18 October 2013

Letter of Transmittal

Department of Architectural Engineering

The Pennsylvania State University

Dr. Boothby,

This letter is to prove submittal of Technical Report 3 for the Orchard Plaza Senior Thesis project. All necessary documents are included with this submittal. Calculations supporting my claims are included in respective appendices.

Thank you for assistance with this assignment,

Christopher Duarte

TECHNICAL REPORT 3

ORCHARD PLAZA

AE SENIOR THESIS

Christopher Duarte - Structural
Advisor: Thomas Boothby
October 18th, 2013

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EXECUTIVE SUMMARY

Orchard Plaza is a six story office building with street level retail on the ground and first level. The building is located in southwestern Pennsylvania on the corner of two streets in an urban environment. Completed in 2006, Orchard Plaza is also a LEED Certified building.

The structural system elements include a foundation of caissons, grade beams, and slabs on grade. The site of Orchard building slopes upward from the ground floor on the eastern side to the first level on the western side. Because of this difference in grade, a concrete retaining wall is found along the western half of the building. The gravity system comprises of a system of W-shape beams, girders, and columns that carry all vertical loads to the caissons. The lateral resisting system is composed of six eccentrically braced steel frames that are evenly distributed to resist both North-South and East-West forces.

The façade is composed of four major materials. At its base, Orchard Plaza is wrapped in a limestone veneer that extends up to the second or third level. Next, red-orange brick veneer is found between levels two and five. The sixth level is wrapped in sleek metal paneling, proving a modern crown to the building. All glazing is green in color and gives the building a very contemporary appearance.

SITE PLAN

Orchard Plaza is situated in an urban environment with close proximity to neighboring streets. The building is located in southwestern Pennsylvania. Orchard Plaza was constructed next to an above ground parking garage which accommodates parking requirements for the building. The building is primarily open office space with some retail on the ground and first levels.

CODES

The following codes were used for the design of Orchard Plaza

- 2003 International Building Code
- Minimum Design Loads for Building and Other Structures (ASCE 7-02)
- Building Code Requirements for Structural Concrete (ACI 318-02)
- AISC Manual of Steel Construction, Allowable Stress Design (ASD)

GRAVITY LOADS

A complete estimate of the building's gravity loads can be found in Appendix A

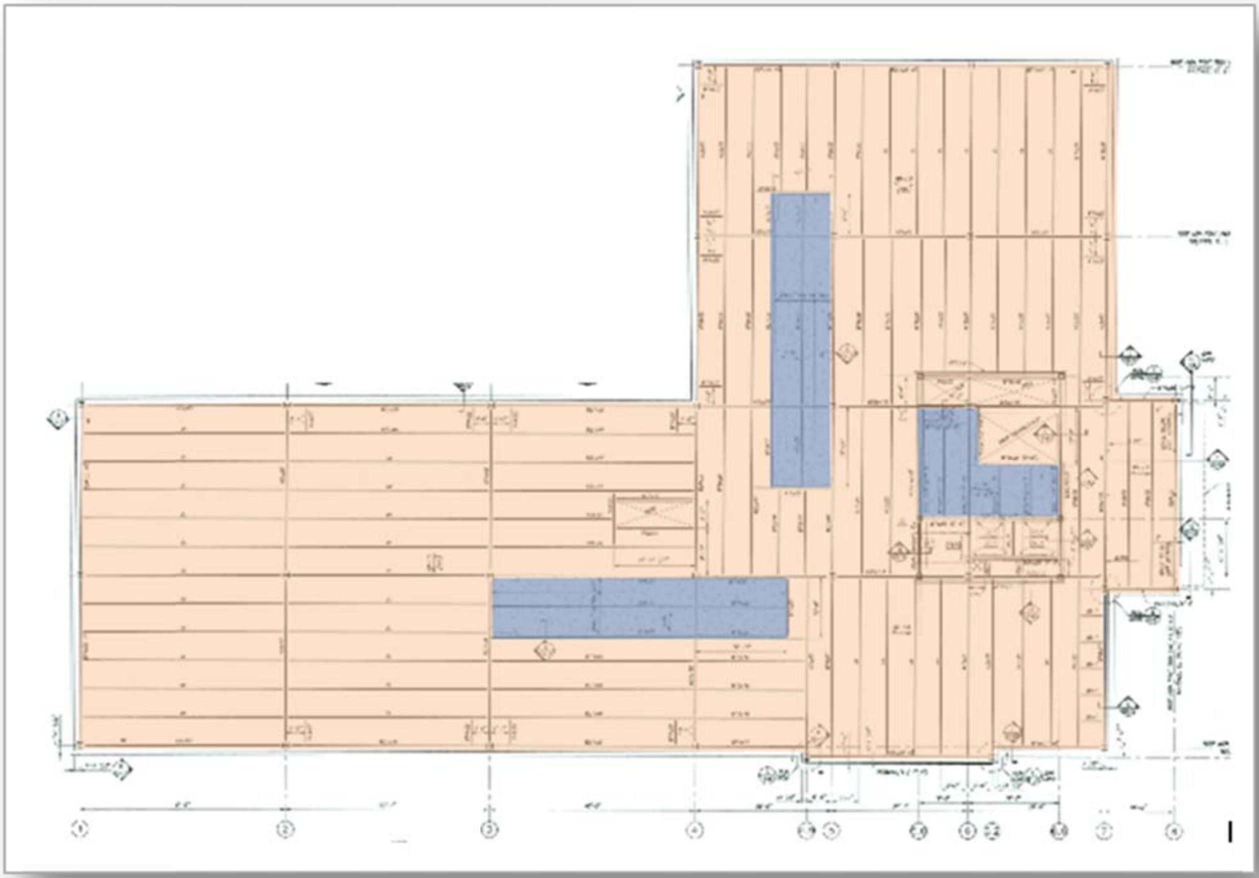
Dead Loads	
Description	**Load (psf)**
Ceiling + Misc. Mechanical	15
Roofing	11
Exterior Walls (Exterior Surface Area)	56
Floor Slab - Level 1	72
Floor Slab - Levels 2-6	66

Live Loads	
Description	**Load (psf)**
Lobbies & Corridors	100
Office Areas	80
Main Corridors Above Ground Level	80
Electrical & Mechanical Rooms	200
Stairs & Landings	100
Light Storage	125
General File Areas	175
Heavy Storage	250
Roof Live Load	30

Snow Loads	
Description	**Value**
Ground Snow Load P_g	25 psf
Flat-Roof Snow Load P_f	18 psf
Snow Exposure Factor C_e	1
Snow Importance Factor I_e	1
Thermal Factor	1
Wind Directionality Factor K_d	0.85

ROOF LOADS

The roof system of Orchard Plaza is comprised of the two components shown below.

