.15.

4m 3m B C 3m F 4m 60 kN E 3m A D P4.19 Joint C Reactions: + ΣM A = 0; 60´3 - Dy 10 = 0 FBC C Dy = 18 kN FFC + 18 ΣFy = 0; Ay - 60 + 18 = 0 FCD 42 Ay = 42 kN + ΣFx = 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 BC. 5 in.2 5 in.2 20' 4 in.2 D 2 in.2 2 in.2 A C P = 40 kips 15' 15' P8.14 Q-System P-System L A æ ö çç ΣF F L ÷÷ ÷ ... 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 A = 800 mm .

Inversely Proportional to L FL AE AE (ΔL) F= L FAB 12 = FBC 18 12 \ FAB = FBC 18 ΔL = 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 ΣM A = 30(10) + 20(20) + 50(30) + 70(40) + 90(50) - Ey (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 I 6' F H J H 6' D C 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 = 30 k (C) 3 FBJ = X BC = 24 k (T) Joint C + ΣFy = 0: 18 + 18 -12 - YCD = 0 YCD = 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 P6.12 Use General Cable Theory: H · h = M = 16 kN · m at "C " H = 16 = 8 kN. 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 kN · m 12 12 FEM23 = 72 kN · m FEM23 = - Joint 2 Moments Produced by A 1 Radian Rotation of Joint 2, Multiplied by θ2 M12JD = 2 EI EI (-1) = 4 2 M21JD = 2 EI (2(-1)) = -El 4 M23JD = 2 E (2 1)(-1) El = 12 3 D 2 K = 2 M21JD + M23JD = - El - El 3 5 K 2 = - El 3 14-9 Copyright © 2018 McGraw-Hill Education. 12 P13.13. 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 , Δ 20 Δ ΨCD = ΨDC = 15 40(20) PL == -100 k·ft FEM BC = 8 8 FEMCB = 100 k·ft ΨAB = ΨBA = Member end Moments (a) Sum of moments at B about the arch ...

Draw the shear and moment curves for each member of the rigid frame in Figure P5.41. (NG) (c) Case 2 Unbraced Frame with Shear Joints 7-55 Copyright © 2018 McGraw-Hill Education. E = 3000 kips/in. 4 2 Given: IAC = 340 in. (b) To restore joint B to its initial position in the horizontal direction, how much must bar AB be shortened? (b) Repeat the analysis of the arch if a single 48-kip vertical load acts 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 McGraw-Hill Education. (a) Analyze the structure for full live and dead loads on both spans, that is, establish the shear, moment, and axial load diagrams for the girders, the forces in the cables, and the maximum deflection of the girders. See Figure P10.2.

Clamp joints, Compute Fixed-End Moments ... Determine the total wind force on the roof, wall and building. Draw the influence lines for reactions at A, B, and C, and the shear and moment at points B and D. The influence line for the horizontal reaction at A and the horizontal deflection at B and at the midspan of member CD in Figure P8.38. What do you observe? P = 66 kips P8.39. 12-52 Copyright © 2018 McGraw-Hill Education. Compute Total Wind Force (kips) 50.7 + 48.02 ... Assume that the 120-kip load is removed from the truss in Figure P9.25, compute the reactions and bar forces if the temperature of bars AB and BC increases 60°F; the coefficient of temperature expansion = 6.5 × 10 (in./in.)/°F. P12.59. P6.5. Compute the support reactions and the maximum tension in the main cable in Figure P6.5. The hangers can be assumed to provide a simple support for the roadway girders. Compute the initial elastic deflection at midspan of the girder in Figure P8.44, given that the support at B is a roller. (b) Evaluate the moment MA and the reaction RA at end A produced by the loaded one-carrier when its 30-kip rear wheel is positioned at point 2. ...

Consider that the frame is laterally braced, and that the deflected shape is 2 D A B C 6' 8 kips 12 kips P5.5. Determine the support reactions and draw the shear and moment diagrams for each member. If the settlements are of the order 0.5 in., what will be the effect on the moments? Case 3 Braced Frame with Shear Joints (a) Moment Diagram (k·ft) Shear Diagram (kips) 7-57 Copyright © 2018 McGraw-Hill Education. Consider columns 2 4 2 A = 13.1 in. 4-45 Copyright © 2018 McGraw-Hill Education. Determine the reactions and member forces. Draw the shear and moment diagrams for the girder of the frame. Compute the reactions and draw the shear and moment curves. Determine the moment that must be applied at joint A to produce a 1-in. vertical displacement at joint B. ...

Assuming the container is water-tight, will the tsunami wave be capable of carrying away the container as debris? Determine the axial forces in all bars. Draw the influence lines for the force in member AB, and the vertical reactions at supports. Calculate the absolute maximum moment produced in the girder by the HS20 truck. ... 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) hinge (c) hinge 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 truss shown in Figure 4 2 P8.30. horizontally and connected to the pin support 4. Compute the horizontal displacement of joint B. = 0.444 91 1.000 ΣK ... 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. Consider column bases are fixed. 1.35 kips 30 kips 35 kips hinge H G 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. Determine the reactions at A, B, C, and D and calculate the force in the brace. Equations: 0.814 c2 - 3.75 + 1.525 X A = 0 ΔA02 = -1.5 + XA = RA = 1.925k. Remaining Reactions by Statics RC = 10.89k. Released Structure w/Support "B" Settled. 3' P12.5. (a) Draw the influence lines for reactions RB, RD, and RF of the beam in Figure P12.5 and the shear and moment diagrams.

SOLUTIONS MANUAL CHAPTER 14: INTRODUCTION TO THE GENERAL STIFFNESS METHOD 14-1 Copyright © 2018 McGraw-Hill Education. ... (a) Calculate the reactions at support A. w = 3.5 kips/ft A B 12' 12' P11.1 Joint B Distribution Factors ... Live Load 700 lb/ft (1.33) 2 Total Girder Load (D + L) per foot: Live x Impact Factor: = 0.47 kip/ft ... Analysis of Members 2 El MCA = ... (a) Compute the change in length of member AB. The joint rotations; (b) the moments and reactions at supports A and B. Use the General Slope Deflection Method. Using the RISA-2D, compare the results. 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 24-kip loads? Load moves along the span, compute the maximum ...

SOLUTIONS MANUAL CHAPTER 6: ANALYSIS OF INDETERMINATE BEAMS AND FRAMES BY THE MOMENT DISTRIBUTION METHOD 11-1 Copyright © 2018 McGraw-Hill Education. ... Member End Moments: ... 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.

(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 ... (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. Determine the uniform dead load in kips per linear foot acting on the beam. w = 3 kips/ft P9.4. Determine the reactions and draw the shear and moment curves for the beam in Figure P12.4. (b) Assuming that the span is loaded with a 1.2 kips/ft uniform load of variable length, determine the maximum positive and negative values of the reactions.

40 kips P5.50. hinge hinge (b) (c) hinge hinge (d) (e) P3.33 (a) Indeterminate 1° (b) Indeterminate 3° (c) Unstable (R < 3 + C) (d) Unstable (e) Indeterminate 4° 3-34 Copyright © 2018 McGraw-Hill Education. Draw the shear and moment curves of the frame in Figure P5.49. P13.7. Continued Check Results using Moment Distribution ...

M P8.18. , are used to support the deck at the third points of each 120 ft span. Load Case 2: ... 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. ... Determine the forces in all members of the truss and reactions. ... Freebody Left of Section (1) Joint A. Compute FAD: ... Determine the influence line for axial load in column AB and indicate the spans that should be loaded to maximize the axial load. ASCE Approximate Fundamental Period: T = 0.1N N =5 T = 0.5seconds 0.3'6750 Ve = 810 kips ... 5(5) ... Determine the forces produced in each member and the components of reactions. ...

Consider both axial and bending deformations. Beam [1]: M [] @ 0.50 FEM = 0.50 wL2 = 60 ft k 12 V[] @ 0.45 ... Determine the forces in the truss. (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? ... Continued Member Numbers & Joint Letters Exact Analysis - Risa Software Results, (C) Reactions X Force Y Force Moment (k-ft) ... Determine the maximum positive and negative moment ... Member Properties (w) Area ... Consider the truss in Figure P8.44; the truss supports the roof ... Draw shear and moment curves ...

Beam Figure P5.7b. In addition to the load at joint C, the temperature of member BD is increased 80°F. In addition to the 60-kN load, the support at A settles 0.02 m. ... (b) Determine the reactions and 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. ... The building, located on level flat ground, is classified as stiff because its natural period is less than 1 s. ... Determine the axial forces in all links. 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) ... P10.33. B A K = 40 kN/m C 10 m P9.21 Select RB as the Redundant. ... Sketch the deflected shape and find the reactions. ... Assume I = 0.5IG. ... Determine the stresses and redundants. ... Determine all reactions and draw the shear and moment diagrams ...

Assume 1 = 0.5IG. ... Treat the concrete slab and composite beam as the reference structure ... Determine the force in each bar in the truss of Figure P4.1 ... Use the exact solution ... Determine the reactions and sketch the deflected shape. Draw the shear and moment diagrams for each member ... Compute the vertical deflection of joint E. ... Compute the force in the cable and draw the shear and moment curves for the beam. ... Determine the influence lines ... The analysis of the frame ... Determine all reactions and member end moments ...

