LETTER OF TRANSMITTAL

DATE: October 25, 2013

TO: Dr. Linda Hanagan

FROM: Alyssa Stangl

ENCLOSED: AE 481W – Senior Thesis | Structural Technical Report 3 - Addition

Dear Dr. Hanagan,

This report was prepared as an additional design for Technical Report 3 for AE 481W – Senior Thesis. The building in question is La Jolla Commons Phase II Office Tower. Items included in this report are as follows:

- Alternative Floor System #4 – Pan Joist System
- Weight and Cost Analysis – Pan Joist System Only
- Updated overall system comparison chart

Thank you for your time reviewing this report. I look forward to discussing it with you in the near future.

Sincerely,

Alyssa Michelle Stangl

Technical Report 3 - Addition

October 25, 2013

La Jolla Commons Phase II Office Tower

San Diego, California

Alyssa Stangl | Structural Option | Advisor: Dr. Linda Hanagan

Table of Contents

Executive Summary

La Jolla Commons Phase II Office Tower is a 13 story office building in San Diego, California. Each floor is about 40,320 square feet, and the structure reaches 198 feet from ground level to the top of the penthouse. With two levels of underground parking, the building extends about 20 feet below grade. Serving as an office building for LPL Financial, the building has open floor plans and large areas of glass curtain wall. La Jolla Commons Tower II received a LEED-CS Gold Certification and is the nation's largest and most advanced net-zero office building.

The building's gravity system begins with a mat foundation, two stories below grade. The mat foundation was chosen for its constructability, when compared to a system of footings and grade beams. The super structure consists of two-way, flat plate, concrete slabs on a rectangular column grid. A typical bay is 30 feet by 40 feet. Each level varies in thickness, ranging from 12 to 18 inches with reinforcing, as required, by code. Camber was used for the slab at each level, excluding Lower Level 2 where the mat foundation serves as the floor. The designers determined that the large construction loads would cause the slab to crack; therefore, slab deflections were calculated for a cracked slab section. As a result, the deflections calculated for post-construction loading were significant. The maximum camber applied to the slab is 2 ¼" at the center of a bay.

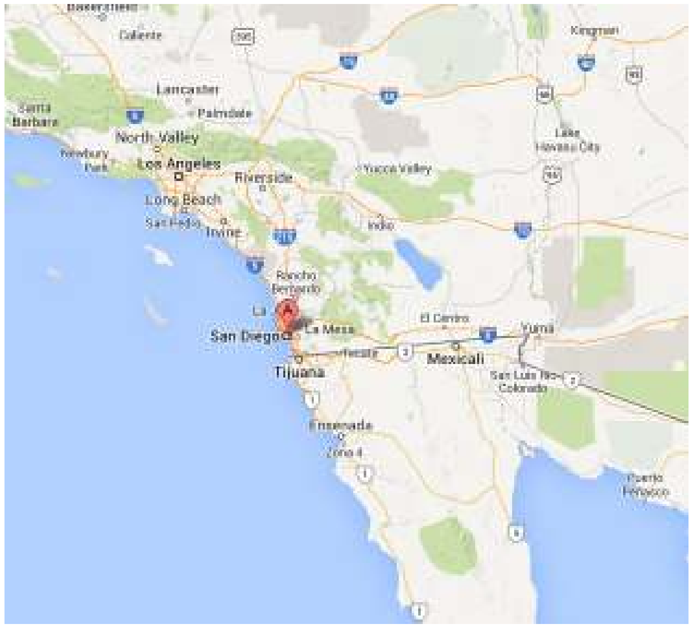
Laid out at the core of the building, the lateral system of La Jolla Commons Tower II consists of reinforced concrete shear walls. Due to the high shear forces associated with earthquake loading in this Seismic Category D structure, the diaphragm alone is not relied upon to transfer lateral loads to the shear wall system; instead, collector beams are used to aid in the transfer of lateral loads at levels below grade in the north-south direction.