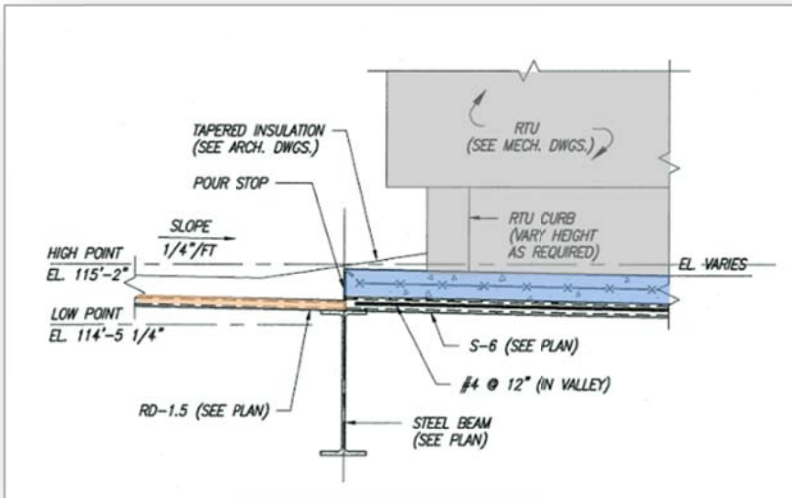


Concrete Mechanical Pad

- 4" Normal Weight Concrete
- 2"-18Gage Composite Decking
- 6x6 – W2.9 x W2.9 Welded Wire Frame

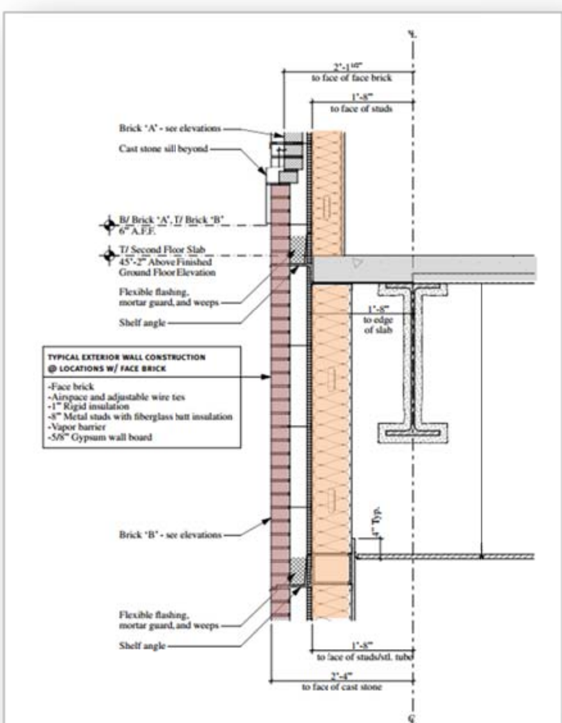
1.5B20 Roof Decking – Vulcraft

A cross section of both the roof decking and concrete mechanical pad is shown below.



EXTERIOR FAÇADE LOADS

Weight of the exterior façade was estimated using ASCE 7-02. Exterior façade loads are estimated to account for forty percent of the total building weight.



ALTERNATIVE FRAMING SYSTEMS

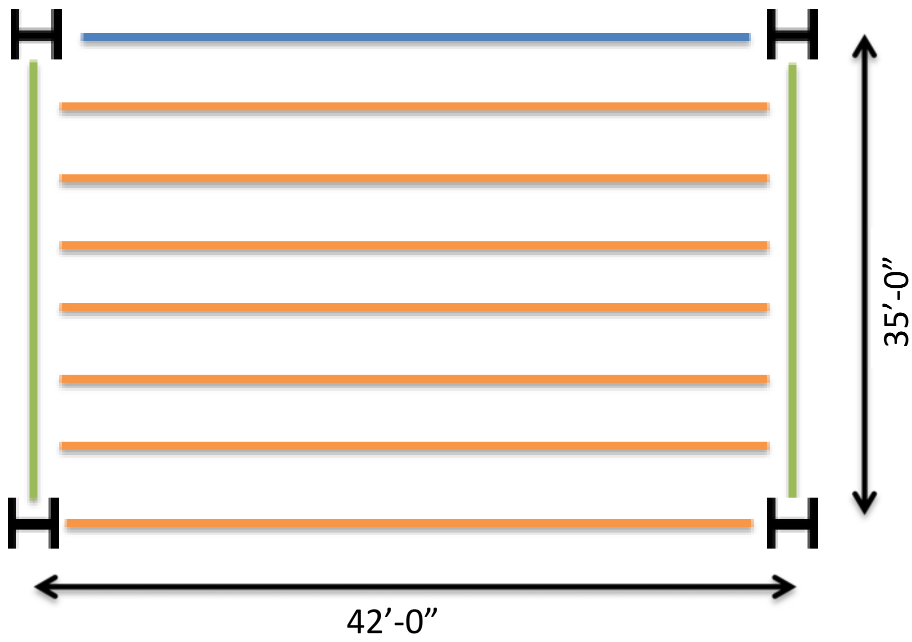
For this report, three alternatives were explored for the floor framing system of Orchard Plaza. The building consists of two standard bay sizes. For this exercise, a 35' x 42' bay was analyzed. The existing floor system uses composite beams and girders. Details regarding existing roof, beam, girder, and column gravity checks can be found in Appendix B.

35" x 42' Bay area coverage – Analyzed for this report

35' x 28' Bay area coverage

SYSTEM 1 – NON-COMPOSITE STEEL JOISTS

The first system explored was non-composite steel joists. This system was chosen for its lightweight design and ease of construction. Also, steel joists function well for spanning long distances. Below is a layout of the joist and girder system. Calculations for System 1 can be found in Appendix C.