Education. Indeterminate, stable Determinate, stable 4-3 Copyright © 2018 McGraw-Hill Education. (b) Compute the vertical deflection of the spring produced by a vertical load of 15 kips. A P6.11. 6' B A 8' C 12' 6' P16.5 Stiffness Matrix Apply Unit Rotation @ B : θ1 = 1 Rad, & clamp @ C: c =M AB + Joint B: ΣM B = 0; K11 - 0.4 EI - 0.33EI = 0 + Joint C: ΣMC = 0; K12 - 0.2 EI = 0 \ K11 = 0.733EI P1 B P1 C = 120 ft · k = M BA B \ K 21 = 0.2 EI Apply Unit Rotation @ C : θ2 = 1 Rad, & clamp @ B: + Joint B: ΣM B = 0 ; + joint C: ΣM C = 0; K12 - 0.2 EI = 0 \ K12 = 0.2 EI K22 - 0.4 EI = 0 \ K22 = 0.4 EI Stiffness Matrix: é 0.733 0.2 ù ê K = EI êë 0.2 0.4 úû 16-9 Copyright © 2018 McGraw-Hill Education. 5m P13.9. Continued Check Results using Moment Distribution EI Constant I = 0.2 DFBA = 0.941 5m 1 ... Copyright © 2018 McGraw-Hill Education. Accuracy of the approximate method for end moments (M A , M B) in columns and girder is 97.2% 13-14 Copyright © 2018 McGraw-Hill Education. A B 10' 12' C D 12' 10' P15.16 ΣM A = -200 - 170 -100 - 44 By = 0 By = -2.95 kips ΣFy = 0 Ay - 2.95 = 0 Ay = 2.95 kips E By Shear Moment Deflect. Using the Müller-Breslau principle, draw the influence lines for the reactions and internal forces noted below each structure. • 3/8 in. Gilbert SOLUTIONS MANUAL CHAPTER 6: CABLES 6-1 Copyright © 2018 McGraw-Hill Education. Draw the influence lines for bar forces in members ML, BC, CD, EJ, DJ, and FH of the cantilever truss in Figure P12.35 if the live load is applied through the lower chord panel points. B C D 10' 24 kips G F E 10' 24 kips 24 kips H 10' 10' H' P4.29 Solve by Method of Joints 45 Copyright © 2018 McGraw-Hill Education. D F2.20. B 2' C Bm A E 5m 12 m P9.46 = 1 2 at© 4208 (8)(8)(17) + ... (8)(8)(8)+ ... ΣM3 = 0 + ... EI = 3 ... Compatibility Equations: ∆BE + 6ΣE E + ... V 1 = 0.7 kips A F Solver Σ x as the Redundant By statics: Ax = 16.8 kN(), Ay = 7 0.6 kN(), EY = 29.4 kN() 9-52 Copyright © 2018 McGraw-Hill Education. B C 12' 18' D 18' E 12' RB, VB (to left of support B),VB (to right of support B), MC, and VC P12.12 18 ft 12 ft A 18 ft B C 12 ft D E 1.33 1 0 0.5 0 By (kips) -0.33 VB (kips) (Left of B) 0 0 -1.0 1.0 0.5 0 0.33 VB (kips) (Right of B) 0 -0.33 θ 1 0 MC (kip·ft) 0 -6 -0.33 VC (kips) 0.5 0 -0.5 -0.33 12-13 Copyright © 2018 McGraw-Hill Education. The modulus of elasticity of the concrete is 3000 kips/in. and E = 26,000 ksi. 60' 5' FUNDAMENTALS OF STRUCTURAL ANALYSIS 5th Edition Kenneth M. ... P B 30' rod A C turnbuckle 80' 80' P6.20 Equilibrium Joint C: ΣFx = 0 = -T + 750 cos(20.56) 750 kips T = 702.25 kips 20.56 α ΣFx = 0 = Cy - 750 sin(20.56) T C y = 263.39 kips Cy = 0.5P By symmetry: Ay = Cy = 263.39 kips The rod size is found as: Areq = rod = dreq = T = 21.95 in 2 32 ksi 4 Areq = 5.29 in. 18 kN w = 12 kN/m 3m A C B 8m P14.4 Equilibrium at Joint 1 ΣM 2 = 0 ... Continued ... Copyright © 2018 McGraw-Hill Education. Draw the influence lines for the bar forces in members AB, BC, BC, and LK of the live load is applied to the truss in Figure P12.30 through the lower chord. Calculate ... 9-51 Copyright © 2018 McGraw-Hill Education. The tail building in Figure P9.49 is constructed of structural steel. Continued y P@ ... Coupled ... Solving ... Copyright © 2018 McGraw-Hill Education. Fixed End Moments FEM AB = 0 ... Continued. Draw the influence lines for the bar forces in members AB, BC, and LK if the live load is applied to the truss in Figure P12.30 through the upper chord. (b) The truss in Figure P12.17. Draw influence lines for the bar forces in members AB, BC, DE. ... Copyright © 2018 McGraw-Hill Education. Determine all bar forces. Reactions –3 Equations of Equil. A 12 kips 195 kips • ft C B D E hinge 25' 5' 5' 5' 5' P5.18 FBD of segment CDE 3 kips VC ΣMC = 0 = 3(5) -195 + 12(15) - Dy (10) Dy = 0 kips 12 kips C D ΣFy = 0 = -VC - 3 -12 E 195 kips VC = -15 kips 5 ft FBD of segment ABC 5 ft VC = 15 kips ΣM A = 0 = -By (25) + 15(30) By = 18 kips C ΣFy = 0 = -15 + 18 + Ay 25 ft VC = -3 kips Ay = -3 kips 5 ft By = 18 kips 15 ft Shear (kips) -3 75 Moment (kip-ft) -75 Deflected Shape POI POI 5-20 Copyright © 2018 McGraw-Hill Education. at Joint 1) 6-18 Copyright © 2018 McGraw-Hill Education. , 2 AC = 4 in. If windows are insulated, calculate the expected hydrostatic loading on the adjacent outside walls due to water retained by the floor, or floors. ... Copyright © 2018 McGraw-Hill Education. Continued (a) 6 Reactions – 3 Equations of Equilibrium – 1 Equation of Condition (MHINGE = 0) \ Indeterminate 2° (b) Reactions Form A Parallel Force System : Unstable Remove Restraints to Establish Degree of Indeterminacy ... Determine all bar forces. ... Computer study ... 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 inflection in each span in Figure P13.2. Compute the values of moment at supports B and C, and draw the shear and moment curves.
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.
Rsh P5.54. (2) Forces must both be tension or compression. C P8.1. For the truss in Figure P8.1, compute the horizontal and the vertical components of displacement at joint B produced by the 100-kip 2 load. PCG = Same area as Max. Compute the vertical deflection of a point C in Figure P8.23. ... Copyright © 2018 McGraw-Hill Education. P12.32, P10.34. The girders are 62-ft long and weigh 400 lb/ft. ... Copyright © 2018 McGraw-Hill Education.
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 B, C, D and E, and moments at C and D. What value of jacking force T must be applied at supports B and C to yield a ... Copyright © 2018 McGraw-Hill Education. ... P9.34 ... Copyright © 2018 McGraw-Hill Education.
... (a) In Figure P6.15 compute the vertical and horizontal components of displacement of joint E produced by the loads. Computer analysis of beam of varying depth. ... Copyright © 2018 McGraw-Hill Education. ... Since the truss is ... we can determine the reactions of the frame ... Determine the vertical deflection of joint D. Computer study ... the loads on the rigid frame in Figure P9.43. ... Copyright © 2018 McGraw-Hill Education. ... FUNDAMENTALS OF STRUCTURAL ANALYSIS 5th Edition Kenneth M. Leet, Chia-Ming Uang, Joel T. Lanning, and Anne M. Gilbert SOLUTIONS MANUAL ... Copyright © 2018 McGraw-Hill Education.
... Practical Application Arched Frame Truss Member Type Bar Force, (kips) Tension or Compression Length, L (ft) M1 M2 M3 M7 M8 M9 M13 M15 M17 M18 M20 Bot-chord Bot-chord Bot-chord Top-chord Top-chord Top-chord Top-chord Vertical Vertical Mid-Vertical Diagonal ... Copyright © 2018 McGraw-Hill Education. ...
... 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. ... 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. ... Copyright © 2018 McGraw-Hill Education. Draw the shear and moment curves produced by the total dead load for beams B1 and B2. ... Copyright © 2018 McGraw-Hill Education. FUNDAMENTALS OF STRUCTURAL ANALYSIS 5th Edition Kenneth M. Leet ... Figure P5.52, produce a trapezoidal distribution of soil pressure. ... settles vertically downward 3/4 in. Uniform Load = 0.9375 k ft \ Analyze by Moment Distribution ...
Computer investigation of wind load on a building frame. ... Copyright © 2018 McGraw-Hill Education. ... The resultant reaction at A is equal to the cable tension ... Copyright © 2018 McGraw-Hill Education. ... 6-8 Copyright © 2018 McGraw-Hill Education. ... P6.32). w = 2 kips/ft P5.27. Determine the member end moments and 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 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 moment reaction at B 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 = 45 kN/m P10.15.
Determine the location of the 40-kN load such that sag 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 indeterminate beam in Figure P5.33. w = 2 kips/ft P10.6. Draw the shear and moment curves for the frame in Figure P10.6. ... Copyright © 2018 McGraw-Hill Education. ... Figure P11.17 ... Copyright © 2018 McGraw-Hill Education. Determine the reactions at A and C. ... 464.6 kip·ft -464.6 kip·ft 3.87 kips 12 ft A 3.87 kips Shear Moment (kip·ft) Copyright © 2018 McGraw-Hill Education. 2 AC = 4 in. ...
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. 10.5° 0.8 roofs with obstructions or non-slippery surfaces 0.6 unobstructed slippery surfaces with thermal resistance, R ≥ 30°F·h·ft²/Btu (5.3°C·m²/W) for unventilated roofs or R ≥ 20°F·h·ft²/Btu (3.5°C·m²/W) for ventilated roofs ... Copyright © 2018 McGraw-Hill Education. ... A Pyramids rotation and horizontal displacement but permits vertical displacement. ... Moment Distribution FE, DE, and CB, and V = 25 kips. ... Copyright © 2018 McGraw-Hill Education. Determine the reactions at A and C. VA = 45 kN ...
Calculate the axial forces produced by the live load in column C1 in the third and first stories. ... Copyright © 2018 McGraw-Hill Education. ... produce a uniform compressive stress in the arch ... Copyright © 2018 McGraw-Hill Education. ... P8.38. 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 JKL 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 deflection at joint B and (b) the change in slope of member BC. Compute the deflection at B and the slope at C in Figure P9.22. 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 the fixed base and continuous beam models. Gilbert SOLUTIONS MANUAL CHAPTER 2: DESIGN LOADS AND STRUCTURAL FRAMING 2-1 Copyright © 2018 McGraw-Hill Education. ... Vierendeel truss in Figure P11.31 by moment distribution. ... Floor beams are connected to columns by rigid joints. ... 2) Mmax = -1.5(12.672) + 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 = P1 = 1.714 k·el. Moments @ ends of beam ... @ top of column.
Neglect all the applied loads. 5' 10' 10' P5.22 Left Side: Displacement ... Copyright © 2018 McGraw-Hill Education. ... P12.30 ... Copyright © 2018 McGraw-Hill Education. ... 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 RA and the bar forces in members AD, EF, EM, and NM. ... 18 in. 8-9 Copyright © 2018 McGraw-Hill Education. For both the girder and the arch, determine the axial and bending forces acting on the arch joints as well as the joint displacements. E P4.44. For all members, E = 4000 ksi 4 and I = 100 in. Express answer in terms of EI.
... 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 ... 13-53 Copyright © 2018 McGraw-Hill Education. 9 kips P=?
To construct this sag, a cable and post are added beneath the beam. (kN) 9-29 Copyright © 2018 McGraw-Hill Education. (c) Using 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' 3k A 8' 16' P16.6 Force Vector Joint "B" Joint "C" Freebody "BC" M1 + 20 + M2 - 20 + 10 = 0 F3 = 3 ... Copyright © 2018 McGraw-Hill Education. Global Stiffness Matrix Unit Rotation at Joint "C" Unit Rotation at Joint "B" 16-11 Copyright © 2018 McGraw-Hill Education. 6 Bars are ...
9' P11.10. (f) Evaluate the moment at section 1 , and E = 29,000 kips/in.
The objective is to establish the difference in response of a parabolic arch to (1) a uniformly distributed load and (2) a single concentrated load. ... what is the minimum required value of f? Load is transferred from the roadway to the upper panel points by a system of stringers and floor beams (not shown). P13.18. ... Copyright © 2018 McGraw-Hill Education. , and E = 30,000 kips/in. Evaluate the influence line ordinates at the quarter points of the span. (b) Compute the deflection of point C. Determine all reactions and draw the shear and moment curves, locating points of inflection and labeling values of maximum shear and moment in each span. Compare the magnitude and direction of the forces ... Copyright © 2018 McGraw-Hill Education. Shear Moment (kip·ft) 90 o Deflected Shape 5-46 Copyright © 2018 McGraw-Hill Education.
= 8.1k 2 J + M A max. 3 kips P13.14. ... Copyright © 2018 McGraw-Hill Education. ... 33.554 K 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. C11, P 9a kips 22" CL C D B 9' 9' 26" ... Copyright © 2018 McGraw-Hill Education. ... (a) Roller Support at D B-51 Copyright © 2018 McGraw-Hill Education. (b) Diagonal bars do not buckle and may carry either tension or compression. Expansion, α = 6 ⋅ 10-6 in / in of Determine the Bar Forces and Reactions ... Copyright © 2018 McGraw-Hill Education. ... Computed Previously Computed Compatibility EQ. The moment in the ... Compute the deflection of ... @ 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. EI 5 6000 = 1214 in. P13.20. Computer Study–comparison of cantilever and portal methods with an exact analysis. ... Copyright © 2018 McGraw-Hill Education. ... P12.19. draw the influence lines for the reaction at (a) the lift frame, (b) the moment at B, and (c) the shear between CE. If stable, indicate if determinate or indeterminate. ... the joints, establish the elevation of the lower chord joints at B and D. ... Copyright © 2018 McGraw-Hill Education. ... Compute the vertical displacement of the hinge at C for the funicular loading shown in Figure P16.5. ... Assume a flexible joint at B and D. ... Copyright © 2018 McGraw-Hill Education. ... FEM ... Determine the bar forces in all members of the truss in Figure P7.3 using the method of joints. ... Copyright © 2018 McGraw-Hill Education. ... Determine the reactions and draw the shear and moment curves. ... Copyright © 2018 McGraw-Hill Education. The cable stretching in this truss ... Copyright © 2018 McGraw-Hill Education. ... (c) Determine the maximum ... Determine the forces in all bars. ... Copyright © 2018 McGraw-Hill Education.
M EXT = 15 (6 ...) ... Copyright © 2018 McGraw-Hill Education. ... Use the shear at hinge B as the redundant.
B w = 6 k/ft C 5m A 5m P8.29 Virtual Work Compute δCx: ... Copyright © 2018 McGraw-Hill Education. ... P-System Q1 System for δCx Compute δCy: ... Copyright © 2018 McGraw-Hill Education. ... 16-4 Copyright © 2018 McGraw-Hill Education. Indeterminate 6°: Rotation @ A, B, C, & D (c) Indeterminate 13°: 10 Joint Rotations & 3 Degrees of Sidesway 10-42 Copyright © 2018 McGraw-Hill Education. Using the method of sections, determine the forces in the bars listed below. Their analysis results are different 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 1⁄4 in. Draw the shear and moment curves for the footing in the longitudinal direction. Express answer in terms of E, I, L, and w. Analyze the frame in Figure P10.16. ... Copyright © 2018 McGraw-Hill Education. ... Compute the reactions and draw the shear and moment curves for the beam in Figure P11.27. Assume that the supports at A and D are fixed. ... Copyright © 2018 McGraw-Hill Education. ... Determine the reactions and draw the shear and moment curves for the beam in Figure P3.37. ... Copyright © 2018 McGraw-Hill Education. ... 62 kips ... Copyright © 2018 McGraw-Hill Education. ... Statically determinate, stable ... Copyright © 2018 McGraw-Hill Education.
K J P12.32. P5.51. 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 30 kips 30 kips ... The load on ... Since the Y-components are equal, the only way equilibrium can be satisfied is ... Copyright © 2018 McGraw-Hill Education. ... A tower is ... Copyright © 2018 McGraw-Hill Education. ... P12.59, may be treated as a fix-ended beam of varying depth. P16.4. Using the influence lines ... Draw the influence lines for ... Compute the reaction of ... Copyright © 2018 McGraw-Hill Education.
(see sec 11.25, for evaluation of δA) 4.375 E 4.375(123) = 29,000 in ... Copyright © 2018 McGraw-Hill Education. Compute the Deflected Shape Moment Created by Fabrication Error Units (ft,kips) 9-46 Copyright © 2018 McGraw-Hill Education. Published on Jul 30, 2020 About Download Solution Manual Fundamentals of Structural Analysis 5th edition 0073398004 Fullink download Want more? ... Determine the shear and moment at section 1, and the deflection at C. Compute the slope at B and the shear to the right of support A. Determine the reactions at supports A, B, C, and E. ... Copyright © 2018 McGraw-Hill Education. ... The load on the roof ... Copyright © 2018 McGraw-Hill Education. ... Determine the reactions at supports A and ... Copyright © 2018 McGraw-Hill Education. ... P12.35 ... Copyright © 2018 McGraw-Hill Education. ... Copyright © 2018 McGraw-Hill Education. ... Copyright © 2018 McGraw-Hill Education. ... 0.0145 X B ... 12 kips P13.6 ... Copyright © 2018 McGraw-Hill Education. ... Reactions ... Education. K2y and K3y are stiffness coefficients.
The diagram is close to the exact values. (b) Compute the vertical deflection of joint B. "C": + ΣFy = 0; ... Copyright © 2018 McGraw-Hill Education. ... Construct the influence lines for RC and MC in Figure P12.7, using the Müller-Breslau method. (b) Compute the force in member HC if panel points B, C, and D are each loaded by a concentrated vertical load of 10 kips. The cable is tensioned until the force in the post causes a moment equal in magnitude, but opposite in sense, to the moment in the beam. 9 C A P12.16.
B 12' C 12' D 6' MA , MC , and VC (to left of support C) P12.21 ... Copyright © 2018 McGraw-Hill Education. ... w = 4 kip/ft B 6 Compute δBH, δCV ... Copyright © 2018 McGraw-Hill Education. ... Determine the reactions and draw the shear and moment curves ... Copyright © 2018 McGraw-Hill Education. ... Determine the forces in all bars ... Copyright © 2018 McGraw-Hill Education. ... Draw the shear and moment curves for each member of the frame in Figure P5.43. ... Copyright © 2018 McGraw-Hill Education. ... Copyright © 2018 McGraw-Hill Education. ... Compute the reactions at supports and draw shear and moment ... Copyright © 2018 McGraw-Hill Education. ... Copyright © 2018 McGraw-Hill Education. ... Copyright © 2018 McGraw-Hill Education. ... Copyright © 2018 McGraw-Hill Education. ... Copyright © 2018 McGraw-Hill Education. ... Copyright © 2018 McGraw-Hill Education. ... Apply Loads. Compute the reactions and draw 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 vt. for exterior girders and 0.50 vt. from interior girders. Determine all bar forces.
E 4m P3.16. z V(x) P5.12. P11.13. B = 10 kip/ft. P4.36. Represent the arch by a series of straight segments between joints. ∴ ΔCO 1 3/4 (T) 0 ( ) 5 = ΣFQ ΔLP = · (-0.25) 4 ΔCO = 0.3125 in. (OK) Relative Displacement between 2nd and 3rd floors is 0.24 – 0.136 = 0.104 in. , columns 2 4 A = 13.1 in.

P4.27. The frame is composed of members connected by frictionless pins. Hint: For part (b) use the influence line for moment. 35 kips 30 kips I J 35 kips H G C B 40 kips F D 50' 38.46' 39 kips A E 4@30' = 120' 90 kips 90 kips P8.16 E = 30,000 ksi, A = 20 in2, I = 600 in4 Vertical Deflection of Joint C P = -45in 52 -18cos 52 = -46.55k FAB P = (45cos 52 -18sin 52) x = 13.52 x M AB P = 15sin 21 -16cos 21 = -11.23k FBC M BC = 659.5 - 20.52 x Q FAB = FDE = -0.763 M AB = M DE = -0.165 x Q P = FCD = -0.739k FBC Q Q M BC = -MCD = -8.05 x -0.25k P = -15sin 52 -16cos 52 = -22.9k FED P = -4.95 x M ED P = -15sin 52 -16cos 52 = -22.18k FOC P = -241.46 + 7.51x MOC 8-19 Copyright © 2018 McGraw-Hill Education. 1 ft = 1 ft, 1 ft and 1 ft. Compute Stiffness Matrices (See Equation 15.48) Member 1 x 2 the first four = 250 k/in , 2 and members BD, DE, and CE have A = 1 in. = 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 reactions and draw the shear and moment curves for each member 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. 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. Continued Copyright © 2018 McGraw-Hill Education. 2 3 B A KS = 1. Compute ... P13.1 C = Constant P6.2 C ... 2 2 E I 4 3 D B 1 1 4 ... P = ... (w L)...
Hinges Freebody left of Hinge to "C" ... P P.16 ... Copyright © 2018 McGraw-Hill Education. Compute the fixed-end moments for the members. ... P10.34 Unknowns: θ A, θB, θC, Δ ... ΨAB = ΨBA = 4 10 ΨCD = ΨDC = - Δ 10 (50)(24) FL == -150 k·ft 8 8 = 150 k·ft FEM BC = FEMCB Member end moments 12.5 13-7 Copyright © 2018 McGraw-Hill Education. ... Compute study of a cable-stayed bridge. "B": JT. Determine the reactions at supports A and C of the three-hinged circular arch in Figure P6.6.