La Jolla Commons Tower II has two unique structural and architectural features. The north and south sides of the building feature 15 foot cantilevers that start at Level 3 and continue up to the roof level. The structure of each cantilever is similar to that of the rest of slab; though, it does have additional reinforcement. Also, the building has a plaza area on the Ground Level which carves out a portion of Ground Level 1 and Level 2. Main building columns are exposed here, and additional 18 inch columns are added to support the slab edge above.

La Jolla Commons Tower II was designed using the 2010 California Building Code which corresponds to ASCE 7-05 and ACI 318-08. CBC 2010 and ASCE 7-05 were used to calculate live, wind, and earthquake loads. ACI 318 – 08, Chapter 21, references the design of concrete Earthquake-Resistant structures, and ASCE 7-05, Chapter 12, details the Seismic Design Requirements for Building Structures. Both of these documents were used heavily in the design of LJC II in order to account for seismic loading and detailing.

La Jolla Commons Phase II Office Tower is full of educational value. It has several structural challenges and unique conditions: punching shear, seismic loading and detailing, concrete shear wall design, and computer modeling. The following report explains the building structure, design codes, and design loads in more detail.

Building Site Information



San Diego California *(Google Maps)*

Building Site Plan *(Courtesy of Hines)*

La Jolla Commons Phase II Office Tower

San Diego , California | LPL Financial Office Tower

Primary Project Team

Owner | Hines
Tenant | LPL Financial
Architect | AECOM
Structural Engineer | Nabih Youssef Associates
MEP Engineer | WSP Flack + Kurtz
Civil Engineer | Leppert Engineering

General Building Data

Construction Dates | April 2012 – May 2014
Building Cost | $78,000,000
Delivery Method | Design-Bid-Build
Height| 198' – 8" | 13 Stories
2 Levels | Underground Parking
Size | 462,301 GSF

Architecture

- Modern style building with glass curtain wall
- 12 foot floor-to-floor height
- Very open and spacious office area
- Interior features and build out by tenant

Mechanical

- Chilled Water, floor-by – floor VAV Dual Path Air Handling Units
- Ventilation and cooling through underfloor air distribution, overhead air to perimeter zones.

Sustainability Features

- First Class A, NetZero Office Building in the USA
- Building returns more energy to the grid than it uses on an annual basis
- LEED – CS Gold Certification

Lighting and Electrical

- High efficiency, low glare lighting fixtures
- High power factor electronic ballasts
- Lighting control system integrated with Building Management System, local override at each floor
- Two 400 Amp, 480/277V, 3-phase, 4 wire switchboards service building
- One services the lower level bus riser and the other services the upper level bus riser
- One diesel fuel standby engine generator.

Structural

- Two-way, flat plate , reinforced concrete slab
- Concrete columns on a regular column grid
- Special reinforced concrete shear walls
- Mat foundation system

NABIH YOUSSEF ASSOCIATES STRUCTURAL ENGINEERS
AECOM Hines
LPL Financial

Alyssa Stangl [Structural Option] http://www.engr.psu.edu/ae/thesis/portfolios/2014/ams6158

Documents Used to Create This Report

- *American Concrete Institute*
    - ACI 318 – 11
- *American Institute of Steel Construction*
    - AISC – Steel Construction Manual, 14th Edition
- *Concrete Reinforcing Steel Institute*
    - CRSI Handbook 2008
- *Reinforced Concrete Mechanics and Design, 6th Edition*
    - By James K. Wight and James G. MacGregor
- *International Building Code*
    - IBC 2012
- *Reed Construction Data | RS Means*
    - Square Foot Cost 2013
- *American Society of Civil Engineers*
    - ASCE 7 – Minimum Design Loads for Buildings
- *La Jolla Commons Phase II Office Tower*
    - Construction Documents
    - Technical Specifications

ALTERNATIVE FLOOR SYSTEM

DESIGN #4 – PAN JOIST SYSTEM

One Way Slab Pan Joist System - Design 4

Slab thickness = 4.5"

Max beam spacing:

Table A-9 from Wright & MacGregor

$h_{min} = l/16$

$4.5 = l/16 \rightarrow l = 72" = 6 ft$

Try Pan-Joist system @ 30" spacing:

$DL = (150)(4.5/12) + 23 = 79.25$ PSF

$LL = 80$ PSF $K_{LL}A_T = (5ft)(42ft) = 210 ft^2 < 400 ft^2$

*No LL Reduction

$W_u = 1.2(79.25) + 1.6(80) = 223$ PSF

$w_u = (223 \text{ PSF})(2.5 ft) = 557.5$ lb/ft

$M_u = \frac{w_u l^2}{8} = \frac{(557.5 \text{ lb/ft})(42')^2}{8} \cdot \frac{1}{1000}$

$M_u = 123^{\text{'k}}$

Estimate Joist Size: (limit depth to 18")

$h = d + 2.5 \rightarrow 18 = d + 2.5 \rightarrow d = 15.5 in$

$b \cdot d^2 = 20 M_u$

$b(15.5)^2 = 20(123)$

$b = 10.24 \rightarrow 11"$ but 12" is more common

Try $b = 12"$, $h = 18"$

Note: Now that I know this system will yield reasonable beam sizes, I am going to fully design the slab.

Design Slab:

$h = 4.5"$

LL = 80 PSF
DL = 79.25 PSF
} $W_u = 223$ PSF $= 0.223$ ksf

Design Moments: (1 ft strip)

$$M_u^- = \frac{W_u l_n^2}{11} = \frac{(0.223)(2.5)^2}{11} = 0.127 \text{ ft-k}$$

$$M_u^+ = \frac{W_u l_n^2}{16} = \frac{(0.223)(2.5)^2}{16} = 0.0871 \text{ ft-k}$$

$$d = 4.5" - 3/4" - \tfrac{1}{2}(0.375) = 3.5625"$$

$$A_s^+ = \frac{M_u^+}{4d} = \frac{0.0871^{1k}}{4(3.5625)} = 0.00611 \text{ in}^2/\text{ft}$$

$$A_s^- = \frac{M_u^-}{4d} = \frac{0.127^{1k}}{4(3.5625)} = 0.0089 \text{ in}^2/\text{ft}$$

As,min: $\rho_{min} = 0.0018$ (ACI 318-11 §7.12.2.1)

$$\rho = \frac{A_s}{b \cdot h} \rightarrow 0.0018(12")(4.5") = A_{s,min}$$

$$A_{s,min} = 0.0972 \text{ in}^2/\text{ft}$$

$A_s^+ = 0.0972$ in²/ft
$A_s^- = 0.0972$ in²/ft
} #3 @ 12" O.C.

Bar Spacing: (§10.6.4)

$$S_{max} = \min \begin{cases} 15\left(\frac{40{,}000}{60{,}000}\right) - 2.5(3/4") = 8.125" \\ 12\left(4/6\right) = 8" \end{cases}$$

$S_{max} = 8"$ ← Max for crack control

Use #3 bars @ 8" O.C.

$$A_{s\,provided} = \frac{0.11 \text{ in}^2}{8"} \times 12 = 0.165 \text{ in}^2/\text{ft}$$

$A_{s,provided} > A_{s,min}$ ✓

Verify Slab Moment Capacity: $d = 3.5625''$

$$a = \frac{A_s \cdot f_y}{0.85 f'_c \cdot b} = \frac{(0.165)(60{,}000)}{0.85(4000)(12)} = 0.2426$$

$$c = a/\beta_1 = \frac{0.2426}{0.85} = 0.285$$

$$\varepsilon_s = \frac{\varepsilon_u}{c} \cdot (d-c) = \frac{0.003}{0.285}(3.5625 - 0.285)$$

$$\varepsilon_s = 0.0345 > 0.00207 \checkmark$$

$$\phi M_n = \phi A_s \cdot f_y (d - a/2)$$

$$= (0.9)(0.165)(60{,}000)(3.5625 - 0.2426/2)/12 \cdot 1000$$

$$\phi M_n = 2.56^{'k} > M_u^- = 0.127^{'k} \checkmark$$

$$> M_u^+ = 0.0871^{'k} \checkmark$$

Check Shear Capacity: BM size: $b = 12''$

V_u @ d from support:

$$V_u = (15'' - 6'' - 3.5625'')/12 \,(42')(0.223)$$
$$= 4.24^k$$

$$\phi V_n = 1.0\,\phi V_c = (0.75)\,2\,(1.0)\sqrt{4000}\,(42 \times 12)(3.56)/1000$$

$$\phi V_n = 170^k \gg V_u \checkmark$$

Typical Slab			
Slab Thickness	N-S		E-W
	Top	Bottom	
4.5"	#3@8"	#3@8"	#3@8"