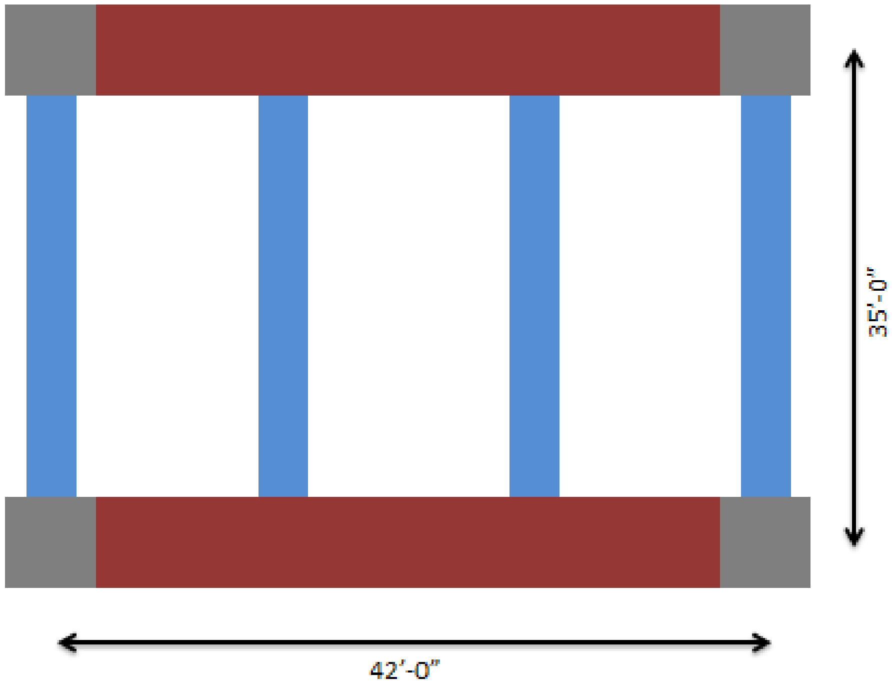
W30x90 [44] for exterior façade support

32LH13 Joist Girders spaced 5' OC

W30x108 Girder

SYSTEM 2 – ONE WAY CONCRETE SLAB

The second system considered was a one-way concrete slab on concrete beams and girders. The bay size was kept at 35' x 45' with two beams placed inside the bay to achieve a 14' tributary width between beams. Calculations for System 2 can be found in Appendix D.



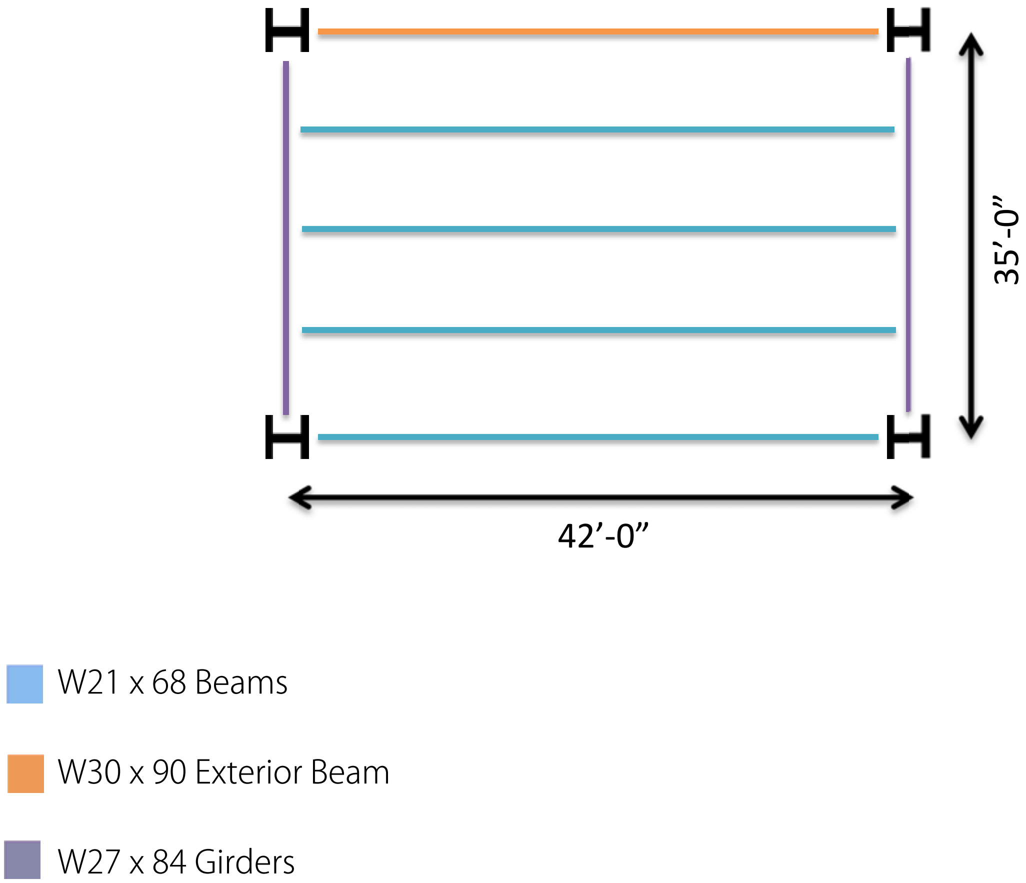
12" x 23" Beam with (3) #6 rebar

24" x 24" Girder with (5) #8 rebar

24" x 24" Column

SYSTEM 3 – NON-COMPOSITE DECKING

A non-composite deck was analyzed to compare to the existing composite deck system. An identical beam and girder layout was used for this system, as well as 3+ span decking as prescribed by the building's specifications. Appendix E details non-composite decking design calculations.



APPENDIX A – GRAVITY LOAD CALCULATIONS

Gravity Loads — Avg Floor Area = 20580 ft^2

Floor Weights ASD

First Level Deck + Concrete = 63 psf – 2VLI18 Vulcraft

Level 2-6 Deck + Concrete = 57 psf – 2VLI18 Vulcraft

Steel Beams (typical)

$28' / 3_{spaces} = 9.33'$

$35' / 4_{spaces} = 8.75'$ ← controls

W21×44 @ 8.75' $\quad 44_{plf} / 8.75' = 5.03$ psf

Steel Girders (typical)

42' spacing

W30×99 $\quad 99_{plf} / 42' = 2.36$ psf

Exterior Wall

Total Surface Area (approximate)

$(213.33')(88')(2) + (144')(88') + (144')(88-18') = 60300\ ft^2$

Assume exterior = 40% bldg weight

$60300\ ft^2 (0.4)(56psf) = 1350.7\ K$

$\frac{1350.7K}{6\ levels} = 225.1\ K/level$

Steel Columns (typical)

W14×159 as average $\left(\frac{\text{heaviest level 1 + heaviest level 6}}{2}\right)$

Avg 14 columns/level $\quad \left(\frac{257plf + 61plf}{2}\right) = 159\ plf$

$159 \times 14'\ story\ (typ) \times 14 = \frac{31164\ lb}{Floor\ Area} = \frac{31164\ lb}{20580\ sf} = 1.51\ psf$

APPENDIX A – GRAVITY LOAD CALCULATIONS

Gravity Loads

Floor Self Weights ASD

First Floor Self weight = 63 + 5.03 + 2.36 + 1.51 = 72 psf

Floor 2-6 Self Weight = 57 + 5.03 + 2.36 + 1.51 = 66 psf

Roof

Concrete Pad Area

2(60.83')(12.75')(4" thick) + (23')(29')(4" thick) = 481 cf conc.