Fig P8.30. 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 overturning moment. P5.55. 20 kips A C B D 12' E 9' F 9' 12' 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 ... Copyright © 2018 McGraw-Hill Education. ... SOLUTIONS MANUAL CHAPTER 8: WORK-ENERGY METHODS FOR COMPUTING DEFLECTIONS ... Copyright © 2018 McGraw-Hill Education. Determine the reactions at the supports of the frame in Figure P9.25. Determine the forces in all members of the truss. ... P8.26 Determine the reactions at supports ... determine the forces in all bars of the truss. Determine the reactions at all supports and the moment diagrams for the members. ... Copyright © 2018 McGraw-Hill Education. Compute the vertical deflection ... Copyright © 2018 McGraw-Hill Education. The composite concrete floor slab construction spans over simply supported steel beams ... Copyright © 2018 McGraw-Hill Education. ... Compute the reactions and draw the shear and moment curves ... Copyright © 2018 McGraw-Hill Education. ... Diagonal bars are assumed to carry ... Copyright © 2018 McGraw-Hill Education. ...

... Copyright © 2018 McGraw-Hill Education. ... Construct the influence lines for the reactions ... Copyright © 2018 McGraw-Hill Education. ... Determine the reactions and the forces in all bars of the truss ... Copyright © 2018 McGraw-Hill Education. ...

Consider Joint F Force in Bar FC: FFCy 3m 4m 3m 3m FFC B FFCx B 3m C 4C -12 kN 42 F F 60 kN 42 FFE 60 kN FFCx F ... Copyright © 2018 McGraw-Hill Education. ... Copyright © 2018 McGraw-Hill Education. ...

Continued 6.6 kips ... Copyright © 2018 McGraw-Hill Education. ... Analyze the beam in Figure P14.4. After member end moments are determined, compute all reactions and draw the shear and moment curves. ... Copyright © 2018 McGraw-Hill Education. ... Copyright © 2018 McGraw-Hill Education. ... Draw the influence lines ... Copyright © 2018 McGraw-Hill Education. ...

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' 12' A P3.22 By 10 kips/ft 10 kips/ft B Bx C 6 ft 12 ft Dy 55 kips Dx D Dx E D 12 ft E 12 ft Dy Fx A B Ax Ay ... Copyright © 2018 McGraw-Hill Education. ...

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.

P12.27. A simply supported crane runway girder has to support a moving load shown in Figure P12.49. ... Copyright © 2018 McGraw-Hill Education. ... Copyright © 2018 McGraw-Hill Education. ... Copyright © 2018 McGraw-Hill Education. ...

kN 3m D 6 kN/m 12 kN 3m Ax Z A Ay 4m 4m 4m ΣM z = 4(Ay) + 3(12) + 2(6)(4) + 20(4) = 0 Ay = -41 kips ΣFy = 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 ΣFx = Ax + 12 + Dx = 0 Ax = -125.3 kips ¬ 3-21 Copyright © 2018 McGraw-Hill Education. wearing surface of asphalt. VP. floor beam upper chord truss lower chord P4.53. A B 1 C D 1 16' E 12' 12' 12' 12' P5.50 Entire Structure: Compute Reactions: + ΣFx = 0; 30 k - E x = 0 E x = 30 k ¬ + ΣMe = 0; 30 k (16 ¢) - 40 k (24 ¢) + Dy (24 ¢) = 0 Dy = 20 k + ΣFy = 0; - 40 k + E y - 20 k = 0 E y = 60 k Member "ABCD": + ΣM B = 0; - 40 k (12 ¢) - CE y (24 ¢) + 20 k (36 ¢) = 0 CE y = 10 k CE x CE y = 3 4 + ΣFx = 0; Shear & Moment Curves MBR "ABCD": CE x = 7.5k 30 k - BE x + 7.5k = 0 BE x = 37.5k ¬ ΣFy = 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 αt = 6.6 × 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 ΔB = ΔE = Δ 2. Since all vertical cables carry a portion of the live load applied to the stiff continuous girder, the distribution of forces to the arch is more uniform, which in turn creates a more uniform stress 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 , Ψ EI =K 12 EI KA = = 6K 20 For Columns KC = For Girders PB 2 A 30(16)2 (4) = = -76.8 kip · 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 ΣF x = 0; 8V1 = 100 kN \ V1 = 12.5 kN (b) Shear @ Base of column ΣF x = 0; BV2 = 400 kN \ V2 = 50 kN Portal Method Results (Units in kN, m) 13-34 Copyright © 2018 McGraw-Hill Education.) 10 10 10 11 11 9 9 1 1 1 0.25 0.25 0.25 0.25 δP = ΣFQ FP L AE FP L (in.) AE Cx Cy Dx Dy −0.076 −0.032 0.026 0.056 −0.025 −0.255 0.089 −0.057 −0.024 −0.053 −0.112 0.007 −0.191 0.066 0 0 0 0.206 −0.034 0 0 −0.217 −0.364 0.172 δCx = -0.217 in. wI 2 6(18)2 == -162 kip · ft 12 12 FEM21 = 162 kip · ft PL 30(24) FEM23 = == -90 kip · ft 8 8 PL FEM32 = = 90 kip · ft 8 Clamp Force = FEM21 + FEM23 = 72 kip · ft FEM12 = Clamp Joint 2 Introduce θ = 1 Radian at Joint 2 2 EI EI (1) = 18 9 2 EI 2 EI = (2) = 18 9 2 EI EI = (1) = 24 6 2 EI EI = (1) = 24 12 M12JD = M21JD M23JD M32JD K 2 = M21JD + M23JD = 2 EI 7 EI + = 9 6 18 Equilibrium at Joint (2) ΣM 2 = 0 72 + θ 2 K 2 = 0 7 EI =0 18 -72 ´18 -185.14 θ2 = = EI 7 EI 72 + θ 2 14-7 Copyright © 2018 McGraw-Hill Education. In Figure P10.10, support A rotates 0.002 rad and support C settles 0.6 in. 8 kips 12' D E 24 kips 12' 36 kips B A 16' 16' P4.9 + ΣMA = 0; 8 (24 ¢) + 36 (16 ¢) + 24 (12 ¢) - By (32 ¢) = 0 k k 5.32 ΣFy = 0; 4 12 ft By = 33k + C 8 kips k Ay - 36 k + 33k = 0 2.67 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 ¢ 38.67 29 B 16 ft ΣMA = 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: δCY WQ = ò 6 1k · δCy = ò 0 MQ M P dx FP L EI AE 12 (-1· x)(-16 x) dx (-0.5 x)(-8 x) dx (2.5)40(15) +ò + EI AC EI AE 0 + FQ · é16 x 3 ù é 3ù ú + ê 4 x ú + 2.5´ 40 ´15 δCy = ê ê 3EI ú ê ú 5E ë û 0 ë 3EI û 0 δCy = 0.73 in 6 12 Compute ΔLBD to Eliminate Defl at C WQ = UQ 1k · 0.73 in = FQ · ΔL 0.292 ¢¢ = ΔL 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 wL) = 4(0.45(3.6 ´ 20)) = 129.6 k Moment: Moment to top of column created primarily by load on grider 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 kN 2. Continued Axial Force Diagram (kips) Relative Displacement between 1st and 2nd floors = 0.136 – 0 = 0.136 in. + + D E F G L M pinned connections typical N O P J 15' pinned base typical A K 6 @ 5' = 30' P13.21 ΣFy = 0; \ Ax = K x = 5k ¬ Ay = 6.67k K y = 6.67k + I 5' 10 k (20 ¢) - Ay (30 ¢) = 0 FBD Column "AB" ΣM B = 0; H B Assume the lateral load is shared equally to each column Entire Structure ΣM 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 the fixedend beams. A preliminary design of the truss in Figure P4.53 produces the following bar areas: 2 members 1 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 is used for both (a) Portal Method and (b) Cantilever Method. Construct the influence lines for the reactions at C, and shear, axial load and moment at point D. Given: I = 150 in. Use E = 29,000 ksi and I = 100 in . 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 ΔCO : 1kN · ΔCO = ΣFQ FP δCC : é (4 2) ùú ΔCO = êê-0.707(141.42) 2 AE úûú êê 1131.82 ΔCO = AE L 1kN · δCC = ΣFQ 2 AE 12 (4) (-0.707)2 (5.66)[2] δCC = + AE AE 9.66 δCC = AE Δ A = ΔL of Bar AC ΔA = Use RC as the Redundant FAC L P-System AE 117.17(4m) m 2 200kN ´106 500mm 2 ´ 6 10 mm 2 m2 Δ A = 0.00469 m = 4.69 mm = 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. 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 VBA = 15 kips 16 ft VAB = 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 90 kip-ft 30 kips -15 kips (C) 15 kips 15 kips 26.4 kips 26.4 kips -7.2 kips (C) 7.2 kips (T) 90 kip-ft 90 kip-ft 12 ft 90 kip-ft 90 kip-ft 15 kips 15 kips 15 kip 330 kip-ft -33.6 kips (C) 240 kip-ft 16 ft 15 kips 15 kips 15 kips 15 kips 15 kips 33.6 kips 15 kips 330 kip-ft 33.6 kips (T) 240 kip-ft 25 ft 33.6 kips (c) Results from the Cantilever and Portal Methods are the 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 ΔCO (Use Moment-Area) 1 324 17, 496 ΔCO = (9)´12 = EI 2 EI PL3 1(15)3 1125 = = δCC = EI 3E 3EI See Table A.3 Compatibility Equation 17496 1125 + RC =0 EI EI RC = 15,552 kips ΔC = 0; - RA = 36 - RC = 20.448k 1 90.72 93,312 (10.563) (4.437) + EI 2 EI 2 291.57 θC = EI θC = - 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 3m P4.40 ΣFx = 0 \ Ax = 0 Cut Section (2)–(5) ΣFy = 0; ΣM f = 0; Ay - 30 = 0 Ay = 30 kN Ay 3 - FBC 2 = 0 FBC = + ΣFx = 0; 3 A = 45 kN 2 Y - FBC + FFE = 0 FFE = FBC = 45 kN Entire Structure ΣM D = 0; 30 ´ 9 - 30 ´6 - E y 3 = 0 E y = 30 kN + ΣFy = 0; 30 - 30 - 30 + RD = 0 RD = 30 kN Final Results Shown Above 4-41 Copyright © 2018 McGraw-Hill Education. 15 Therefore, w 15.71(0.025) 0.393 kips/ft arc π (120) 15.71 ft 360 Using the General Cable Theorem: 1 60 ΣM B ¢ = -Ay ¢ (60) + 0.393(60) = 0 2 3 15.71 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 Stiffness. 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 = 73.3 - 46.4(5.30) = -172.6 » M BA = -173 kip-ft M BC = -73.3 + 46.4(5.30) = 172.6 » M BC = 173 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 = 173 kip-ft Ax = -8.3 - 3.87(5.3) » Ax = -28.8 kips ¬ Dx = -8.3 - 3.87(5.3) = Dx = -28.8 kips ¬ 11-48 Copyright © 2018 McGraw-Hill Education. Deflections computed by moment-area or by in Table A.3. Compatibility Equations Δ B = 0 = Δ BO + δBB RB + δBM M B θB = 0 = θBO + αBY RB + αBM 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 in2, E = 24,000 ksi BAR Fp FQ1 (δBy) FQ2 (δBx) L FpFQ1L (δBy) FpQ2L (δBx) AB BC CD DE AE BE BD 0 +125 –75 0 0 +150 –125 0 0 0 0 0 –1 0 0 0 0 0 0 +0.75 –1.25 25' 25' 30' 20" 20" 15" 25" – – – – – –2250 – – – – – – 1687.5 3906.25 Σ = –2250 Σ = 5593.75 δBy : ΣQ1 · δBy = ΣFQ1 FP L AE -2250 -2250(12%) 1 · δBy = = AE 4(24000) δBy = 0.281¢¢ k δBx : ΣQ2 · δBx = ΣFQ2 Fp L P-System AE 5593.75 k 1 · δBx = AE δBx = 0.70 ¢¢ Q1-System for δBy Q2-System for δBx 8-2 Copyright © 2018 McGraw-Hill Education. Continued Member Section Forces 2 –.089 6.713 68.482 Member Label Section Axial (kN) Shear (kN) Moment (kN-m) 3 –.089 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 52.056 234.251 3 –89.032 I00.047 300.141 5 –161.854 52.056 312.335 4 –89.032 100.047 450.212 1 –26.846 20.756 –16.136 5 –89.032 100.047 600.283 2 –26.846 20.756 4.62 1 –16.24 25.965 9.72 3 –26.846 20.756 25.376 2 –16.24 25.965 35684 4 –26.846 20.756 46.132 3 –16.24 25.965 61.649 5 –26.846 20.756 66.883 4 –16.24 25.965 87.614 1 162.68 51.892 0 5 –16.24 25.965 113.578 2 162.68 51.892 77.838 1 268.7 –135.007 328.471 3 162.68 51.892 155.677 2 268.7 –135.007 159.712 4 162.68 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 71.556 –17.829 5 269.7 –135.007 –345.565 2 27.409 21.556 3.727 1 193.462 –62.437 243.172 3 27.409 21.556 25.283 2 193.462 –62.437 118.297 4 27.409 21.556 46.839 3 193.462 –62.437 –6.578 5 27.409 21.556 68.395 4 193.462 –62.437 –31.453 1 88.336 100.249 0 5 193.462 –67.437 –256.328 2 88.336 100.249 150.373 1 104.419 –62.478 256.438 3 88.336 100.249 300.746 2 104.419 –62.478 131.481 4 88.336 100.249 451.12 3 104.419 –62.478 6.524 5 88.336 100.249 601.493 4 104.419 –62.478 –118.433 1 15.766 25.01 11.756 5 104.419 –62.478 2 15.766 25.01 36.766 1 30.336 –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 5 15.766 25.01 111.797 4 30.336 –135.271 –160.094 1 –.13 95.756 0 5 30.336 –135.271 –329.183 2 –.13 95.66 143.634 1 79.244 –25.846 66.888 3 –.13 95.756 287.268 2 79.244 26.846 33.33 4 –13 95.756 430.901 3 79.244 –26.845 –.228 5 –.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 Cy 30 kN-m 2m B A Ay C Cx D C E Dy 6m 4m 4m Figure (a) 8m Figure (b) Figure (a): ΣM A = -15 + 2(40) + 10C y = 0 C y = -6.5 kips ΣFy = Ay - C y = 0 Ay = -6.5 kips ΣFx = 40 - C x = 0 C x = 40 kips → Figure (b): ΣMe = -30 - 6(2)(12) + 8Dy + 12C y = 0 Dy = 31.5 kips ΣFy = Dy + C y -12(2) + E y = 0 E y = -1 kips ΣFx = C x + E x = 0 E x = -40 kips ¬ 3-15 Copyright © 2018 McGraw-Hill Education. Draw free-body diagrams of column AB and beam BC and joint B by passing cutting planes through the rigid frame an infinitesimal 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) FQ1 (k) FQ2 (k) L (ft) A(in 2) AB +150 –5/8 0 25 5 –5625 0 BC +60 – – – – – – CD –100 – – – – – – DE –90 +3/8 –1 15 3 –2025 +5400 EA –90 +3/8 0 15 3 –2025 0 BE 0 – – – – – – BD –150 –5/8 0 25 5 +5625 0 Σ = -4050 +5400 FQ1 FP (a) δEH L 5400 = AE 30,000 = 0.18¢¢ 1k · δEH = ΣFQ2 FP δEH δEV L -4050 = AE 30,000 = -0.138¢¢ Due to Loads 1k · δEV = ΣFQ FP δEV (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 çè 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 x + c1 24 L 120 30 dx w 5 9wL 3 wL2 2 EIy = x + x x + c1 x + c2 120 L 360 60 dy at x = 0, = 0 \ c1 = 0; and at x = 0, y = 0 \ c2 = 0 dx ΣM A = 0 = M (x) + dy w 3wL 2 wL2 =x4 + x x dx 24 EIL 40 EI 30 EI w wL 3 wL2 2 x5 + x x Δ= y =120 EIL 40 EI 60 EI θ= 5 3 2 5 3 2 Compute ΔB : Set x = L : 4 ΔB = − w L wL L wL2 L wL4 + − = −0.000659 EI 120 EIL 4 40 EI 4 60 EI 4 Compute ΔC : Set x = L : 2 ΔB = − w L wL L wL2 L wL4 + − = −0.001302 EI 120 EIL 2 40 EI 2 60 EI 2 ΔC = 1.98 ΔB 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 BCD Reactions + ΣM B = 0: 25´ 20 + 40 (15) -10 RC = 0 3 RC = 76.67 kN + ΣFy = 0: -By - 25 - 40 + 76.67 = 0 By = 11.67 kN ΣFx = 0: E D Entire Structure Bx = 0 Member AB 5-33 Copyright © 2018 McGraw-Hill Education. 13-55 Copyright © 2018 McGraw-Hill Education. Analyze the frame in Figure P11.16 by moment distribution. Continued (b) Uniformly distributed dead load + 48-kip live load Reactions (kips), Deflections (in), and 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 Compatibility Equation Δ B. Continued Axial Force Diagram (kips) Moment Diagram (kip-ft) 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 ΣM A = 0; By = 75.58k ΣFy = 0; - 27 -13.5 - 2(36) + 75.58 + Ay = 0 + Ay = 36.92 k FBD: Sim-Triangles 3k / ft y = 9¢ x x Y= 3 1 æxö ΣFy = 0; 36.92 k - 3 x - x çç ÷÷÷ = 0 2 èç 3 ø V = 0@ x = 8.394 @ x = 9¢ V = -36.92 + 3 x + @ x = 8.4 ¢ DM = ò Vdx x = -3.58k 6 Δ M = ò 36.92 dx + ò -3 xdx + ò - x2 dx 6 3 x 2 8.4 ¢ x 3 8.4 ¢ - ò = 171.5 0 2 ò0 15 0 M @8.4 ¢ = 171.4 ft · k - 30 ft · k = 141.5 ft · k Max + M = 36.92 x ò 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 funicular loading produces direct stress on all sections of the arch. , and α = 6.5 × 10 (in./in.)/°F. Moment Case 3 Mmax = 24.84 ´ 9.31 = 231.26 kN · m Case 5 32 kN load at £ Resultant = 8 + 32 + 24 = 64 kN R · x = 32 ´8 + 24 ´11 x = 5.625 m Moment of midspan M £ = 39´12 - 24 ´8 = 276 kN · m 12-46 Copyright © 2018 McGraw-Hill Education. . 30 kips 15 kips/ft 150 kip • ft A hinge D C B spring, k = 30 kips/in. FBD "AB" IV. at Joint B M BA + M BC = 0 162 θB = EI \ M AB =-58.8 k · ft, M BA = 107.4 k · ft M BC =-107.4 k · ft, MCB = 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 = 0.16 ΔLTNT = αT (L)ΔT = 6.6(10-6)(15¢ + 20 ¢)12%(80 ¢) = 0.2218¢¢ Δ 6(29,000)(200)(ΔL AB) = 8.5ft·k FEM DB = 6 EI 2 = (15 12%)(12) ´ L Where ΔL AB = 6.6(10-6)(15´12%)(80) = 0.09504 ¢¢ 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 = 25¢; Y= @ x = 25¢; 4(20)(25)2 = 5¢ 1000 dy 8(20)(25) = = 0.4 dx (100)2 Y= 4 = 0.371 10.77 10 cos θ = = 0.928 10.77 sin θ = + ΣM D = 0; 60(50 ¢) - H (20) - 60(25) = 0 H = 75k (a) dx before the1/4 point (left of "D") + ΣFy = 0; 55.68k - 27.83k - VDL = 0 VDL = 27.85k + ΣM D = 0; 60(25¢) - 75(15) - M = 0 M = 375k + ΣFĬ = 0; 69.6 k + 22.86 k - T = 0 T = 91.86 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 = 55.60 - 27.85 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 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 x1 δC ·1k = Σ ò MQ M P 8 δC = ò 0 8¢ dx EI Q-System 12 ¢ dx dx x1 (0.6 x + 6 x1) + ò (x2 + 8¢) éé 0.6(x2 + 8)2 + 6(x2 + 8)ùù ë û 2 EI EI 0 2 1 12 ¢ dx dx + ò 0.6 x23 + 20.4 x22 + 211.2 x2 + 691.2 2 EI EI 0 0 0.15 x14 + 2 x13 8 0.15 x24 + 6.8 x23 + 105.6 x22 + 691.2 x2 12 ¢ δC = + 0 0 EI 2 EI 1638.4 19180.8 20,819.2 = + = EI EI EI 20,819.2(1728) = 29000(1200) δC = ò (0.6 x13 + 6 x12) P-System δC = 1.034 in.