Notes: 1. 4000 PSI, NW Concrete
2. $F_y = 60{,}000$ PSI

Finish Joist Design:

b=12" h=18" , d≈15.5"

* subtract 2' from span of joists b/c of girders

$$M_u = \frac{W_u \ell^2}{8} = \frac{(557.5 \text{ lb/ft})(40)^2}{8} \cdot \frac{1}{1000} = 111.5^{\text{ 'k}}$$

$$W_{sw} = \frac{(12 \times (18-4.5))}{144} \times 150 \cdot \frac{1}{1000} = 0.169 \text{ k/ft}$$

$W_u = 1.2(0.169) = 0.2028$ k/ft

$$M = \frac{(0.2028)(40)^2}{8} = 40.56^{\text{ 'k}}$$

$W_{u,total} = 0.7603$

$M_{u,total} = 152.06^{\text{ 'k}}$

Required Steel:

$$A_s = \frac{M_u}{4d} = \frac{(152.06)}{4(15.5)} = 2.45 \text{ in}^2$$

Try (3) #9 bars

$A_{s,provided} = 3.0 \text{ in}^2$

$d = 18" - 1.5 - 0.5 - \tfrac{1}{2}(1.128) = 15.436"$

Check ØMn:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(3.0)(60,000)}{(0.85)(4000)(12)} = 4.41"$$

$$c = a/\beta_1 = 4.41"/0.85 = 5.19"$$

$$\varepsilon_s = \frac{0.003}{5.19}(15.436 - 5.19) = 0.0059 \geq 0.00207 \checkmark$$

$\geq 0.005 \checkmark$

$\phi = 0.9$ is ok!

$$\phi M_n = \phi A_s f_y (d - a/2)$$
$$= (0.9)(3.0)(60)(15.436 - 4.41/2) \cdot \frac{1}{12}$$
$$= 178.6^{\text{ 'k}}$$

$\phi M_n = 178.6^{\text{ 'k}} > M_u = 152.06^{\text{ 'k}} \checkmark$

Check As,min :

$$A_{s,min} = \max \begin{cases} \dfrac{3\sqrt{4000}}{60000}(12")(15.436") = 0.586 \text{ in}^2 \\ \dfrac{200(12)(15.436)}{60{,}000} = 0.617 \text{ in}^2 \end{cases}$$

$$A_{s,min} = 0.617 \text{ in}^2 < 3.0 \text{ in}^2 \checkmark$$

Check As, max :

$$A_{s,max} = 0.85\,\beta_1 \frac{f'c}{fy}\left(\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_y}\right) b_w \cdot d$$

$$= (0.85)^2 \frac{4000}{60{,}000}\left(\frac{0.003}{0.003+0.004}\right)(12)(15.436)$$

$$A_{s,max} = 3.82 \text{ in}^2 > 3.0 \text{ in}^2 \checkmark$$

Check Min + Max # Bars:

Using table A.7 from ACI 3.8 reference:

3/4" aggregate
$b_w = 12"$
#9 bars
} 3 max per layer ✓

Using table A.8 adapted from ACI §10.6.4:

2" cover w/ stirrup
$b_w = 12"$
9 bars
} 2 minimum / layer ✓

Check Shear Capacity:

$V_c = 1.1\ 2\lambda\sqrt{f'c}\ bw \cdot d$ (§ 8.13.8)
$= 1.1\,(2)(1.0)\sqrt{4000}\,(12)(15.436)/1000$
$= 25.77^k$

$\phi V_n = 0.75\,(25.77^k) = 19.3^k$ § 11.4.6.1 (c)