$$481\text{ cf }(150\text{ lb/cf}) = \frac{72129\text{ lb}}{20580\text{ sf}} = 3.5\text{ psf}$$

1.5B20 Gage – Vulcraft = 2.14 psf ⟶ 2.5 psf per ASCE

Roof Total = 2.5 psf + 3.5 psf + 1 psf + 4 psf = 11 psf

1 psf = Acoustic ceiling
4 psf = Mechanical Duct } ASCE 7-02 Table C3-1

Floor Total Weights

Level 1 = 72 psf + 1 psf + 4 psf + 10 psf = 87 psf

10 psf = misc. loads

(87 psf)(20580 sf) + 225.1K = 2016 K (225.1K: exterior wall / level)

Levels 2-6 = 66 psf + 1 psf + 4 psf + 10 psf = 81 psf

(81 psf)(20580 sf) + 225.1 K = 1892 K

Roof = 11 psf (20580 sf) = 227 K

Total Building Weight

227K + 2016 + 1892(5) = 11703 K

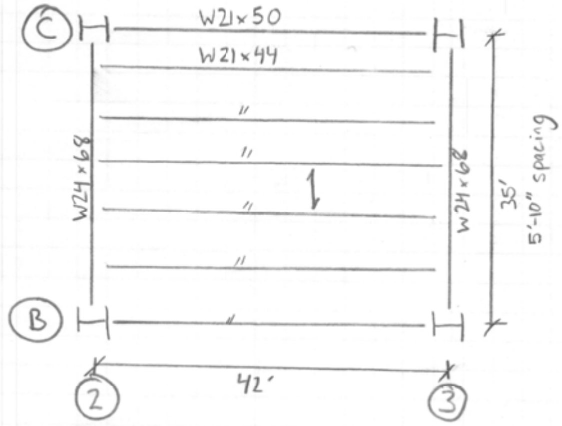
APPENDIX B – EXISTING GRAVITY SYSTEM CHECK

Roof Gravity Check

1607.11 Roof loads

- Flat roof snow load – 18 psf
- Dead Load – 11 psf – Tech II
- Live Load – 30 psf
- Total D+L = 59 psf

Typical Bay



Decking Check

- 59 psf load – not reduced
- 5'10" ⟶ Use 6'0" span in table
- Triple Span or more per specs

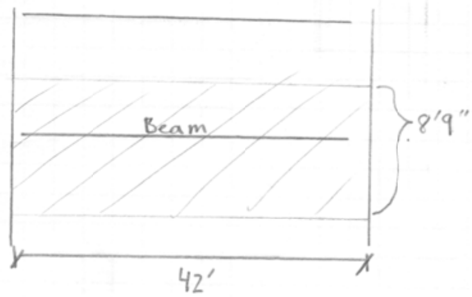
Using Vulcraft 1.5B20 Gage

3+ span – 6'0" spacing ⇒ Allowable Total = 111 psf > 59 psf ✓

APPENDIX B – EXISTING GRAVITY SYSTEM CHECK

Beam Gravity Check   W21x44 [21]   camber = 1.75"

LL Reduction   ASCE 7-02   LL = 80 psf



L_o = 80 psf

42' · 8.75' = 367.5 SF

K_{LL} = 2

$K_{LL}A_T$ = 367.5 (2) = 735 SF

735 SF > 400 SF ∴ can be reduced

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$$

$$L = 80\left(0.25 + \frac{15}{\sqrt{735}}\right) = 64.3 \text{ psf}$$

64.3 psf (8.75') = 563 plf

D = 66 psf + 5 psf (beam) + 15 psf SDL = 86 psf (8.75') = 753 plf

D + L = 563 + 753 = 1.32 klf

$$M = \frac{wl^2}{8} = \frac{1.32(42)^2}{8} = 291.1 \text{ k-ft}$$

Composite Beam Check

b_{eff} = min | 42/8 = 5.25'
| 1/2(8.75') = 4.375' → controls

b_{eff} = 4.375'(2) = 8.75' = 105"

Ø = 0.75   f'c = 4000 psi   per S4.01

Fu not given - assume 65 ksi

Q_n = min | $0.5 A_{sc}\sqrt{f'_c E_c}$
| $A_{sc} F_u R_g R_p$

APPENDIX B – EXISTING GRAVITY SYSTEM CHECK

Composite Beam Check (continued)

$A_{sc} = \pi(3/8)^2 = 0.4418\ in^2$ $R_g = 1.0$

$E_c = 145^{(1.5)}\sqrt{4} = 3.492\ ksi$ $R_p = 0.6$

$Q_n = \min \begin{cases} 0.5(0.4418)\sqrt{4000(3.492)} = 26.1\ ksi \\ 0.4418(65)(0.6) = 17.2\ ksi \rightarrow \text{controls} \end{cases}$

Check W21x44 [21]

Assume $a = 1.0$ $t = 5.5''$ per S4.01

$Y_2 = t - a/2 = 5.5 - 1.0/2 = 5''$

Table 3-19 PNA at point 7 (weakest point)

$M = 344'^k$

$\Sigma Q_n = 163k$

$\frac{163}{17.2} = 9.5$ $9.5(2) = 19$ studs required < 21 ✓ok

Check a $\frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{163}{0.85(4)(105)} = 0.457 < 1.0$ ✓ok

Check LL Deflection

$I_{LB} = 1460\ in^4$ (Table 3-20) $Y_2 = 5''$

$W_{LL} = 563\ plf$ $\Sigma Q_n = 163k$

$\Delta_{LL} = \frac{Y_2(W_{LL})(L)^4(1728)}{384(29000)(1460)} = \frac{5(0.563)(42)^4(1728)}{384(29000)(1460)} = 0.931''$

$\Delta LL_{max} = \frac{L}{360} = \frac{42(12)}{360} = 1.4'' > 0.931''$ ✓ok

APPENDIX B – EXISTING GRAVITY SYSTEM CHECK

Check Total Load Deflection

$W_T = 1.2$ klf.

$$\Delta_T = \frac{5(1.32)(42)^4(1728)}{384(29000)(1460)} = 2.18'' - 1.75''_{\text{camber}} = 0.43''$$

$$\Delta_{T\,max} = \frac{L}{240} = \frac{42(12)}{240} = 2.1'' > 0.43'' \checkmark \text{OK}$$

Check Wet Concrete Deflection

Slab + Decking = 66 psf (Vulcraft)