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 9' A B 8' 1.5I 5 kips 10' 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 P (b)a 2 5(20 2)5 5(20)52 = + = + = 20 kip-ft FEMCD = -FEM DC = L2 L2 252 252 FEM BC = K BC = 1.5I 1 = , 18 12 KCD = 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 12.35 FEM 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 δBB = Δ BO AM/EI = x= Δ BO 10(20) 2(8) = 661.33 3 3 ö 2(8) æ 2 ö÷ 10(20) æç 3 çç ÷ ç 10 - 2÷÷ø÷ 3 èç 4 3 èç 4 ø÷ 661.33 661.33 2446.05 = 5.548 = EI 1.5EI δBB C B 4 kips/ft = 5.548 ft tBA = Δ BO 1.5EI EI x 8 kip-ft Therefore: (Δ BS + Δ BO) + δBB X B = Δ B 200 kip-ft æ ö çç-0.288 - 2446.05 (1728)÷÷ + 113.78 (1728) X B = -0.3 çè ø÷ EI EI X B = By = 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 =- K AB = K BC = I 10 2 10(10) = -25 k · ft, FEMCB = 25 k · ft 2 12 = -16.67 k · ft, FEM DC = 16.67 k · ft, DFBA = DFBC = , K BC = KCD = KCE = I 10 , 1 2 DFCB = DFCD = DFCE = 1 3 11-31 Copyright © 2018 McGraw-Hill Education. 6 kN/m A B C 3m 12 m P13.6 wL2 6(12)2 = = 72 kN · m 12 12 In Cantilever = 3´ 24 = 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 flange 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. = Q – Sys.) δBV δCV dx dx = ΣM P 2 EI EI 15 10 dx 2 dx 12 · δCV = ò (84 -12 x) + ò (9.6 x)2 EI EI 0 0 é 15 ê 16 é 126 2 9 3 ù 92.16 ê 441x δCV = ê x + x ú + ê12 EI êê ú 2 3 û0 3EI êê δCV = 0.96 ¢¢ Q · δP = ΣMQ M P é x3 ù êê úû 0 12 10 ù ú (1728) ú úû δBV 26 1k · δBV = ò (15 - x)(84 -12 x) 0 δBV = P & Q-System for δCV dx EI 2 3ù é ê315 x - 66 x + 3 x ú (1728) ê 2 3 úû 0 ë δBV = 0.432 ¢¢ 4 BI 15 Q-System for δBV 8-36 Copyright © 2018 McGraw-Hill Education. Compute the vertical and horizontal displacements at joint B. The truss frame in problem P13.22 is 5 ft 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 · m 2 B Shear in Columns MA = ΣM A = 0; 612.8 + 306.4 - VBA (5) = 0 VBA = VAB = 183.84 kN Reactions 13-13 Copyright © 2018 McGraw-Hill Education. 1 15 m B C D A hinge hinge F 20 m 30 m E 20 m 40 m 20 m 20 m P12.25 Reaction at support A: DL = 10 k / ft é1´ 20 1´60 ù ú RA = 10 kips/ft ê ê 2 2 úû RA = -200 kN Reaction at support F: DL = 10 k / ft é 2 ´80 ù ú RF = 10 kips/ft ê ê 2 úû RF = 800 kN 12-28 Copyright © 2018 McGraw-Hill Education. 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 + ΣM B = 0; M BA + M BC + M BD = 0 (3.133θB + 0.03556Δ x)EI = 0 θB = -0.011349Δ x + ¬ (5¢) = 0 ΣM A = 0; M AB + M BA - VBA Segment AB M AB + M BA (1.2θB - 0.04Δ x)EI = 5¢ 5¢ VBA = (0.24θB - 0.008Δ x) EI Where θB = -0.011349Δ 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 P5.48. P2.19. Compute the reactions at supports A and E of the three-hinged parabolic arch 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 ΣK 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 ΣK 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" 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 (2θ A + 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 , M AB = -16 k·ft θA = - \ M AD (b) Axial Forces Axial force in AD = 0 k Axial force in AB = 12 k 10-13 Copyright © 2018 McGraw-Hill Education. 10' A MA B C L = 40' RA P12.51 -Pab 2 -9 L = 64 L2 2 Pba 3L = 2 = 64 L æ 3L ÷ö 9 L 3L RA L -1çç ÷ + =0 çè 4 ÷ø 64 64 54 RA = 64 M A = FEM AB = M B = FEM BA ΣM B = 0; æ 27 ö Max RA = 60 (1) + 30 çç ÷÷÷ = 85.31k èç 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. ΣFy = 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: + ΣM A = 0; 25k (25¢) + 15k (60 ¢) - C y (120 ¢) = 0 C y = 12.71k + ΣFy = 0; -25k -15k + cy + Ay = 0 Ay = 27.29k FBD "AB": + ΣM B = 0; Ay (60 ¢) - Ax (45¢) - 25k (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 + ΣM B = 0 = -RDY B -103.03(4) + 200 ´ 4 RDY = 48.485 kN RBY = 48.485 kN (Symmetry) RDX = RDY = 48.485 Compute Δ‡AX Consider Bars AC and AD in the Q and P-Systems. 1 1' B C 18' D A 48' P12.28 + -M + Ay (1¢) = 0 M = Ay (1¢) 47 Load @ x = 1 : M = (1¢) = 0.979 48 24 Load @ x = 24 : M = (1¢) = 0.5 48 ΣM = 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 9m RD = 9 kN 6m P5.38 Since Structure Symmetrical Analyze Left Half. P11.28. and A = 17.9 in. M = 30 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 σ= 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 400 kip • ft B C 12' D A 16' P11.12 3æ I ö K AB = çç ÷÷ , 4 çè12 ÷ø 1æ I ö K BC = çç ÷÷ 2 çè16 ÷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. θ A = θF = θ and θC = θD = -θ. A P12.52.

Use BCD Compute ΔCO : 1kN · ΔCO = Σ ò M E M P + dx EI RD = 5.463kN æ 2 ö 2 æç x ö dx dx x ç-6 x - 4 x · ÷÷÷ + ò çç- x ÷÷÷(6 x) ç ç 3 ø 3 2 2 EI EI è ø è 0 0 3 3 æ 1 ö÷ x dx dx +ò - (12 x) + ò çç- ÷ (3 + x)[12(x + 3) -18 x] ç ÷ 3 EI EI è 3ø 6 3 1· ΔCO = ò 0 ΣM B = 0 = -19.61´3 + 18´6 - 9 RD Shear & Moment Diagrams 0 3 3 æ 4 x 2 ö÷ Dx 4 x 2 dx dx ÷÷ = ò çç-4 x 2 +ò + ò (-4 x 2) çè 3 ø÷ 2 EI EI EI 6 ΔCO 0 0 0 3 dx +ò 2 x 2 - 6 x - 36 EI 0 9-14 Copyright © 2018 McGraw-Hill Education. S = 0.97k 6 EIΔ L2 Assume Δ 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 = 3 I = 0.0625 4 12 DFCB = 0.6 KCD = 1 I = 0.0417 2 12 DFCD = 0.4 ΣK = 0.10417 1.0 (6)2 = 361 ft · k 2 = FEMCB = FEMCD FEM BA = 2 k /l FEM BC = 2 k /l (12)2 = 24 ft · k 12 + Mmax Member "CD" + 2 k /l (12 ¢)2 - VDC (12 ¢) = 0 2 = 12 k ΣMC = 0; - 21.6 + 21.6 + VDC = VCD P.I. 14.4 ft · k = 2 (L ¢)2 /8 L ¢ = 7.6 ft ¢ (12 – L) = 2.21 ft from C & D 2 k / ft P.I. @ = 0.184 L from C & D Accuracy of approximate method for locating the point of inflection from supports "C" & "D" is 91.5%; yielding midspan moment 90% accurate. Indeterminate, stable Determinate, stable b=7 b = 10 (g) r =3 n=5 r =4 n=7 (b + r = 10) > (2n = 10) (b + r = 14) > (2n = 14) Geometrically unstable: concurrent reactions Determinate, stable b = 11 b=9 (h) r =4 n =6 r =3 n =6 (b + r = 15) > (2n = 12) (b + r = 12) > (2n = 12) 3 deg. If your solution is statically inconsistent, what 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 kips stringer 94 kips D E 51 kips F bracing 12' A J I H G A 26' 4 @ 16' = 64' Section A–