$V_u = \left(\frac{40'}{2} - 15.436/_{12}\right)(0.7603) = 14.23^k$

$\phi V_n = 19.3^k > V_u = 14.23^k$ ✓

No shear reinforcement required

Pan Joist Summary: B1

$\boxed{\begin{array}{c} 12'' \times 18''\ \text{NW CONC. } 4000\ \text{PSI} \\ (3)\ \#9\ \text{bottom bars} \end{array}}$

DESIGN GIRDER: G1 , $\ell n = 30' - 12''/12'' = 29'$

* Many point loads, can be approximated as a distributed load.

$$LL = 80 \times \max \left| \begin{array}{l} \left(0.25 + \frac{15}{\sqrt{21' \times 30'}}\right) = 0.848 \\ 0.5 \end{array} \right.$$

$LL = 67.81$ PSF

$1.6(67.81 \text{ PSF}) = 108.5$ PSF

$W_L = (108.5 \text{ PSF})(21 \text{ ft})/1000 = 2.28$ k/ft

Dead load:

DL = 79.34 PSF (including bm selfweights)

$1.2(79.34) = 95.2$ PSF

$W_D = (95.2)(21)/1000 = 2.0^k/ft + 1.2(.118)$
$= 2.14^k/ft$

$W_{Total} = 4.42$ k/ft

$M_u^+ = W_u \ell n^2/16 = (4.42)(29)^2/16 = 232'^k$

$M_u^- = W_u \ell n^2/11 = (4.42)(29)^2/11 = 338'^k$

Estimate Girder Size: (limit to 18" depth)

$h = d + 2.5 \rightarrow d = 15.5''$

$b \cdot d^2 = 20 M_u$
$b \cdot (15.5)^2 = 20(338)$
$b = 28.14'' \rightarrow 29''$ but 30" more common

Try $b = 30'' \times 18''$

Add SW: $w_{sw} = \frac{(30 \times 18)}{144}(150)/1000 = 0.563 \,^{k}/_{ft}$

$w_u = 1.2(0.563 \,^{k}/_{ft}) = 0.675 \,^{k}/_{ft}$

$$w_{u,new} = 0.675 \,^{k}/_{ft} + 4.42 \,^{k}/_{ft}$$
$$= 5.1 \,^{k}/_{ft}$$

$Mu^+ = (5.1)(29)^2/16 = 268^{\prime k}$
$Mu^- = (5.1)(29^2/11 = 390^{\prime k}$

Required Steel:

@ Mu^+ (bottom bars @ midspan):

$$As^+ = \frac{Mu^+}{4d} = \frac{268}{4(15.5)} = 4.3 \text{ in}^2 \rightarrow (6) \# 8$$

As^+, provided = 4.74 in^2

$d^+ = 18'' - 1.5'' - 0.5'' - \frac{1}{2}(1'') = 15.5''$

@ Mu^- (top bars over supports):

$$As^- = \frac{Mu^-}{4d} = \frac{390^{\prime k}}{4(15.5)} = 6.29 \text{ in}^2 \rightarrow (8) \# 8$$

As^-, provided = 6.32 in^2

$d^- = 18 - 1.5 - 0.5 - \frac{1}{2}(1) = 15.5''$

Check ϕMn^+:

$$a = \frac{(4.74)(60{,}000)}{0.85(4000)(30'')} = 2.79''$$

$c = a/\beta_1 = 3.28''$

$$\varepsilon_s = \frac{0.003}{3.28}(15.5 - 3.28) = 0.0112 > 0.00207 \checkmark$$

$$\phi Mn^+ = (0.9)(4.74)(60{,}000)(15.5 - 2.79/2)/1000 \cdot 12$$
$$= 300^{\prime k}$$

$\phi Mn^+ = 300^{\prime k} \geq Mu^+ = 268^{\prime k} \checkmark$

Use (6) #8 bars (As^+)