W21×44 — $I_x = 843\text{ in}^4$ Table 3-2

$$W_{wc} = 42(8.75) + 44 = 411.5 \text{ plf}$$

$$\Delta_{wc} = \frac{5(.4115)(42)^4(1728)}{384(29000)(843)} = 1.18''$$

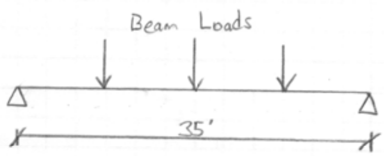
$$\Delta_{wc\,max} = \frac{L}{240} = \frac{12(42)}{240} = 2.1'' > 1.18'' \checkmark \text{OK}$$

Conclusion

W21×44 [21] with 1.75″ camber is adequate

APPENDIX B – EXISTING GRAVITY SYSTEM CHECK

Girder Gravity Check



W27 x 102 [30]   camber = 0.75"

$\phi = 0.75 \quad A_{sc} = 0.4418 \text{ in}^2$

$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} = 26.1 \text{ K}$

$n = 30 \quad n/2 = 30/2 = 15$ studs per side

$\Sigma Q_n = Q_n(n) = 26.1(15) = 391.5 \text{ K}$

ΣQ_n per AISC Table 3-19 $= 375 \text{K}$

$b_{eff} = 2 \times \min \begin{cases} \frac{(35)(12)}{8} = 52.5" \rightarrow \text{controls} \\ \frac{1}{2}(42)(12) = 252" \end{cases}$

$b_{eff} = 2(52.5) = 105"$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{375}{0.85(4)(105)} = 1.05$

$y_2 = t - a/2 = 5.5 - 1.05/2 = 4.975 \rightarrow$ try 5"

$M_c = 1040 \text{ k-ft}$

$L_o = 80 \text{ psf} \quad A_T = (42)(35) = 1470 \quad K_{LL} = 2$

$K_{LL} A_T = 1470(2) = 2940 > 400 \text{SF} \checkmark \text{ok}$

$LL = 80\left(0.25 + \frac{15}{\sqrt{2940}}\right) = 42.13 \text{ psf} > 0.5(L_o) = 40 \checkmark \text{ok}$

Assume Distributed Load

$W = 102 \text{plf} + (42.13 + 86)(42) = 5.483 \text{ klf}$

$M_R = \frac{W l^2}{8} = \frac{5.483(35)^2}{8} = 839.6 \text{ k-ft}$

APPENDIX B – EXISTING GRAVITY SYSTEM CHECK

Girder Gravity Check (continued)

Check Live Load Deflection

$W_{LL} = (42.13)(42) = 1.77 \text{ klf}$ $\qquad I_{LB} = 5680$

$$\Delta_{LL} = \frac{5(1.77)(35)^4(1728)}{384(29000)(5680)} = 0.363''$$

$$\Delta_{LL\,max} = \frac{L}{360} = \frac{35(12)}{360} = 1.17'' > 0.363'' \checkmark \text{OK}$$

Check Total Deflection

$W_T = 5.483 \text{ klf}$

$$\Delta_{TL} = \frac{5(5.483)(35)^4(1728)}{384(29000)(5680)} = 1.12'' - \underset{\text{camber}}{0.75''} = 0.37''_{\text{actual}}$$

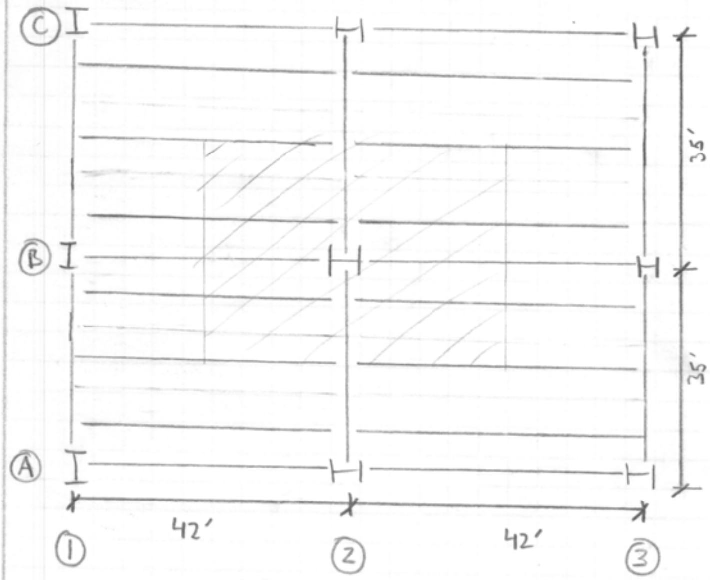
$$\Delta_{TL\,max} = \frac{L}{240} = \frac{35(12)}{240} = 1.75'' > 0.37'' \checkmark \text{OK}$$

Conclusion

W27×102 [30] with 3/4" camber is adequate

APPENDIX B – EXISTING GRAVITY SYSTEM CHECK

Interior Column Check   W14×159 – supports roof & levels 2-6



Roof

$A_T = (35')(42')$

$= 1470\text{ SF}$

Load = 59 psf (cannot reduce)

Total Load - Roof

59 psf (1470 SF) = 86.8k

* Floors 2-6 have identical loading and structure

LL Reduction

– $A_T = 1470\text{ sf} > 400\text{ SF}$ ✓ OK

$K_{LL} = 4$

$LL = 80\text{ psf}$

$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right) = 80\left(0.25 + \frac{15}{\sqrt{4(1470)}}\right) = 35.6\text{ psf}$

$0.4 L_o = 0.4(80) = 32 < 35.6\text{ psf}$ ✓ OK

↳ columns support multiple floors

Dead Load = 86 psf (1470) = 126.4K

Live Load = 35.6 psf (1470) = 52.3K

Total load = 86.8K (Roof) + (126.4 + 52.3)(5) (# levels) = 980.3K

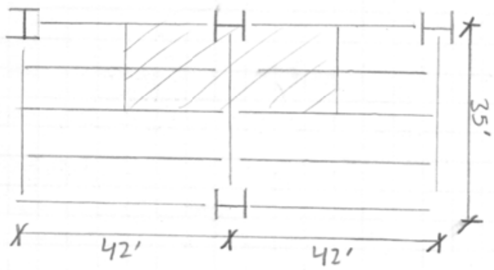
Unbraced length = 14'

Table 4-1 $P_n/\Omega = 1230\text{K} > 980.3\text{K}$ ✓ OK

APPENDIX B – EXISTING GRAVITY SYSTEM CHECK

Exterior Column Check

W14×132 supports roof + Levels 2-6



$A_T = (42)(35/2) = 735$ SF

$K_{LL} = 4$

$K_{LL}A_T = 735(4) = 2940 > 400$ ✓

Roof Load = 59 psf

L_o Levels 2-6 = 80 psf

$L = 80\left(0.25 + \frac{15}{\sqrt{2940}}\right) = 44$ psf $> 0.4(L_o) = 32$ psf ✓ ok