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 K5 = 5EI L3 L C D spring L L EI = constant P16.2 Stiffness Matrix K Joint "B": + ΣFy = 0; + ΣM B = 0; 2(12 EI) 5EI + 3 - K11 = 0 L3 L 29 EI K11 = 3 L -K 21 K 21 = 0 Unit Vertical Displacement Δ1 = 1 6 EI 6 EI + 2 =0 L2 L Joint "B": Joint "C": Joint "C": + ΣMC = 0; 6 EI =0 L2 6 EI K31 = 2 L K31 - Unit Rotation @ Joint "B" θ2 = 1 Joint "B": + ΣFy = 0; -K12 K12 = 0 + ΣM B = 0; 6 EI 6 EI + 2 =0 L2 L 2(4 EI) =0 L 8EI K22 = L K22 - Joint "B": Joint "C": + ΣMC = 0; 2 EI =0 L 2 EI K32 = L K32 - Joint "C": 16-4 Copyright © 2018 McGraw-Hill Education.
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. What conclusions do you reach from the above study? Given: E = 200 GPa 2 and A = 500 mm for all members. 12-60 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 hanger Mechanical Floor Plan (beams not shown) (a) floor framing above supports vertical lateral bracing beyond hanger hanger mechanical unit floor grating mechanical support framing Section (b) P2.11 a) Live Load Impact Factor = 20% b) Total LL Machinery = 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 Acting on one Hanger = 4 /4 Hangers = 1 kip k k k ∴ Total DL + LL on One Hanger = 2.3 + l = 3.3 kips 2-12 Copyright © 2018 McGraw-Hill Education. D 4′ w = 3 kips/ft P10.9. (a) Under the applied loads support B in Figure P10.9 settles 0.5 in. F . Determine the degree of kinematic indeterminacy for each structure in Figure P10.35. Entire Structure 5.71 - 20 + 28.62 - 30 + RC = 0 RC = 15.67 kN Colclusion: Exact and Approx. Carry out an approximate analysis of the truss in Figure P13.10 by treating it as a continuous beam of constant cross section. P12.29. The area of 2 both the top and bottom chords is 10 in. Determine the 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 8 4 Moment (kN·m) Shear (kN) 11-28 Copyright © 2018 McGraw-Hill Education. As part of the analysis, evaluate the forces in members DE and EF and compute the reactions at A and K.
w = 2 kips/ft B C 15′ P = 30 kips P = 30 kips 10′ 10′ A D 30′ P11.11 Consider Symmetry, No Joint Translation Joint B Distribution Factors I 25 1æ I ö = çç ÷÷÷ 2 èç 30 ø K BA 12 = ΣK S¢ 17 K 5 = BC = ΣK S¢ 17 K BA = D · FBA = K DC D · FBC ΣK S¢ = 17 I 300 Moment Distribution Fixed end Moments wL2 2 (30) = = = 150 k 12 12 2 Pab 2 30 (10) (15) = 2 = = -108k 2 L 25 ( ) 2 FEM BC FEM AB Pab 2 30 (15)(10) = 2 = = +72 k 2 L (25) 2 FEM BA Girder Leg AB (Leg CD Similar) 11-13 Copyright © 2018 McGraw-Hill Education. (in.3) 3930 Vol. 2 P13.17. A P12.3. Draw the influence lines for the reactions at supports A and C, the shear and moment at section B, and the shear just to the left of support C. P10.35. Compare the results of this analysis with part b of P6.32 by plotting to scale the vertical deflections of all joints along the roadway from joints 1 to 11.
Continued Express equilibrium EQ's in forms of displacements 6.4 KθB + 1.2 KθC - 6 KΨ = 76.8 1.7 KθB + 6.4 KθC - 6 KΨ = -19.2 0.5 KθB + .5 KθC - 2 KΨ = 0 (a ¢) (b ¢) (c ¢) Solve EQ's for θB , θC & Ψ and substitue into EQ in (1) θB = 15.49 K 2.97 K 3.13 æ 12 ö Ψ= = 3.13çç ÷÷÷ è EI ø K 3.13´12 = (12)2 29,000 ´ 240 θC = - Ψ = 7.771´10-4 rad Δ B = ΨL = Ψ(12 ´12) = 0.112 ¢¢ 10-33 Copyright © 2018 McGraw-Hill Education. A mechanical support framing system is shown in Figure P2.11. End Moment 456 ftk Max.
B 4 kips C 12′ A 18′ P9.43 Selecting Cy as the redundant, Δ2 the compatibility equation is: (ΔCS + ΔCO) + δCC XC = ΔC Δ1 = θB LBC = 12 1944 EI Δ2 = tCB = = 4 kips EI B θB = ΔCO 2EI 2EI A A ( ) ( ) 1 18 2 (18) 18 2 EI 3 C B 1 kip 3888 EI 9/EI 1 æ 24 ö = θB LBC = 12 çç ÷÷÷ (18) 2 è EI ø 2592 ΔCO = EI 24/EI A 0.667 12 C B C B Therefore: (ΔCS + ΔCO) + δCC XC = ΔC 2592 + X (0 - 3888) EI EI C 1 kip 1944 EI δCC = ΔCO 9 (18) EI Δ1 C EI B δCC = Δ1 + Δ2 = θB C =0 XC = C y = 0.667 kips A A Final Shear (kips) 36 Final Moment (kip-ft) 9-48 Copyright © 2018 McGraw-Hill Education. The equation for 2 2 the arch axis is y = 4hx /L . E = 200 GPa, I = 60 × 10 mm . 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: 8-52 Copyright © 2018 McGraw-Hill Education. The deflected shape of AB, CB, FE and DE are identical. 6 kips P5.10.
, and A = 3 in. EI ΔCO = tCA = (ΔCS + ΔCO) + δCC XC = - θB tCA = ΔCO xC 240/EI XC K X (0 - 7.188) + 1.214 XC = - C 5 XC , beem = C y = 5.08 kips ΔC = -5.08 = -1.02 in. Determine all reactions at points A and D in Figure P10.30. (b) Determine truss 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. EFx - CBx = 0 CBx = 40 k ¬ æ 3ö CBy = 40 k çç ÷÷ = 30 k çè 4 ÷ø ΣFy = 0; + - 30 k + 30 k - 30 k + Dy = 0 Dy = 30 k 4-49 Copyright © 2018 McGraw-Hill Education. Indicate whether stable or unstable. 1 B C 2 10′ A 10′ P16.7 For Members 1 and 2: A EA 0.45´106 = = = 0.03 in-2 I EI 15´106 12 12 = = 8.33´10-4 in-2 P= 2 2 (10 12) ´ L 6 6 Q= = = 0.05 in-1 L 10 ´12 EI 15´106 = = 125,000 kip-in L 10 ´12 N= f = 90, For Member 1 é NC + PS ê EI ê [k1F] = ê L ê ê SYM . Computer Study—comparison of approximate analysis with exact analysis. Also determine the lateral deflection of the cable tower. = (1.5(9))1.2 k / ft. For the trusses in Figure P9.28, 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: + ΣMA = 0; P (4m) + 20 kN (8m) + 40(14m) – E y (17m) = 0 E y = 0.235P + 42.353 + ΣME = 0; Ay (17m) – P (13) – 20(9) – 40(3) = 0 Ay = 0.765P + 17.65 II. EI AE 1 1 1 1 1 1 1 6(430) 6 6 2(430) 466 6 12 430 2(466) 6 CM1 M3 L EI EI 2 EI 6 EI 6 32,040 (1728) δBH 10.61 in. P5.35. Given: P = 9 kN. Check the results by moment distribution. The area of 2 2 all bars = 5 in. Include the negative pressure on the leeward side. 13-20 Copyright © 2018 McGraw-Hill Education. Dy P6.15. P5.54. 85 kip • ft C B A 15′ 15′ P8.21 1 kip 3 kips/ft P-System (kip-ft) A 85 kip-ft B C δBV Q-System (kip-ft) C A B 30.9 7.5 Shear (kips) Moment (kip-ft) -14.1 159.3 θC Q-System (kip-ft) 126.3 Moment (kip-ft) -85 x1 δB = = ò MQ 15 1 B Moment (kip-ft) x2 EI 1 2 x1 (-1.5 x1 + 30.9 x1) dx1 + 0 = C M P dx ò2 EI 1 1 kip-ft A 1 15 1 ò 2 x2 (14.083x2 - 85)dx2 EI 0 (7889.1 + 3140.4) EI 11, 029.8 δB = qC = = ò MQ 15 1 EI M P dx EI 1 2 0 = 1 15 1 ò - 30 x1 (-1.5 x1 + 30.9 x1) dx1 + EI ò (30 x2 - 1)(14.083 x2 - 85)dx2 EI 1 0 (-526 - 100.1) EI qC = -626.1 EI 8-26 Copyright © 2018 McGraw-Hill Education. Determine the value of moment that must be applied to the left end of the beam in Figure P8.22 if the slope at A is to be zero. and 30 in. The arch has a constant cross section with A = 24 in.2, 4 I = 2654 in. 20 kips 20 kips C B A D 12′ E 9′ 9′ F 12′ 9′ 9′ P11.17 Half Structure: ΣM = +50.5 DEM -16.88 -16.88 -16.88 Due to symmetric structure & symmetric loads; evaluate half structure: θ£ betw BC = 0 Joint B Distribution Factors 3 EI 3EI = 4 18¢ 72 1 EI 3EI = = 2 12 ¢ 72 DFBA = 0.25 EI 6 EI = 12 ¢ 72 12 EI = 72 DFBE = 0.50 K BA = KB£ K BE = ΣK AB FEM AB, BA = Segment "AB": DFBC = 0.25 1.0 PL 20 k (18¢) = = 45ft·k 8 8 Segment "BC": + + ΣM A = 0; + 50.5ft·k + 20 k (9 ¢) - VBA (18¢) = 0 VBA = 12.8k + ΣMC = 0; -16.88 + 16.88 - VCB (12 ¢) = 0 VCB = VBC = 0 ΣFy = 0; VAB - 20 k + 12.8k = 0 VAB = 7.2 k 11-20 Copyright © 2018 McGraw-Hill Education. Determine the roof slope factor, Cs using the ASCE graph shown in Figure P2.19. A B C 0.6″ 12′ 15′ P10.10 I = 144 in 4 ; E = 29000 ksi; All FEM = 0 2 EI M AB = (12) éë 2(-0.002) + θB ùû = -0.008E + 2 EθB 12 2 EI M BA = (12) éë 2θB - 0.002ùû = 4 θB - 0.004 E 12 2 EI M BC = (12) éë 2θB + θC - 3(0.00333)ùû = 3.2θBE + 1.6θC E - 0.016 E 15 2 EI MCB = (12) éë 2θC + θB -3(0.00333)ùû = 3.2θC E + 1.6θB E - 0.016 E 15 (1) M BA + M BC = 0 (2) MCB = 0 4 EθB - 0.004 E + 3.2θB E + 1.6θC E - 0.016 E = 0 (3) 7.2θB E + 1.6θC E = 0.02 E (4) -1.6θB E - 3.2θC E = -0.016 E θB = 0.001875 rad Substitute θB into Equation (4), Solving for θC θC = 0.004063 rad M AB = -0.008(29000) + 2(29000)(0.001875) = -123.25ft·k = -10.27ft·k M BA = 4(0.001875)(29000) - 0.004(29000) = 101.5ft·k = 8.45ft·k 10-11 Copyright © 2018 McGraw-Hill Education. C P5.49. Comparison of problems P13.21 and P13.22 generate different results in the column moments and shears, and is primarily due to the knee brace in P13.22.
= RA 18 - 90 k ´ 9 ¢ = 0 RA = 45 kips Moment Distribution Results + ΣMC = 0 = 45k (44) + RB 20 - 220(22) RB = 143k ΣFy = 0 = -220 + 45 + 143 + RC RC = 32 kips Results of Approx. Express the equilibrium equations in terms of the appropriate displacements. ΔBO = (10 m)(0.005 rad.) = 0.05 m δBB [xRB] δBB = 0.0278 m (See P9.21) δS = 1 = 0.025 m 40 Compatibility Equation: -Δ BO + δBB RB = -0.025 RB RB = 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 in2 E = 30,000 ksi Geometry ΔCY = 0 Deflections Dco Released Structure w/Applied Load WQ = UQ = ΣFQ FP L/AE é -22.5(18´12) ù æ -5 ÷öé 37.5(25´12) ù ú + çç ÷ ê ú 1k ΔCO = 2 ê êë úû èç 3 ø÷ êë úû AC AE ΔCO = 0.448k δcc WO = UQ = ΣFO2 LAE / Released Structure w/Unit Value of RC also use as Q-System for ΔCO and δCC æ 5 ö (25´12)(2) (2) (15´12) 1k δCC = çç ÷÷÷ + è 3ø AE AE 2 2 æ 4 ö é 20 ´12 ù + çç ÷÷÷ ê ú2 è 3 ø êë AE ûú = 0.054 ¢¢ 2 δCC ΔCO + δCC 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 E 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 æ 600 ÷ö K AB = çç 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.

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. Δ D () = ΔDO () - δDD Dy Dy 1840 196 Dy = K EI EI 1840 Dy = 196 E K EI 200 ´ 40 And = 200 40 K 1840 Dy = 4.65 kN 200 + 196 ΣM A = 0 : RB = 24.84 kN ΔD = ΣFy = 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) f + 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 · k Joint C: M2 -150 = 0 M2 = 150 ft · k é 83.33 ft · k ù ú Force Vector F = êê ú 150 ft k · ëê ûú Stiffness Matrix: Unit Rotation @ B: + Joint "B": ΣM B = 0; K11 - 0.2 EI - 0.133EI = 0 K11 = 0.333EI + Joint "C": ΣMC = 0; K 21 - 0.067 EI = 0 Unit Rotation @ C: + Joint "B": ΣM B = 0; K12 - 0.067 EI = 0 K12 = 0.067 EI Unit Rotation θ2 @ C: + Joint "C ": ΣMC = 0; K 22 - 0.133EI = 0 K 22 = 0.133EI é 0.33 0.067ù ú K = EI êê 0.067 0.133úú ë û 16-2 Copyright © 2018 McGraw-Hill Education. 20' P4.53. 15' P4.54. F D 3m 2m A C B 30 kN 60 kN 4m 4m 4m P4.49 Entire Structure + ΣM D = 0; ΣFy = 0; + C y (4) - 60(8) - 30(12) = 0 C y = 210 kN 210 - 60 - 30 + Dy = 0 Dy = 120 kN + ΣFx = 0; Ax = 0 Isolate Joint A + ΣFy = 0; FAFY = 30 kN FAFX = 40 kN + ΣFx = 0; 40 - FAB = 0 FAB = 40 kN Isolate Joint F + ΣFx = 0; FAFX = 40 k FEFY = 10 k ΣFy = 0; 10 - 30 + FFB = 0 + Isolate Joint B ΣFy = 0; + FBEY - 20 - 60 = 0 FBEY = 80 kN FBEX = 80 kN T + ΣFx = 0; 40 + 80 - FBC = 0 2m FBC = 120 kN C Isolate Joint E ΣFy = 0; + FECy = 0 Continue by method of joints to finish. Dy 5 ft P5.19. Continued Sloving (1a), (2a) and (3a) gives. Determine the shear, axial load, and moment on sections an infinitesimal distance to the left and right of the loads. (c) Sketch the qualitative influence line for positive moment at the midspan of girder AC. Continued Influence Lines 12-54 Copyright © 2018 McGraw-Hill Education. × 8 in. 10' P11.4. Analyze each structure by moment distribution. w = 5 kips/ft C 32 kip • ft A x 4' B x 3' D x 6' P5.7 + ΣM D = 0 = -10 ´13 - 35´ 9.5 + 32 + 9 RB RB = 47.83 kips + ΣFy = 0 = -10 - 5 ´ 4 - 35 + 47.83 + 9 RB RB = 47.83 kips ... Segment BC: 0 £ x2 £ 3 + ΣFy = 0 = -10 - (4 + x2)5 + 47.83 - V V = -30 - 5 x2 + 47.83 V = 17.83 - 5 x2 + æ 4 + x2 ÷ö ÷÷ + 47.83 x2 ΣM Z = 0 = -M -10(4 + x2) - 5çç è 2 ø÷ 5 M = ... 6 ¢) = -49 ft · k 5-8 Copyright © 2018 McGraw-Hill Education. C FUNDAMENTALS OF STRUCTURAL ANALYSIS 5th Edition Kenneth M. 4 kips B C D 10 kips 15' A 8' 8' P5.27 + ΣM A ... Copyright © 2018 McGraw-Hill Education.

w = 3 kips/ft A B 4' C 6' D 5' P5.5 Freebody Diagram Segment Origin Range BC 4 ¢ £ x1 £ 10 ¢ A V Equation M Equation + V1 = 8 - 3 x1 ΣMO = 0; -4 x1 - M1 - 3(x1 - 4 ¢) (x1 - 4 ¢) = ... M2 - 3x2 æ x2 ö÷ çç ÷ = 0 çè 2 ø÷÷ 3 M2 = -16 - 4 x2 - x22 2 5-6 Copyright © 2018 McGraw-Hill Education. This can be a useful tool for the designer in selecting the most efficient truss ...)dx1 + ò 2 x22 dx2 0 ö 12 ÷ ÷ 0 ÷ ÷ø æ 4608 2304 ö÷ 6912 = çç + ÷= è EI EI ø÷ EI 3 δDD ò 24 é æ 3 ö12 ù 3 öù x ÷ ú 1 éê æç -x1 ê 2 ÷ú ÷ 60 çç = + 6 x1 ÷ú - ê120 ççç 2 ÷÷ ú ê ÷ ... 155,520 6912 + D =0 EI EI Y DY = 22.5 kips - Reactions: P-System EY = 120 - 3(22.5) EY = 52.5 kips Q-System 9-50 Copyright © 2018 McGraw-Hill Education.