Check ϕM_n^- :

$$a = \frac{(6.32)(60000)}{0.85(4000)(30)} = 3.71''$$

$$c = 4.36''$$

$$\varepsilon_s = \frac{0.003}{4.36}(15.5 - 4.36) = 0.007 \geq 0.00207 \checkmark$$

$$\phi M_n^- = 0.9(6.32)(60{,}000)(15.5 - 3.72/2)/1000 \cdot 12$$
$$= 388^{1k}$$

Try (9) #8 : $A_s^- = 7.11 \text{ in}^2$

$$a = \frac{(7.11)(60{,}000)}{0.85(4000)(30)} = 4.18$$

$$c = 4.92$$

$$\varepsilon_s = \frac{0.003}{4.92}(15.5 - 4.92) = 0.0065 \geq 0.00207 \checkmark$$

$$\phi M_n^- = 0.9(7.11)(60{,}000)(15.5 - 4.18/2)/1000 \cdot 12$$
$$= 429^{1k}$$

$$\phi M_n^- = 429^{1k} > M_u^- = 390^{1k} \checkmark$$

Use (9) #8 Bars (A_s^-)

Asmin & Asmax:

$$A_{s,min} = \left| \begin{array}{l} \dfrac{3\sqrt{4000}}{60{,}000}(30)(15.5) = 1.47 \text{ in}^2 \\ \text{max} \quad \dfrac{200(30)(15.5)}{60{,}000} = 1.55 \text{ in}^2 \end{array} \right.$$

$$A_{s,min} = 1.55 \text{ in}^2 \begin{array}{l} < A_s^+ \\ < A_s^- \end{array} \checkmark$$

$$A_{s,max} = 0.85(0.85)\frac{4000}{60{,}000}\left(\frac{0.003}{0.003+0.004}\right)(30)(15.5)$$

$$= 9.6 \text{ in}^2 \begin{array}{l} > A_s^+ \\ > A_s^- \end{array} \checkmark$$

Check Min & Max Bars:

Table A-7:

3/4" Aggregate, 30" = bw, #8 bars } 11 bars maximum

6 bars < 11 bars ✓
9 bars < 11 bars ✓

Table A-8:

2" cover w/ Stirrup, bw = 30", #8 bars } 4 bars minimum

6 bars > 4 bars ✓
9 bars > 4 bars ✓

Check Shear Capacity:

$V_{max} = (5.1^k/ft)(30\ ft)/2 = 76.5^k$

$V_u @ d = 76.5^k - 5.1(15.5)/12 = \underline{69.9^k}$

$V_c = 58.9^k$

$\phi V_n = 0.75(0.5)2(1.0)(\sqrt{4000})(30'')(15.5)/1000$
$= 22.1^k$

$V_s = V_u/\phi - V_c = \frac{69.9}{0.75} - 58.9^k = 34.3^k$

$8\sqrt{4000}(30)(15.5) = 235^k > V_s$ ✓

$4\sqrt{4000}(30)(15.5) = 117.6^k \geq V_s$

then... $S_{max} = \min \left| \begin{array}{l} 15.5/2 = 7.75'' \\ 24'' \end{array} \right.$

$S_{max} = 7.75'' \rightarrow 7''$

$A_{v,min} = \left| \begin{array}{l} 0.75\sqrt{(4000)}(30'')(7'')/60{,}000 = 0.166\ in^2 \\ 50(30'')(7'')/60{,}000 = 0.175\ in^2 \end{array} \right.$ max

$A_{v,min} = 0.175\ in^2 \rightarrow$ 2 legs of #3

$A_v = 2(0.11) = 0.22\ in^2$

$S = A_v f_{yt} d/V_s = (0.22)(60{,}000)(15.5)/34.3\times1000$
$= 5.97'' \rightarrow 6''$

$S = 6''$

Use 2 legs of #3 @ 6" spacing

Distance where ϕV_n occurs:

$w_u = 5.1^{k/ft}$ $V_{max} = 76.5^k$

$0.5\phi V_c = 22.1^k$

$22.1^k = 76.5^k - 5.1(l_v)$

$l_v = 10.67\ ft = 128\ inches$

$2'' + (n-1)(6'') \geq 128''$

$n = 22$ stirrups

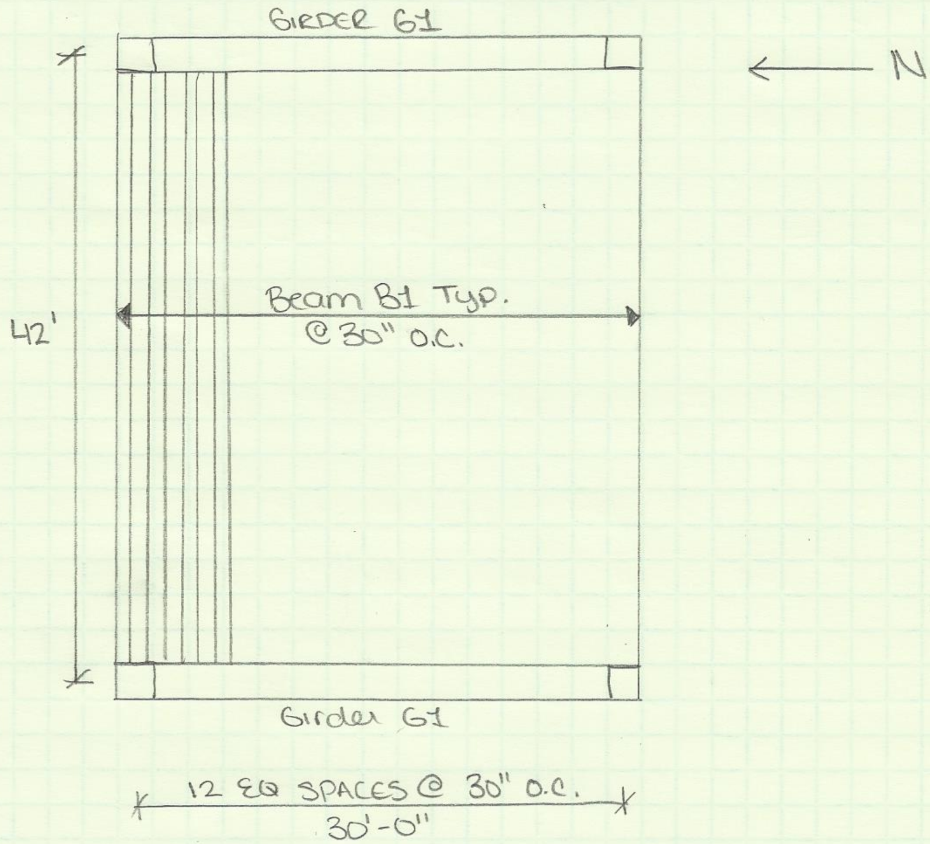
(22) #3 × @ 6" @ 2" away from each support

GIRDER DESIGN SUMMARY:

30" × 18" NW CONC, 4000 PSI
(6) #8 bars @ bottom midspan
(9) #8 bars @ top over supports
(22) #3 × @ 6" @ 2" away from face of each support

A. Stong1

Design # 4 Summary: One-Way Slab & Pan Joists



TYPICAL SLAB			
SLAB THICKNESS	N-S		E-W
	TOP	BOTTOM	
4.5"	#3 @ 8"	#3 @ 8"	#3 @ 8"

Notes:

1. 4000 PSI NW Concrete
2. $F_y = F_{yt} = 60000$ psi

BEAM SCHEDULE				
LABEL	SIZE BxH	LONG. REINF.		Shear Reinforcing
		BOTTOM	TOP	
B1	12"x18"	(3) #9	NONE	NONE
G1	30"x18"	(6) #8	(9) #8	#3 x [stirrup] @ 6"

COST AND WEIGHT ANALYSIS

OF DESIGN #4

Design #3: System Cost

Volume of Concrete:

$$\left(\frac{10''}{12}\right)(30 \times 42) + 4\left(\frac{17 \times 14}{144}\right)(30') + 2\left(\frac{33 \times 31}{144}\right)(42)$$

$$= 1845.1 \text{ ft}^3 \left(\tfrac{1}{27}\right)$$

$$= 68.3 \text{ CY}$$

$$V = (68.3)(108) = \$7380.30$$

Formwork:

$$(30' \times 42') + 4(2)(14''/12)(30') + 2(41''/12)(42')$$

$$+ 2(31''/12)(42) = 2044 \text{ ft}^2$$

$$F = (2044)(1.24) = \$2534.56$$

$$\text{TOTAL COST} = \$9914.86 / (42 \times 30)$$

$$\text{COST} = \$7.86/\text{SF}$$

From R.S. Means: $\dfrac{\$23.89/\text{SF}}{\$7.86/\text{SF}} = 3.039$

Factor To Scale
Method To Compare
w/ R.S. Means Values

Design #4: System Weight

Slab: $\left(\frac{4.5}{12}\right)(30' \times 42')(150) = 70875 \text{ lbs}$

Joists: $13\,\frac{(12'' \times 13.5'')}{144}(42')(150) = 92137.5 \text{ lbs}$

Girders: $2\,\frac{(30'' \times 13.5)}{144}(30')(150) = 25312.5 \text{ lbs}$

$$\text{TOTAL} = 188325 \text{ lbs} / (30' \times 42')$$

$$\boxed{W_T = 149.5 \text{ PSF}} < W_{T,\,system3} = 215.4 \text{ PSF}$$

Design #4: System Cost

Formwork ≈ \$1.24 / SF (material + installation)

Concrete ≈ \$108 / CY (material + installation)

Volume:

$$\left(\frac{4.5}{12}\right)(30' \times 42') + 13\,\frac{(12 \times 13.5)}{144}(42)$$

$$+ \frac{2(30 \times 13.5)}{144}(30) = 1255.5 \text{ ft}^3 \left(\frac{1 \text{ CY}}{27 \text{ ft}^3}\right)$$

$$V = 46.5 \text{ CY} \times \$108/\text{cy} = \underline{\$5022}$$

Formwork:

$$42' \times 30' + \frac{13.5}{12} \times 26 \times 42' + 30' \times \frac{13.5}{2} \times 4 = 3298.5 \text{ ft}^2$$

$$F = 3298.5 \text{ ft}^2 (\$1.24/\text{ft}^2) = \$4090.14$$

$$\text{TOTAL COST} = \$9112.14 / (42' \times 30')$$

$$\text{Cost} = \$7.23/\text{SF} \times \underset{\substack{\uparrow \\ \text{for comparison} \\ \text{w/ R.S. means}}}{3.039} = \$21.98/\text{SF}$$

OVERALL SYSTEM COMPARISIONS

Alyssa Stangl
Technical Report 3
Gravity System Comparisons

		Gravity Floor Systems				
Considerations		**Flat-Plate Concrete Slab**	**Non-Composite Steel**	**Composite Steel**	**One-Way Concrete Slab**	**One-Way Concrete Slab - Pan Joists**
Architectural Considerations						
Maximum Depth		18"	36.5"	36.5"	41"	18"
Fire Protection Required on structural members?		No	Yes	Yes	No	No
2 hr Fire Rating achieved between levels?		Yes	Yes	Yes	Yes	Yes
System Statistics						
Cost Per Square Foot		$21.53/SF	$38.19/SF	$25.50/SF	$23.89/SF	$21.98/SF
System Weight		180.4 PSF	80.6 PSF	79.8 PSF	215.4 PSF	149.5
Are vibrations major concern?		No	Yes	Yes	No	No
Durability		Acceptable	Acceptable	Acceptable	Acceptable	Acceptable
Future Design Considerations						
Lateral System Options	Reinforced Concrete Shear Walls	Yes	Yes	Yes	Yes	Yes
	Steel Moment Frame	No	Yes	Yes	No	No
	Steel Braced Frame	No	No	No	No	No
	Concrete Moment Frame	No	No	No	Yes	Yes
Advantages		Maximum floor to ceiling heights, cheapest system, no fire protection required	Light system weight, several options for lateral system	Lightest system weight, several options for lateral system, system used in LJC Tower I	Same material as existing system, cheap alternative	Same material as existing system, cheapest of alternative systems, same depth as existing system
Disadvantages		None	Most expensive system, fire protection required, large beam depth, vibrations	Higher cost than concrete systems, large beam depth, fire protection required	Heaviest system, large beam depth will significantly decrease floor to ceiling height	Heavy system but less than orginal system and other concrete alternative
Viable Option?		**N/A**	**Yes**	**Yes**	**No**	**Yes**