DL Levels 2-6 = 86 psf

Curtain wall = 56 psf → wall panel per floor

= (14' height)(42' wide) = 588 SF

Curtain wall load (per Level) = 56(588 SF) = 32.93 K

Total DL = 86 psf (735 SF)(5 levels) = 316.1 K

Total LL = 44 psf (735 SF)(5 levels) = 161.7 K

Total Roof Load = 59 psf (735 SF) = 43.4 K

Total Curtain Wall Load = 32.93 K (5 levels) = 164.7 K

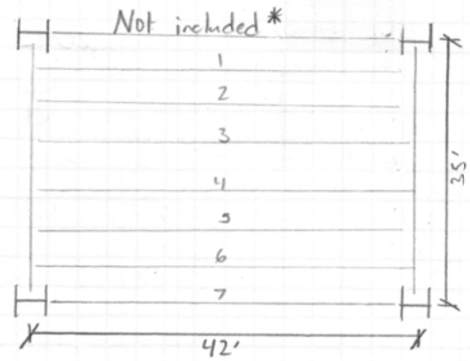
Total load on column = 316.1 + 161.7 + 43.7 + 164.7 = 686.2 K

Unbraced Length = 14'

Table 4-1 for W14×132 $P_n/\Omega = 1000$ K > 686.2 K ✓ ok

APPENDIX C – ALTERNATIVE SYSTEM 1

System 1 – Non-Composite Steel Joists



Original Beam Depth = 21"

Distance from floor slab to acoustic ceiling below = 2'-3" or 27"

* exterior joist remains a beam. A joist is not suitable for carrying curtain wall loads

Member should remain a W-Shape as noted in drawing

DL = 86 psf for Levels 2-6

LL = 80 psf

Factor Loads

Span = 42'

$W_{ult} = [1.2(86) + 1.6(80)](5') = 1156\ plf + 1.2(\text{joist weight})$

$W_{TL} = (86 + 80)(5) = 830\ plf + 1.2(\text{joist weight})$

$\Delta_{TL\,max} = \frac{L}{240} = \frac{42(12)}{240} = 2.1"$

Try 28LH12 : 27 plf self weight
$W_{ult} = 1255\ plf$
Span = 42 ft

$1255 > 1156 + 1.2(27) = 1188.4$ ✓ ok

w for L/360 = 520 plf

w for L/240 = 520 (1.5) = 780 plf < 830 plf ∴ No good

Check 28LH13

w for L/360 = 543

w for L/240 = 543(1.5) = 814.5 < 830 plf ∴ Strogest possible 28" LH joist is no good

APPENDIX C – ALTERNATIVE SYSTEM 1

System 1 continued

Must Use SAFE LOAD for 32" depth LH - Joists

$$\text{SAFE LOAD} = \frac{\text{safe load}}{42' + 8''} = \frac{\text{safe load}}{42.67'}$$

Try 32LH13 → $\frac{60900 \text{ lb}}{42.67} = 1427.2$ plf capacity

1427.2 plf > 1156 plf + 1.2(27) = 1188.4 plf load ✓ OK

For spans between 49' and 55', the L/360 capacity reduces an average of 20 plf per one foot of added length.

So between 42' and 49', the joist should gain an average of 20plf (7') = 140 plf

To be conservative, use an additional 100 plf capacity for L/360 added to a 49' span capacity of 500 plf

W for L/360 = (500 + 100) = 600 plf

W for L/240 = 600 (1.5) = 900 plf > 830 + 1.2(30) = 866 plf

Conclusion

For 42 × 35 foot bay, seven 32LH13 joist girders (labled on initial diagram) are adequate for load and deflection spaced 5' apart

With spans reaching 42', bridging is necessary to keep joists from rolling during construction

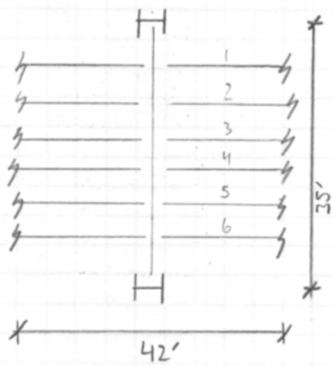
Joist Girder 7 requires no redesign as decking and loading is uniform into the adjacent bay

APPENDIX C – ALTERNATIVE SYSTEM 1

System 1 Continued

32LH13

Design Girder



Joist Self Weight = 30 plf

LL = 80 psf

DL = 86 psf

$A_T = 42(35) = 1470$ SF

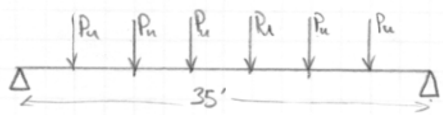
$K_{LL} = 2$

$K_{LL} A_T = 2(1470) = 2940 > 400$ ✓

$L = 80\left(0.25 + \frac{15}{\sqrt{2940}}\right) = 42.1$ psf

Joist self weight = 30plf (42') = 1.26k

$P_u = 1.26 + (42.1 + 86)(5' \text{ width})(42 \text{ length}) = 26.9$ K

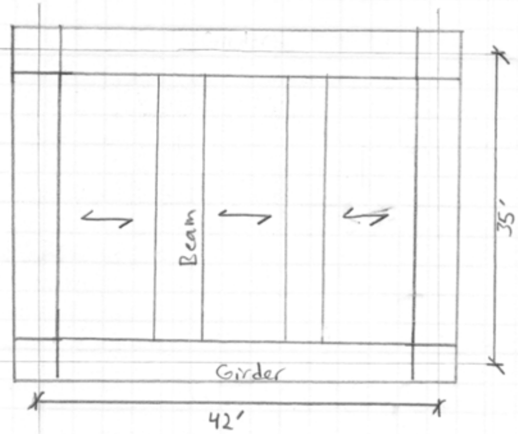


$M_A = 823.5$

W30 × 108 → $M_P/\Omega = 863$ k-ft

APPENDIX D – ALTERNATIVE SYSTEM 2

System 2 – One Way Slab on Beams



- Beam are spaced equally
- Assume 24" x 24" columns
- Assume 12" wide beams
- Beams spaced 14' apart OC
- Clear span between beams = 14 - 2(0.5) = 13'
- Assume #6 bar in slab

$h_{min} = \frac{l}{24}$ → One end continuous Solid One-way slabs (Table 9.5 A)

$h_{min} = \frac{14(12)}{24} = 7"$

DL = 15 psf SDL + 7"(150 pcf) = 15 + 87.5 = 102.5 psf

LL = 80 psf

Design Slab

h = 7"