Compute the reactions and draw the shear and moment curves for the beam in Figure P9.19. Tension in FCE æ 30 kN ö÷ Tmax = (0.3125 + 0.1042) çç ÷ = 6.25 kN çè 2 ÷ø 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 · k M B = -15(60 k) = 900 ft · k C MB 1 kip Cy 0 0 MC 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 -173 kip-ft B 15 kips 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) 0 352.8 15 ... 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 ... narrow reinforced concrete beam. E = 200 GPa, A = 25 × 10 mm , and 6 4 I = 240 × 10 mm . Moment Maximum Shear + ΣM B = 0 = RA 30 - 20 ´ 30 - 20 ´ 20 Vmax = RA = 33.33 kips 12-44 Copyright © 2018 McGraw-Hill Education. 100' 100' A C cable hB B E w = 4 kips/ft P6.3 T = 600 kips H = T2 - Ay2 = (600)2 - (400)2 H = 447.2 kips HhB = M B 20,000.000 ft · k hB = 447.2 k hB = 447.2 ft 6-4 Copyright © 2018 McGraw-Hill Education.

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, DL, EL, and IJ if the live load in Figure P12.29 is applied through the lower chord panel points.

The influence lines for the forces in the members of the frame in Figure P11.24 is applied through the lower chord panel points.

[illegible]

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[illegible]