$d = 7" - 3/4"$ clear cover $- \frac{(0.75)}{2}$ (rebar) $= 5.875$ → use 6" (constructibility)

Theoretical 1 foot wide beam

$W_u = 1.2(102.5) + 1.6(80) = 251$ psf

$M_u = \frac{(251)(13')^2}{8} = 5.3$ kft

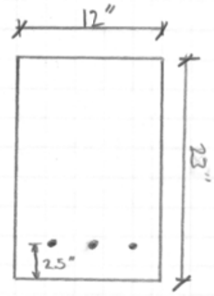
$A_s \approx \frac{M}{4d} = \frac{5.3}{4(6)} = 0.221$ in²/ft   #6 bar @ 6" oc

$A_T \approx 0.0018bh = 0.0018(12)(7) = 0.1512$ #4 bar @ 16" oc

APPENDIX D – ALTERNATIVE SYSTEM 2

System 2 Continued

Beam Design – Assume 12" x 23"



$W_u = 251\,psf\,(14') + 1.2(1)(23/12)(150)$

$W_u = 3.859$ klf $\qquad \ell = 14' - 2(0.5') = 13'$

$M_u = \frac{3.784(13')^2}{8} = 80$ kft

$A_s \approx \frac{M_u}{4d} = \frac{80}{4(23'' - 2.5'')} = 0.975''^2$

↳ cover

(3) #6 = 1.32 in^2 > 0.975 in^2 ✓ ok

$h_{min} = \frac{\ell}{18.5}$ One end continuous

$= \frac{35(12)}{18.5} = 23''$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(1.32)(60)}{0.85(4)(12)} = 1.94$

$f'_c = 4000\,psi$ per S4.01

$c = \frac{a}{\beta_1} = \frac{1.94}{0.85} = 2.28$ in

$\varepsilon_s = \varepsilon_u\left(\frac{d-c}{c}\right) = 0.003\left(\frac{15.5 - 2.28}{2.28}\right) = 0.0174 > 0.00207$ ✓

$\phi = 0.9$

$M_n = A_s f_y (d - a/2) = 1.32(60)(20.5 - 1.94/2)$

$M_n = 128.9$ k ft

$80\,kft = M_u \le \phi M_n = 0.9(128.9) = 116$ k ft ✓ ok

Check $A_{s,min} = \frac{200}{f_y}(bh) = \frac{200}{60000}(12)(23) = 0.92\,in^2 < 1.32$ ✓ ok

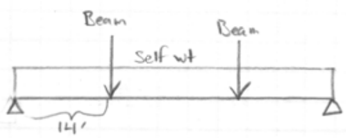
Check $\rho = \frac{A_s}{bd} = \frac{1.32}{12(20.5)} = .00537$

$\rho_{max} = 0.85\beta_1\left(\frac{f'_c}{f_y}\right)\left(\frac{\varepsilon_u}{\varepsilon_u + 0.005}\right) = 0.85(0.85)\left(\frac{4}{60}\right)\left(\frac{0.003}{0.008}\right) = 0.0181$

$\rho < \rho_{max}$ ✓ ok

APPENDIX D – ALTERNATIVE SYSTEM 2

System 2 Continued



Girder Design

Beam Load on Girder (per point load) $= 3.8591\ \text{klf}\left(\frac{17.5}{2}\right) \times 2 = 67.53\ \text{k}$

Girder Self Weight = Assume $b = 24$ = column width

$M_u = 1.1\left(\tfrac{1}{2} \times 67.53 \times 14'\right)\left(\tfrac{1}{2}\right) \times 2 = 520'\text{K}$

$h_{min} = \frac{l}{21}$ both ends continuous

$\frac{42(12)}{21} = 24''$

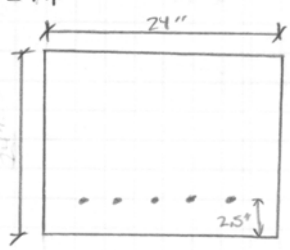
$20 M_u = bd^2 \qquad 20(520) = 24d^2$

$d = 20.8 \rightarrow$ use $21.5''$ to satisfy h_{min}

$h = 21.5'' + 2.5'' = 24$

$M_u = \left(\frac{67.53}{2}\right)(7) + \frac{(24'' \times 24'')(150\text{pcf})(13)^2}{8(1000)} = 249\ \text{kft} < 520\ \text{kft}$ ✓ok

$A_s = \frac{M_u}{4d} = \frac{249}{4(20.5)} = 3.04\ \text{in}^2$



(5) #8 $= 3.95\ \text{in}^2 > 3.89\ \text{in}^2$ ✓ok

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{3.95(60)}{0.85(4)(24)} = 2.90\ \text{in}$

$c = a/\beta_1 = 2.90/0.85 = 3.41\ \text{in}$

$\varepsilon_s = 0.003\left(\frac{15.5 - 3.41}{3.41}\right) = 0.0106 > 0.00207$ ✓ok $\quad \phi = 0.9$

$M_n = A_s f_y (d - a/2) = 3.95(60)(20.5 - 2.9/2)$

$M_n = 376.2\ \text{kft}$

$241.3 = M_u \leq \phi M_n = 0.9(376.2) = 338.6$ ✓ok

Check $A_{s\,min} = \frac{200}{f_y} bh = \frac{200}{60000}(24)(24) = 1.92\ \text{in}^2 < 3.95$ ✓ok

Check $\rho = \frac{A_s}{bd} = \frac{3.95}{(24)(20.5)} = 0.00803$

$\rho_{max} = 0.85(0.85)\left(\frac{4}{60}\right)\left(\frac{0.003}{0.008}\right) = 0.0181$

$\rho < \rho_{max}$ ✓ok

System 2 Continued

Beam Deflection Check

$W_u = 3.859 \text{ klf}$

$E = 57000\sqrt{4000} = 3605 \text{ ksi}$

$I = \frac{bh^3}{12} = \frac{12(23)^3}{12} = 12167 \text{ in}^4$

35/2

$$\Delta_{TL} = \frac{5 w_u l^4}{384EI} = \frac{5(3.859)(17.5)^4(1728)}{384(3605)(12167)} = 0.186''$$

$$\Delta_{TL \text{ Allowable}} = \frac{l}{240} = \frac{17.5(12)}{240} = 0.875 > 0.186 \ \checkmark \text{ok}$$

Girder Deflection Check

$$W_u = \frac{249(8)}{17.5^2} = 6.5 \text{ klf}$$

$$I = \frac{24(24)^3}{12} = 27648 \text{ in}^4$$

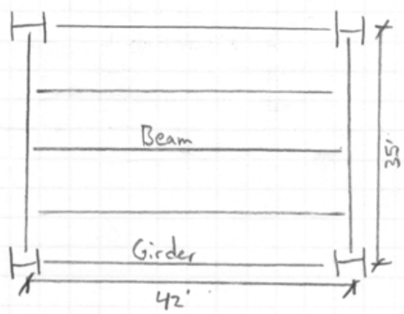
$$\Delta_{TL} = \frac{5(6.5)(21)^4(1728)}{384(3605)(27648)} = 0.285''$$

$$\Delta_{TL \text{ Allowable}} = \frac{l}{240} = \frac{21(12)}{240} = 1.05'' > 0.285'' \ \checkmark \text{ok}$$

APPENDIX E – ALTERNATIVE SYSTEM 3

System 3 - Non-Composite Slab

Use identical Beam & Girder layout



Use 23.5" topping, identical to existing floor slab, for fire proofing

$\frac{35}{4} = 8.75'$ span → Use 9' in tables

Use 3-span as prescribed by specs

Try 2C conform deck – Vulcraft

Total load excluding self weight

LL = 80 psf SDL = 15 psf Total = 95 psf

Must use at least 7" slab for 9' clear span

Reinforcing 4x4 W2.9x2.9 = 142 psf allowable > 95 psf ✓ok

7" slab = 2" deck + 5" topping ✓ok fireproofing

Try 2C18 • 10'–6" max clear span ✓ok
• 75 psf

L/240 allowable uniform load = 95 psf > 80 psf ✓ok

L/180 allowable uniform load = 126 > 80 + 75 = 155 ✗ NO GOOD

Try 3C Conform deck - Vulcraft

Reinforcing 4x4 W2.9 x 2.9 = 142 psf allowable > 95 psf ✓ok

7" slab = 3" deck + 4" topping ✓ok fire proofing

3C 20 : 11'–4" 3 span construction ✓ok
• 69 psf

L/240 allowable uniform load = 156 psf > 95 psf ✓ok

L/180 allowable uniform load = 165 psf > 80+69 = 149 psf ✓ok

USE 3C20 deck

APPENDIX E – ALTERNATIVE SYSTEM 3

System 3 continued

Beam Design

DL = 69 psf + 15 SDL + 5 psf self weight = 89 psf

LL = 80 psf → Reduction = 64.3 psf from gravity check

$W_u = [1.2(89) + 1.6(64.3)]\, 8.75'\text{ span} = 1.83 \text{ klf}$

$V_u = \frac{1.83(42')}{2} = 38.43 \text{ k}$

$M_u = \frac{1.83(42)^2}{8} = 403.5 \text{ kft}$

Try W21 × 50

$\phi M_p = 413 \text{ k-ft} > M_u$ ✓ OK

$\phi V_n = 237 \text{ k} > V_u$ ✓ OK

$I_x = 984 \text{ in}^4$

$W_{TL} = [(69 + 15) + 50]\, 8.75 = 1.173 \text{ klf}$ $\quad W_{LL} = 0.7 \text{ klf}$

$\Delta_{LL} = \frac{5(0.7)(42)^4(1728)}{384(29000)(984)} = 1.72$

$\Delta_{Allow} = \frac{L}{360} = \frac{42(12)}{360} = 1.4 < 1.72''$ ✗ NO GOOD

Try W21 × 62 $\quad I_x = 1330$

$W_{TL} = [(69 + 15) + 62]\, 8.75' = 1.28 \text{ klf}$ $\quad W_{LL} = 0.7$

$\Delta_{LL} = \frac{5(0.7)(42)^4(1728)}{384(29000)(1330)} = 1.27''$

$\Delta_{Allow} = \frac{L}{360} = 1.4'' > 1.27$ ✓ OK

$\Delta_{TL} = \frac{5(1.28)(42)^4(1728)}{384(29000)(1330)} = 2.32''$

$\Delta_{Allow} = L/240 = 2.1'' < 2.32''$ ✗ NO GOOD

APPENDIX E – ALTERNATIVE SYSTEM 3

System 3 Continued

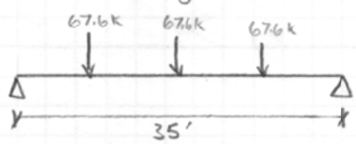
$$I_x \text{ Required} = \frac{5(1.28)(42)^4(1728)}{384(29000)(2.1'')} = 1472 \text{ in}^4$$

Try W21 x 68 $I_x = 1480 \text{ in}^4$

$$\Delta_{TL} = \frac{5(1.28)(42)^4(1728)}{384(29000)(1480)} = 2.087'' < 2.1'' \ \checkmark \text{OK}$$

Use W21 x 68 for beams

Girder Design



LL = 42.13 psf from gravity check

$$W_u = 1.2(89(8.75) + 68) + 1.6(42.13(8.75)) = 1.61 \text{ klf}$$

$$P_u = 1.61(42') = 67.6 \text{ k}$$

$$V_u = 67.6 + \tfrac{1}{2}(67.6) = 101.4 \text{k}$$

$$M_u = 67.6(8.75) + \tfrac{1}{2}(67.6)(8.75) = 887.3 \text{ kft}$$

Try W27 x 84 $I_x = 2850$

$$\phi M_p = 915 \text{ kft} > 887.3 \ \checkmark \text{OK}$$
$$\phi V_n = 246 > 101.4 \text{K} \ \checkmark \text{OK}$$

$$P_{LL} = \frac{42.13(8.75)(42)}{1000} = 15.48 \text{k}$$

$$\Delta_{LL} = \frac{15.48(35)^3(1728)}{28(29000)(2850)} = 0.50''$$

$$\Delta_{Allow} = \frac{L}{360} = \frac{35(12)}{360} = 1.17'' > 0.5'' \ \checkmark \text{OK}$$

APPENDIX E – ALTERNATE SYSTEM 3

System 3 Continued

$DL = 69 psf$ (Slab) $+ 15$ SDL $= 84$ psf

$$P_{TL} = \left(\frac{(84)(8.75) + 84}{1000}\right)(42) + 15.48\,K = 50\,K$$

(girder self wt)

$$\Delta_{TL} = \frac{50(35)^3 1728}{28(29000)2850} + \frac{5(84/1000)(35^4)(1728)}{384(29000)(2850)}$$

$$\Delta_{TL} = 1.6 + 0.034 = 1.634''$$

$$\Delta_{Allow} = L/240 = 1.75'' > 1.634'' \quad \checkmark \text{OK}$$

Conclusion

Use 3C 20 Conform decking with 4" topping with 4×4 W2.9×2.9 WWF reinforcing

Use 21 × 68 for beams

Use W27×84 for girders