= 0 çè 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 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 FEM DC = 75 k · ft I I 3æ I ö , K BC = , KCD = çç ÷÷ 10 12 4 çè10 ø÷ K AB 5 6 DFBA = = , DFBC = K AB + K BC 11 11 K BC 10 9 DFCB = = , DFCD = K BC + KCD 19 19 K AB = Shear (kips) Moment (k·ft) Deflected Shape 11-4 Copyright © 2018 McGraw-Hill Education. B A C I 2m D I 2I 2m E 2m 2m P8.24 I = 46 ´106 mm 4 E = 200 GPa Segment I Orgin Range MP = MQ AB = DE I A 0 ≤ x1≤ 2 9x BC = CD EI B 0 ≤ x2 ≤ 2 9(x + 2) P-System Q-System By Symmetry, Double EIQ From “A” to “C” B C dx dx + ò M P2 EI AB EI BC B ΣQ · δC = 2 ò M P 2 A 2 2 186C = 2 ò (9 x)2 0 2 dx dx + 2 ò éë 9(x + 2)ùû EI AB EI BC 0 2 é 81x 3 ù é 3 ù é + (2)81 ê x + 4 x + 4 x ú = 2ê ê 3EI ú ê ë 2 û0 EI ë 3 û0 108 6C = EI 108(1000)5 δC = (46 ´106) (200 ´106) 2 2 P-System = Q-System for δC 6C = 11.74 mm 8-30 Copyright © 2018 McGraw-Hill Education.
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 w = 2.4 kips/ft 9 kips/ft E D C F 10′ 10′ 12′ 10′ P11.10 Consider Symmetry: Analyze 3 I 4 (20) 1 1 KCD = K DC = 2 (32) ΣK 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 = FEMCB = FEMCD = FEM DC E A 6′ Modified Stiffness 8 kips 11-12 Copyright © 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: θC = -θB : Fixed-End Moments: -0.8(16) = -10.24 kip-ft 20 2 -0.8(16) = = 6.83 kip-ft 30 -0.2(14)2 = = -3.27 kip-ft 12 2 FEM AB = -FEM DC = FEM BA = -FEMCD FEM BC = -FEMCB Equilibrium of Joint B: M BA + M BC = 0 2 EI 2 EI 0= (20θB + 0) + 6.83 + (20θB - θB) - 3.27 16 14 -9.06 θB = 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-ft MCB = 4.56 kip-ft MCD = -4.56 kip-ft M DC = 11.37 kip-ft 1.4 MCB B VBC 0.35 -1.7 NCB NBC VBA 2 EI 2 EI (0 + θB) -10.24, M BA = (2θB + 0) + 6.83 16 16 2 EI M BC = (2θB - θB) - 3.27, MCD = (-2θB + θB) + 3.27 14 14 2 EI 2 EI M = (0 - θB) - 6.83, M DC = (2θB + 0) + 10.24 16 16 M AB = -1.4 -4.55 C VCB (Symmetric) MBA 4.84 NBA MBA VBA 7.667 ft (Symmetric) B -4.69 -11.4 Moment (kip-ft) Shear (kips) 90° A 90° VAB MAB NAB MAB VAB Ax MA Deflected Shape Ay 10-14 Copyright © 2018 McGraw-Hill Education. G H F 6′ 18′ A B C D 4 @ 24′ = 96′ P12.32 FHC = -0.625(12 k) + 0.416 (12 k) + 0.208(12 k) = 0 12-35 Copyright © 2018 McGraw-Hill Education. 30′ D F C 30′ 30 kips A B 40′ 40′ P4.48 FBD: Right Side of Section (1) Entire Structure: + ΣFA = 0; -By (40 ¢) + 30 k (80 ¢) - 30 k (120 ¢) = 0 By = 30 k ΣFy = 0; + - 30 k - 30 k + 30 k + Ay = 0 Ay = 30 k + ΣM" o " = 0; + ΣFx = 0; EFx (30 ¢) - 30 k (40 ¢) = 0 + ΣFx = 0; Ax = 0 EFx = 40 k æ 3ö EFy = 40 k çç ÷÷ = 30 k è 4 ø Remaining Bar Forces Solved by Method of Joints. RA = 9.49 kN 58 80 æ 5 ðæ 1 6 200 RA = ´ çç ÷÷ ç ÷÷ ÷ = EI çè 3 ÷øèç 5 ÷ø 3EI 6 5 ´80 çæ 5 M D = RA ´12 çç ÷ + 1840 ç ´ 4 + 3÷÷ = EI 2 EI è 3 ø ÷ø ÷ 4 ´80 çæ 2 1840 ç ´ 4 + 3÷÷ = EI 2 EI è 3 ø ÷ø 3EI ΔD = 1840 ΔD = EI Calculate Δ DO by “Conjugate Beam.” Pa 2 (L + a) (See Table A.3) 3EI 1(7)2 (12) 196 = = 3EI EI 6DD = 6DD 9-22 Copyright © 2018 McGraw-Hill Education.
The location of the P.I. in span CD is 3.9′ from D rather than 5′ assumed. Member EB is straight ∴ θB = θE = 0 ∴ Joints B and E can treated as fixed supports. 6′ 9 kips P12.53. Select RC as the Redundant æ 2 x ö dx æ 2 x ö dx æ 2 x ö dx 1kN · δCC = ò çç ÷÷÷ + ç ÷÷ + ò çç ÷÷ + ò ç 3 ø 2 EI ò çè 3 ÷ø EI ò çè 3 ÷÷ø EI 0 0 6 2 2 æ x2 ö 1 4 x3 δCC = çç ÷÷ + ÷ çè 3 ÷ø EI 9 3EI 3 6 3 6 2 + 0 x3 27 EI 6 0 28 EI Compatibility Equation: ΔC + RC δCC δCC = Structure with Redundant ΔCO + RC δCC = 0 599 28 + RC = 0 EI 6 EI RC = 19.61 kN · Compute Reactions Using FBD Compute RD. , and E = 24,000 kips/in. 2 2 A = 2 in. w = 40 kN/m P10.14.
P12.39. P6.33. 30 kips P4.33. Compute the support reactions and the maximum value of if the allowable tension force in the cable in Figure P6.10 is 200 kN. Notice that sideway is possible because the load is unsymmetric. 10 kips P3.1. Determine the reactions for the structure. Draw the shear and moment curves for 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 10(10)3 = = 0.0278 m () 3EI 3(200)(60) 1 = 0.025 m () δS = 40 Compatibility Equation δBB = -Δ BO + δBB 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 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 ≥60 Cp −0.9 −0.7 −0.4 −0.3 −0.2 −0.2 0.0 0.2 0.2 0.3 0.4 0.01θ* 10 15 −0.5 −0.5 ≥20 −0.6 Consider Positive Windward Pressure on Roof, i.e. left side. 4-47 Copyright © 2018 McGraw-Hill Education.
+ + ΣM A = 0; 5k (15¢) - M BA - 45ft ·k = 0 M BA = 30 ft ·k ΣM A = 0; 10 k (15¢) - Dy (30) = 0; Dy = 5k + ΣFy = 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”: + ΣFx = 0; + RAx = 0 ΣMb = 0; M A - 4 kN 3m = 0 M A = 12 kN · m + ΣFy = 0; - 4 kN + Fb = 0 Fb = 4 kN Freebody of Member “BD”: + ΣMd = 0; - 4 kN (10 m) + Rc 6 m - 2 kN · m = 0 Rc = + ΣFy = 0; 42 = 7 kN 6 - 4 kN + 7 kN - Rd = 0 Rd = 3 kN ÷ Rd Entire Structure: 3-8 Copyright © 2018 McGraw-Hill Education. éê 2 For Member 2 2 S = sin f = 1, C = cos f = 0 é8.33´10-4 SC (-N + P) -QS ù ù é ê ê 0 NS 2 + PC 2 -QC ú = 125,000 ê ú ê 4 ú ê -0.05 úë ëê f = 0, 6 0.03 0 ê 2 [k F] = 125,000 ê 0 8.33´10-4 ê ê 0 -0.05 éë S = sin f = 0, -0.05úú ú 0.03 0 ú ú 0 4 úú ûú C = cos f = 1 0 ú ú ú -0.05ú ú 4 úú û [K] = [k1F] = [k 2N] ê ê 3854.1 0 -6250 úú ê ê 3854.1 -6250 úú ê éê-6250 -6250 1,000,000úû 16-13 Copyright © 2018 McGraw-Hill Education. 10′ P12.14. w A B L P7.1 Analysis by Double Integration w(L - x)2 = M 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 Ely = 2 3 12 At x = 0, y = 0 \ C2 = 0 (1) (2) Compute DB ; Set x = L in Eq(2) 1 æç wL4 wL4 wL4 ö÷ + + DB = y = + ç2 EI çè 3 12 ÷ø DB = - wL4 BEI Compute θB ; Set x = L in Eq(1) æç wL3 ÷÷ θB = çç-wL3 + wL3 2 EI çè 3 ÷ø 6 EI θB = ç-wL3 + 1 - ÷÷÷÷ = 2 EI è 3 ø 6 EI θB = - D at wL3 6 EI 1 L 22.59 2 = = 0.35 Compare 1 D at B 8 D at L L ; Set x = in Eq2 2 2 17wL4 wL4 D== 384 EI 22.59 7-2 Copyright © 2018 McGraw-Hill Education. Determine the forces in all bars.
Factors AB = = 0.35 .1099 I Distr. F E D 24 kips C 15′ B A 24 kips 3 @ 10′ = 30′ P4.10 ΣM A = 15Gx + 10(24) + 30(24) = 0 Node F: ΣFx = -64 + 48 - AF cos(56.31) = 0 AF = -28.84 kips (C) ΣFy = 28.84 sin (56.31) + BF = 0 Gx = -64 kips ¬ ΣFy = Ay - 24 - 24 = 0 BF = 24 kips (T) Ay = 48 kips F Gx EF GF ΣFx = Ax - 64 = 0 ED CD 24 kips FC AG E DC AF Ax = 64 kips E C BF BC Node E has no applied loads, three members, and two are collinear, therefore: EC = 0 kips AG Likewise from Node C , FC = 0 kips AF B AB 24 kips A Node D: ΣFy = -24 - CD sin (26.57) = 0 Angle of AF = Angle FC = tan−1 (1.5) = 56.31 deg. In the case of trusses, the axial stiffness of members (a function of a member’s cross-sectional area) that make up each load path will influence the percentage of the force in each member of the load path. (c) Determine all support reactions produced by the 15-kip load. Compact fill FUNDAMENTALS OF STRUCTURAL ANALYSIS 5th Edition Kenneth M. w = 2 kips/ft P5.29. θA = 0 θ + αM = 0 -180 4.5(M) + =0 EI EI M = 40 kN · M Compute θ Use Moment-Area Shear and Moment Diagrams 9-20 Copyright © 2018 McGraw-Hill Education. 2) into (1) By = 108 kN Entire Structure: ΣFy = 0; - 6(24) + (By = 108) + Ay = 0 Ay = 36 kN H = 216 kN Max Tension in Cable Occurs @ B: Entire STR: TMAX = (108)2 + (216)2 = 214.5 kN + 6(24)2 + H (4) - By (24) = 0 2 By = 108 kN ΣFy = 0; Ay = 36 kN ΣM A = 0; By Cable Theory WL2 6(24)2 = = 432 8 8 M 432 H= C = = 216 kN HC 2m MC = 6-9 Copyright © 2018 McGraw-Hill Education. 4 = W(100′ 90) 90 lb/ft 2 + (100′ 90) 70 lb/ft 2 = 3,870,000 lbs = 3,870 kips V= V= Vmax SD1 W T (R / I) 0.4 (3870) I = 1 for office bldgs. Analyze the frame in Figure P11.18 by moment distribution. Calculate the vertical hydrostatic load on the 5100-lb empty shipping container in Figure P2.19 subjected to a tsunami inundation height of 3′. 5′ A P12.8. Hoist load moves along beam AB shown in Figure P12.8. Draw the influence lines for the vertical reaction at C, and moments at B and C. In addition the girders are to be designed to support a live load of 0.6 kips/ft that is to be positioned to maximize the design forces in individual members. (b) Using an origin at A, write the equations for shear and moment in member AB in terms of the applied load and the distance x. 20 kips 20 kips w = 3 kips/ft A B 10′ D 20′ 3@10′ 30′ P11.5 3(20) wL2 == -100k · ft 12 12 = 100 k · ft 2 FEM BC = FEMCB 2 (20)(30) 2 PL = -133.33 k · ft 9 9 = 133.33 k · ft FEMCD = FEM DC 3æ I ö I K BC = çç ÷÷ ÷÷ ÷ ÷÷ 4 çè 20 ø 30 9 8 DFCB = , DFCD = 17 17 Shear (kips) Moment (k·ft) Deflected Shape 11-6 Copyright © 2018 McGraw-Hill Education. P9.47. 2-2 Copyright © 2018 McGraw-Hill Education. P12.41. 36 kips 24 kips I H G F E 20′ B A 15′ 15′ C D E 15′ P4.12 FBD Right Side + ΣMC = 0; (24 k)(30 ¢) - FHG (20 ¢) = 0 FHG = 36 k + ΣFy = 0; C y - 24 = 0 C y = 24 k Entire Structure: + ΣM A = 0; - By (15¢) - 24 k (30 ¢) + 24 k (60 ¢) - 36 k (15¢) = 0 By = 12 k ΣFy = 0; Ay - 36 k + 12 k + 24 k - 24 k = 0 Ay = 24 k 4-13 Copyright © 2018 McGraw-Hill Education. Construct the envelope for both maximum positive and negative moments for the beam. If roof trusses are spaced at 16 ft on center, what is the uniform snow load along a truss?
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 + ΣM B = 0; M BC + M BA + 16.8 = 0 ΣM B = 0 ΣFy = 0; VBC - 67.2 + 37.1 = 0 (2) Substitute Eq’s (1) into (2) 4 E (4 I) 4 EI θB -134.4 + 0 + 50.4 = 0 24 12 B 84 EIθB = 84 θB = (3) EI Substitute (3) into Eq’s (1) 4 E (4 I) ö 84 ù ö 84 ù é -134.4 = -78.4 k · ft 24 êë EI úû 4 EI æç 84 ö÷ ÷ = 28 ft · kips ç ÷ + 12 çè EI ø÷ VBC = 30.1 kips Axial Force in BA FBA = 16.8 + 30.1 FAB = 46.9 kips Shear in Column AB + ΣM 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 qhGCp θ h qzGCp Section (b) P2.13 TABLE P2.13 Roof Pressure Coefficient Cp *θ defined in Figure P2.11 Windward Leeward Angle θ 10 15 20 25 30 35 45 Cp −0.9 −0.7 −0.4 −0.3 −0.2 −0.2 0.0 0.2 0.2 0.3 0.4 ≥60 0.010* 10 15 −0.5 −0.5 See P2.13 Solution Windward Roof (Negative Pressure) 33.7 Interpolate between 30 and 35 for negative Cp value in Table P2.12 C p = -0.274 p = qh GC p = 21.76(0.66) 0.85(- 0.274) = -3.34 lb/ft 2 (Suction) Note: Wind pressures on other 3 surfaces are the same as in P2.13. If support A in Figure P11.23 is constructed 0.48 in. For each truss, determine the following: (a) All bar forces, indicate tension or compression. P = (0.55 wL + 0.50 wL ¢)(4) = [0.55(3.6 ´ 20) + 0.50(3.6 ´ 24)]4 = 331.2 k Moment: Since the spans of the griders are nearly the same on each side the girders apply moments that are approximately Equal and opposite to each other; therefore, the moment created at the top of the column is very small, say M = 0 . Continued Axial Loads, Lebels: Member & Joint Nos.
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.
In the case of the arch in Fig. C B P3.3. Determine the reactions for the structure. Area of bar 2 2 1 = 2 in. (x = 8 m) 1 æ 8 öæ 3 ö tC/B = (2) çç ÷÷çç ´2÷÷ = 0.534 mm 3 çè EI ø÷ è4ø 200 ö÷ ÷ 1 = 5 t = B/A = (10) çç ÷÷ (7.5 - 2) ç 3 è EI ø÷ 1 æ 8 öæ 2 ö - (2) çç ÷÷çç ÷÷ ÷ 3 èç EI øèç 4 ø ΔC = (2m)(θB) - tC/B ΔC = 3.026 mm () 3664 () EI PL3 1(8)3 512 Δ BB = = = () 3EI 3EI Compatibility Equation = Δ BO - δBB RB = 0 Shear RB = 21.5 kN () Moment 9-3 Copyright © 2018 McGraw-Hill Education. The objective is to compare the behavior of a determinate and an indeterminate structure. C P4.7. Determine the forces in all bars. Cx C x Q-System Q-System B C (kips) (kips) em P-System (kips) 10 kips 1 in.2 Cy Q-System Cy C E -0.648 (T) 0.25 in.2 B C (kips) 3.898 (T) 1 kip 0.375 (T) Q-System D 0.273 (T) -5.312 (C) B A E 0.727 (T) A E -0.205 (C) A 0.25 in.2 D -0.545 (C) 15 kips Dx Dx Q-System 0 B Q-System C (kips) (kips) E 1 (T) A 1 kip 1 (T) D Member AB BC CD DE EA EB EC FQ (kips) FP (kips) –13.920 –5.312 –4.830 3.898 –0.648 13.133 4.556 L (ft.) Cx Cy Dx 0.455 0.500 –0.455 0.273 0.727 –0.429 0.429 0.341 0.375 0.909 –0.545 –0.205 –0.322 0.322 0 0 0 1 1 0 0 FQ 2 A (in. 39 kips P8.17.
Solving the compatibility equation yields XC , beem = C y = 5.44 kips 9-47 Copyright © 2018 McGraw-Hill Education. For this case assume that all truss members have the same cross-sectional area. Equations at Joint B: M BA + M BC = 0 at Joint C: MCB + MCD = 0 at Joint D: M DC = 0 Shear Equation: V1 + V2 = 0, Where M AB + M BA 20 MCD + M DC V2 = 15 V1 = 10-39 Copyright © 2018 McGraw-Hill Education. Given: 2 E = 30,000 kips/in. Analyze the reinforced concrete box in Figure P11.13 by moment distribution. 3 40 kips 4 6 1 3 12′ 5 4 1 2 16′ P15.5 Member 1 and 3 : AE (30,000) =2 = 416.7 k/in L (12 ¢ ´12%) fx = 90 , cos fx = 0, sin f x = 1, cos2 fx = 0, sin 2 fx = 1, cos fx (sin fx) = 0 (3) (4) (5) (6) 1 2 7 8 é0 é0 0 0 ù 1 (3) 0 é ù é é0 ù 0 ê (4) 0 1 ¢ (K3) = 416.7 k/in éê ù 7 (5) · 250 = é 0 0 0 0 0 é ù é é 0 -1 0 ú é ù 0 ú ê 0 0 -1 0 ú ê 0 1 0 1 ¢ (K1) = 416.7 k/in éê ù 7 (6) ... AE (30,000) Member 2 and 4 : =2 = 312.5 k/in L (16 ¢ ´12%) 0 0ù ú 0 -1.67ú ú 0 0ûú 0 1.67úú fx = 0 cos fx = 1, sin fx = 0 cos2 fx = 1, sin 2 fx = 0 cos f sin f= 0 (1) (2) (3) (4) 7 6 1 é ê ê 0 K 2 = (K 4) = 312.5 k/in éê 6-1 ê 0 0 é ê Member 5 : 8 5 6 0 -1 0 ú ú 0 0ú ú 0 0ûú ú 0 1.25 é é 0 1 · = éë 5 (3) 250 ç÷ 1.25 é 6 (4) 0 éê 7 (1) 8 (2) 0 -1.25 0ú ú 0 0 ú 1.25 0ûú ú AE (30,000) =2 = 250 k/in L (20 ¢ ´12) 20 = 0.8, sin fx = 0.6, C 2 = 0.64, S 2 = 0.36, CS = 0.48 25 1 2 5 6 é 0.64 0.48 -0.64 -0.48ù 1 é ú é é 0.48 ú0 0.36 0.48 0.36 ú K 5 = 250 k/in éê 0 ê 0.48ùú 6 -0.64 -0.48 é -0.48 -0.36 0.48 0.36ùú 6 éê ú 0 cos fx = 15-11 Copyright © 2018 McGraw-Hill Education. Results for moment at B: Mb = 50 ft -k in P13.22 vs Mb = 75 ft-k in P13.21, and horizontal force component of the knee brace = 15k vs 20 k in the bottom chord of the parallel chord truss, resulting in a 33.3% increase in moment and horizontal force at B. Determine the forces in members, a, b, and c by passing vertical section 1-1 through the center of the truss. 6 kips 3 kips/ft A 3′ B D E 6′ 18′ P5.23 Compute Reactions: + ΣM E = 0; 6 k (15¢) + 3KLF (18)(9 ¢) - Dy (18) = 0 Dy = 32 k ΣFy = 0; + - 3k / FR (18¢) + 32 k + E y = 0 E y = 22 k ΣFx = 0; Ex = 6k ¬ 5-25 Copyright © 2018 McGraw-Hill Education. Consider the beam in Figure P9.21 without the applied load and support settlement. Given: EI = constant. For the truss in Example 8.7, use the RISA-2D computer program to (a) recompute the vertical deflection of F if the areas of members BC, CD, DE, EF, FA, FC, and CE are doubled. Loads, Structure Indeterminate to 1° Select “M” as Redundant α = θE + Δθ E - A 1 4.5 α = 0 + (4.5) = EI EI Compatibility EQ.
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 + ΣM 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 = BL L 20 ¢ FBL = 20 k + ΣFx = 0; - VCB 20 k - 5k = 0 VCB = 15k ¬ Truss Bar Forces by Method of Joints: Joint “C” Joint “L” + ΣFy = 0; -6.67 + CLy = 0 CLy = 6.67k C CLx = 6.67k ¬ FCL = 9.43k C + ΣFx = 0 10 k + 15k - 6.67k + FCD = 0 FCD = 18.3k C 13-42 Copyright © 2018 McGraw-Hill Education. Draw the influence lines for shear between supports 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 δBH = 0 3 δBH = 1k WQ = UQ k - 2 (1.5k) + 1k δBY = 0 2 3 δBY = 4 WQ = UQ 1k (1.5k) + 1 ft·k ΔθBC = 0 30 æ1ö 1.5 çç ÷÷÷ ÷÷ çè 30 ø æ ΔθBC = 1k ΔθBC = 0.004167 Radians - 8-33 Copyright © 2018 McGraw-Hill Education. + ΣFx = 0; 15k - FAJ = 0 FAJ = 15k compr.
P = 60 kips P9.45. 9 kips/ft P = 15 kips 25 kips 3 4 E B D A C 4′ 8′ 8′ 4′ P3.11 + ΣM A = 0; 20(4 ¢) + 54(16 ¢) +15(24 ¢) -16C y = 0 C y = 81.5k + ΣFx = 0; 15 - Ax = 0 Ax = 15k ¬ ΣFy = 0; + Ay - 20 - 54 + 81.5 -15 = 0 Ay = 7.5k Isolate Member Be + ΣMb = 0; 54(8) + 15(16) - Dy (12) = 0 Dy = 56 k ΣFy = 0; + Ay + 56 - 54 -15 = 0 Dy = 13k + ΣFy = 0; D x = 0 3-12 Copyright © 2018 McGraw-Hill Education. See, for example, page 240 of the textbook; the highway bridge is composed of an arch, which also forms the lower chord of a truss composed of diagonal members and the girders of the roadway. = 0.45 16 80 3 I 6I K BC = 3 = 2 20 80 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 ΣFy = 0; V1 + V2 = 120.2 k + ΣM 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 the floor system (joints 12 to 20). For the parabolic arch in Figure P6.17, plot w = 6 kips/ft the variation of the thrust T at support A for values of h = 12, 24, 36, 48, and 60 ft. 6′ 15 kips P9.7. Recompute the reactions, draw the shear and moment curves for the beam in Figure P9.6 if support B settles by 1.5 in.. What is the required area of each cable if the allowable 2 stress is 110 kips/in. , and members AE, ED, BE, and 2 E Fave area = 0.25 in. The horizontal load P can act at any location along the length of member AC shown in Figure P12.26. 4 kips/ft 4′ B E D 6′ 6′ 3 kips/ft 6′ 6′ P5.49 Segment BE Segment ABC Deflected Shape 5-51 Copyright © 2018 McGraw-Hill Education. 30 kips w = 6 kips/ft A 12′ B C 18′ 24′ P14.6 Slope θ2 is Unknown Displacement. -24 (C) 0.75 (T) 36 (T) 0 60 (T) F Member FP (kips) AB BC CD DE EF FA AD DF DB 36 36 –60 –75 60 60 15 0 –24 60 (T) E 0 D B 0 1 (T) A F FP L AE 1(6Cx) - (1)(0.4) - (0.75)(0.6) = 0.615 δCx = 1.465 in. 3I I K AB = = = .25 I 0.6 43 4 I K BC = = 0.167 I 0.4 6 ΣK S = 0.41 Assume P.I. at .3L to Left of Support B. (b) Using RISA-2D, recompute the reactions and generate the shear and moment curves when segment AB has 2EI. A tension rod connects A and C. Draw the shear and moment curves for members AB and AF. This permits us to consider two bars AC & AD rather than all 3 bars. ΔH 46.67 ft -10.158 qA = K 19.136 qB = K -4.938 qC = K 0.412 ΨA = K o 25 ft P10.29. Determine the forces in all bars of the truss in Figure P4.27. , area of girder = 10 in. (a)
Compute the horizontal displacement of joint B produced by the 240-kN load in 2 Figure P8.10. Estimate the approximate value of axial load and the moment at the top of columns AH and BG. Analyze and compare two trusses, arched configurations of the Pratt Truss and the Howe Truss in Figures P4.56 (a) and (b), respectively. Calculate the axial forces produced by the live load in column C3 in the third and first stories. Note that support D can translate in the horizontal direction only. Since Opposite in Direction to = 0+ Use Symmetry to Simplify 1. Determine the absolute maximum value of live load shear and moment produced in a simply supported beam spanning 40 ft by the wheel loads shown in Figure P12.45. If the horizontal displacement of joint B of the frame in Figure P8.40 is not to exceed 0.36 in., what is the required I of the members? To reduce deflections, the designer has several choices: 1. 5-61 Copyright © 2018 McGraw-Hill Education.
, 4 −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 = 0.451C y + 0.893C x - 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 0.003P –0.895P –100′ –71.44′ 16.06′ 0.0915P 0.053P –0.106P 0.006P 1.916P –75′ –61.87′ 25.63′ 0.146P 0.085P –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 = Where V1 A is computed in (b) (b) ΣMcentroid : 100(2) = 8F2 + 8F4 + 13F1 + 13F 5 13 200 = (V1 2 A 8)2 + (V1 A13)2 8 200 V1 A = = 2.694 74.25 13 (c) ΣM base : 100(10) + 300(6) = (V1 2 A 8)2 + V1 A13(2) 8 2800 = 37.71 V2 A = 74.25 13 æ 2800 ÷ö F1 = F5 = çç ÷ = 61.3 kN 8 çè 74.25 ÷ø 13 æ 2800 ÷ö F1 = F5 = çç ÷ = 61.3 kN 8 çè 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) Cable Force (kips) Deflection (Max. + + + = 2 EI EI EI AE EI AE Since MQ,EH and MQ,BH are equal and no axial loads are developed in BCED due to applied loading, δBH = δEH : δEH 10.61 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 æç ç2 θ - ÷÷ + (2 θB + 0.005) + 75 = 0 4 çè B 2 ø 3 EI æç 3 θB ö÷ EI ç ÷÷ + (2 θB + 0.005) = -75 2 çç2 ÷ø 3 θB = -0.011767 rad θC = - θB = 0.005884 rad 2 Substi. 0.75 180° 0 50 90.96 0 1 0 0 0 Apply uniform load 2 kip / ft & compute reactions and internal forces of: é1 ù 2 C y = Approx ê (1) 90.96 ¢ + (0.206)(90.96)ú 2 k / ft êë 2 úû 3 = 115.94 k Check using EQ (b.) æ1ö wç ÷÷ ÷÷ çè 2 ø wL2 using Px2 = & Pa¢ = 2 2 2 w(50) w(90.96)2 C y = -0.0092 + 0.0161 2 2 2 C y = 110.2 k Influence Lines: Ay = 2 k / ft (90.96 ¢) - C y = 71.71k 1 C x = (0.656)(90.96 ¢)(2 kip/ ft) = 59.7k ¬ 2 Ax = C x = 59.7k Internal forces of using EQ’s & P = wL N F = 0.714(59.7k) + 0.7(110.2) - 0.714(2 k / ft)(14.3¢) N F = 99.2 kips compression.
Also determine the horizontal 4 displacement of joint B. (b) Determine the cable sag at point B. w = 2.4 kips/ft 8 kips B C A bolted joint C 62′ P3.27 + ΣM A = 0; + ΣFy = 0; + ΣFx = 0; 78¢ + 8 x ´ 28¢ - Rdy 62 ¢ = 0 2 Rdy = 121.37 kips 187.2 k ´ RAy + Rdy -187.2 k = 0 RAy = 187.2 -121.37 = 65.83 K - RAx + 8 = 0 \ RAx = 8K ¬ 3-28 Copyright © 2018 McGraw-Hill Education. I 3 I 3I ´ = = 30 4 40 120 I 3 I 5I = ´ = = 18 4 24 120 8I ΣKB = 120 K AB = K BC 3 8 5 8 Pab 2 1(18)12 2 = = -2.88 L2 30 2 Pba 2 1(12 ´182) = 2 = = 4.32 kip · ft 30 2 L Pab 2 1(12)6 2 == -1.333 k · ft 2 L 182 Pba 2 1(6)12 2 = = = 2.666 k · ft 324 182 FEM B = FEMCB FEM AB = FEM BA + Ordinate of I.L. at = –1 kip.ft ΣM 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, and (c) the horizontal displacement of joint D. 40 kips P8.12. P10.8. If no vertical deflection is permitted at 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 30 kips 4 @ 20′ P4.41 + ΣM B = 0; 60 ´ 20 - FIHX ´ 30 = 0 FIHX = 40 K FIHY = 30 K Joint H Consider Symmetry 4-42 Copyright © 2018 McGraw-Hill 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 16.9Px2 = 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 lines for reactions, construct 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 ΔCS = –0.5 in. Continued Q-System æ Wx 2 ö÷ dx ÷÷ 1· θC = ò 1çççç Px + çè 2 ÷ø EI L 0 é Px 2 Wx 3 ù ê ú + ê 2 6 úû 0 ê PL2 wL3 + θC = 2 EI 6 EI 6(12)2 1(12)3 720 + = θC = EI 2 EI 6 EI L 1 θC = EI Deflection at B æ 6 + x ö÷ Mp = w(6 + x) çç ÷ + P (6 + x) çè 2 ÷ø w = (36 + 12 x + x 2) + 6 P + Px 2 6 1 1k · δB = M M dx EI òò Q P 6 æ ö 1 wx 2 (x) ççç18w + 6 xw + 6 P + Px ÷÷÷ dx δB = ò çè 2 EI 0 ø÷ 6 ö 1 æç w 3 2 2 δB = çç18wx + 6 wx + x + 6 Px + Px ÷÷÷ ò 2 EI è ø 0 1 éê18wx 2 6 wx 3 wx 4 Px 3 úú + ÷ + 3Px 2 + δB = 3 8 3 úû 0 EI ëê 2 1 é = 9w36 + 2 w216 + 162w + 108P + 72 P ùû EI ë 1 é 324 w + 432w + 162 w + 180 P ùû δB = EI ë 918w + 180 P 918 + 180 x 6 = δB = EI EI 1998 δB = EI 6 θB 1k ft · θB = θB = 6 1 EI òM 1 EI ò (1)ççç18w + 6wx + 2 x 0 6 Q · M P dx æ w 0 θB = 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. æ -5 ö FCG = w (negative area) + 24 k çç ÷÷÷ èç 16 ø æ1 ö æ ö -5 = 0.32 ççç (30 ¢ + 15¢) çç ÷÷÷÷÷÷÷ + (-7.5) èç 16 øø÷ èç 2 = -9.75 kips Compression. 8-53 Copyright © 2018 McGraw-Hill Education. Complete the analysis by method of joints.
Select the origin at A. Member end Forces: M AB = -61.2 ft·k M BA = -72.6 ft·k ft· k ft· k M BC = 101.1 MCB = 119.9 ft· k ft· k M BD = -28.3 M DB = -14.2 Segment AB M BA = 26.72 k M BC = 73.4 k Segment BC + ΣFx = 0; ¬ AB VBA - V Segment BD =0 VAB = 26.7k ¬ + ΣFx = 0; ¬ VBC - VCB =0 VCB = 73.4 k ¬ Deflected Shape + ΣM B = 0 (8¢) = 0 -28.3 -14.2 + VDB VDB = 5.3k + ΣFy = 0; VBD = -VDB = 5.3k 10-31 Copyright © 2018 McGraw-Hill Education. 10 kips 10 kips 10 kips B C D 4 kips 6′ E A H G F 4 @ 8′ = 32′ P4.13 Compute RE : ΣM about G + ΣMG = 0; 10 k ´8¢ - 4 k ´6 ¢ - RE 16 ¢ = 0 RE = 3.5 kips + ΣFx = 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 all bar forces. 0.003 rad P9.9. (a) Recompute the reactions, draw the shear and moment curves for the beam in Figure P9.8 if segment AB has 1.5I. 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 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 capacity of a bridge and also reduce deflections and vibrations produced by moving traffic. 5 3(30,000) 3(30,0000)400 = 0.85 kips C æ æ 1 3900 2(20) ö÷ 1 1500 20 2(20) ÷ö 1 300 20 (20) ççç20 + (20) ççç20 + + (20) ÷÷ ÷÷ è è 2 EI 3 ÷ø 2 EI 3 3 ÷ø 2 EI 3 1,920,000 = (1728) ΔCO = 276.48 in. The arch can be stiffened by incorporating a truss. P2.15. FP L = ΣFQ ΔLP AE 2(68.57)(5.66) = -1(.00412) + 500 ´ 200 = -0.00412 + 0.00541 1k · Δ AX = ΣFQ Δ AX = 0.00137 mm Δ AX = 1.37 mm ¬ ‡ Since we know the forces in two bars, we can compute ΔL of each bar. To account for cracking of the reinforced concrete, assume for girder BCD that Ie = 0.35IG; for column AB assume Ie = 0.7IG (compression forces in columns reduce cracking).
(e) What other conclusions can you draw from the study? 15′ 10 0′ 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 (A): 15¢ 24.44 psf (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 ÷øèç 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 + ΣM 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 75(.7407) = 55.56 kN · m + ΣM F = 0 0 = 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) Δ D = 0.834 in. 20 kN 40 kN 40 kN 40 kN 40 kN 40 kN 40 kN 20 kN B C D E F G H O P 3m M N 4m A L K J I 6@4m P4.34 Final Results Joint A Reactions: 240 2 = 120 kN RA - RI = Cut Vertical Section between C & D + Joint B ΣM L = 0 X BA YBA = 4 7 4 X BA = (20) = 11.43 kN (C) 7 FBC = 11.43 (T) FBA = 23.04 (C) -20 ´8 - 40 ´ 4 + 120 ´ 4 + 11.43(7) + X MD 7 = 0 X MD = -34.29 kN C Joint C Cut Vertical Section between D&E X ML = 34.29 kN T 3 X = 25.72 kN (C) 4 MD = X ML = 34.29 kN (T) YMD = YML + Joint D Joint L ΣM k = 0 0 = 120(8) - 20(12) - (40 ´8) -40 ´(4) - 22.86 ´ 7 + 7 X DE X NE = -11.43 kN (C) 3 YNE = - (11.43) 4 YNE = -8.57 (C) X NK = 11.43 kN (T) YNK = 11.43 kN (T) Joint K + ΣFy = 0 = 11.23 + 11.23 - FKE FKE = 22.86 (C) 4-35 Copyright © 2018 McGraw-Hill Education.
= 0 0 = 54.15(49) -165(27.5) + RC 19 RC = 99.17 + ΣFy = 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. » 5.5 in. E P9.48. (Hint: You will need two moment equations: Consider the entire free body for one, and a free body of the portion of truss to either 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 + ΣM A = 0 = 10 ´8 + 20 ´16 + 30 ´32 – 18´ 20 – C y 40 – C x 12 (Eqn1) + ΣM 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 ¬ ΣFx = 0 = Ax -18 -12.5 \ Ax = 30.5 kN + ΣF = 0 = 21.25 - 60 + A \ 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 Δmax 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 ΣFx = 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 VCD = -7.2 Equilibrium of Joint C POI D M1 = 15(6) M = 90 kip-ft 1 MDC = 90 kip-ft FDC = 15 kips 12.5 ft VDC = -7.2 VDC = -7.2 D MDE = 90 kip-ft V1 = 15 kips Equilibrium of Beam CD ΣFx = 0 = 30 - V - FCD ΣM POI = 0 = MCD - VCD (12.5) (left half of CD) FCD = 15 kips VCD = 7.2 kip ΣMC = 0 = M1 - MCD 6) ΣFy = 0 = -7.2 + VDC MCD = 90 kip-ft Therefore, from Joint C: F 2 POI POI 1 16 ft D (whole beam) VDC = 7.2 kip FCB = 7.2 kips 13-24 Copyright © 2018 McGraw-Hill Education. (1) and (2) 625 14625 , θB = 12 EI 108EI = 5.2 k·ft , M BA = 36.4 k·ft , M AD = -5.2 k·ft θA = \ M AB 10-20 Copyright © 2018 McGraw-Hill Education. Assume a pin support at A and a roller at D. Analyze the beam in Figure P11.21 by the moment distribution method. P = 100 kips 15′ B 15′ D A E 20′ 20′ P8.1 All Bars: A = 4 in2, E = 24,000 ksi BAR Fp FQ (δAy) L FpFqL (δAy) Fp2L (δCx) AB 0 +1.67 25′ – – BC +125 0 25′ – 390,625 CD –75 0 30′ – 168,750 DE 0 –1.33 20′ – – AE 0 –1.33 20′ – – BE +150 –2 15′ –4500 337,500 BD –125 +1.67 25′ –5218.75 390,625 Σ = -9718.75 Σ = 1,287,500 ΣFP FQ L δAy : ΣQ · δAy = AE 9718.75(12) 1 · δAy = 4(24000) δAy = 1.214 ¢¢ k ΣFP2 L AE 1,287,500(12) k 100 · δCx = 4(24,000) δCx = 1.61¢¢ ΣCx : ΣQ · δCx = P-System, & Q-System for δCx Q-System for δAy 8-3 Copyright © 2018 McGraw-Hill Education.
D C B E F hinge I 18 kN H A 2m G 4 @ 3 m = 12 m P3.13 Entire Structure: + ΣM A = 0; 12 kN ´3m -18kN ´ 2 m - Rgy 12 m = 0 Rgy = 0 + ΣFy = 0; RAy -12 = 0 \ RAy = 12 kN Freebody Diagram Right Side of Hinge @: + ΣMd = 0; 18kN (2 m) + Rgx (4 m) = 0 Rgx = -9 kN + ΣFx = 0; 2m RAx + Rgx = 9 kN -18kN = 0 9 kN = RAx 3-14 Copyright © 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 12 Member end Moments FEMCB = 16 k·ft FEM BC = - M AB = 20 k·ft , M BA = -20 k·ft 2 EI (2θB + θC) -16, 8 2 EI MCD = (2θC), 8 Equil. Hint: Start with the method of sections. Given: E = 29,000 ksi and 4 I = 100 in. FBD “BCD” + ΣMd = 0; -20 kN (5m) + 60 kN + Dy (15m) = 0 ΣFy = 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 MA = 210.9 kip-ft 6 ft 6 ft KTOT = K S + K S = 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 Y 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 –.003 0 P 1.4 .004 0 J 1.39 –.003 –2.285e–3 C 1.433 .004 1.744e–4 D 1.423 –.015 0 E 1.417 –.017 0 F 1.415 –.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 Axial Forces (See Label Diagram w/Table of MBR Forces) 13-44 Copyright © 2018 McGraw-Hill Education. ΣM A = 0 + Joint B Σ MA = 0 = 12 ´3 - FGCY 4 + ΣFx = 0 = 31.5 - 4.5 - FBCX FGCY = 9 kN tension 9= FGCY 4 = FBCX = 27 kN compr.
Similar analysis of other joints will show all interior bars except 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 L M N F 3m A B 12 kN C 16 kN D E 12 kN 4@4m IJ, MC, and MI P4.39 Forces in Diagonal Bars NI and MC See Section 2-2 Forces in Top and Bottom Chords See Section 1-1 Above + ΣM B = 0; 0 = 20 ´ 4 - FJI 6 FJI = 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 P5.46 ΣF y = 0; + + ΣM A = 0; 2.5(55k) - Ay = 0 Ay = 137.5k 2.5(55k)0.833¢ - E x (50 ¢) = 0 E x = 2.29k ¬ + —— ΣFx = 0; Ax = 2.29k Cantilever Outrigger Beam: Column: 5-48 Copyright © 2018 McGraw-Hill Education. Analyze the frame in Figure P10.20. P7.2. Derive the equations for slope and deflection of the beam shown in Figure P7.2. Assume the moment reactions at each fixed end are as shown. Continued (a) 5 Reactions –3 Eq. of Equil. and A = 11.8 in. Draw the shear and moment curves for each indeterminate beam. However, doubling only AB and BF, in Part (b), only increased volume by 27.5% while deflection decreased by 28.3%.
EI ΔCO = tCA = Therefore, closing the gap due to a unit load at C: 60 kips A C B t CA = ΔCO (ΔCS + ΔCO) + δCC XC = ΔC (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 δFA = 0.004966 δBB = 0.007094 0.002412 0.014472 in X 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.