Letter of Transmittal

Date: October 17, 2014

To: Prof. Heather Sustersic
had132@engr.psu.edu

From: Young Jeon
ybj5001@psu.edu

Enclosed: AE 481W – Senior Thesis Structural Technical Report 3

Dear Prof. Sustersic,

The following report was prepared to be submitted for Technical Report 3 for AE 481W. This report contains spot check calculations of existing typical bay composed of prestressed precast concrete hollow core planks and load bearing masonry wall with masonry piers for gravity loads. This report also delivers you the three alternative framing system for the same typical bay checked previously. These include one-way concrete slab with beams and columns, non-composite steel framing, and composite steel framing.

Thank you very much for taking your time to review this report. I look forward to discussing it with you in the future.

Sincerely,

Young Jeon

Technical Report 3

Hakuna Resort

Swift Water, Pennsylvania

Image Courtesy of LMN Development LLC

Young Jeon
Structural Option
Advisor: Heather Sustersic
October 17th 2014

Table of Contents

Executive summary

Hakuna Resort is a jungle/safari theme hotel that includes a 217,703 square feet indoor water park as well as outdoor pool. The other side of the resort is convention centers which provides multiple meeting spaces. Divided into three distinctive spaces, the hotel is in between the indoor water park and convention space. These spaces are connected with expansion joints, therefore, can be looked at as three separate buildings.

The hotel building has total of eight stories above ground with total height of 101’-5” to the top of roof excluding the basement. With each floor having approximately 45,000 SF, the hotel portion of the resort has 395,938 SF by itself. The scope of this thesis project is limited to the hotel portion of the site; however, future assignment may incorporate an impactful design of hotel to improve cohesiveness of adjacent buildlings.

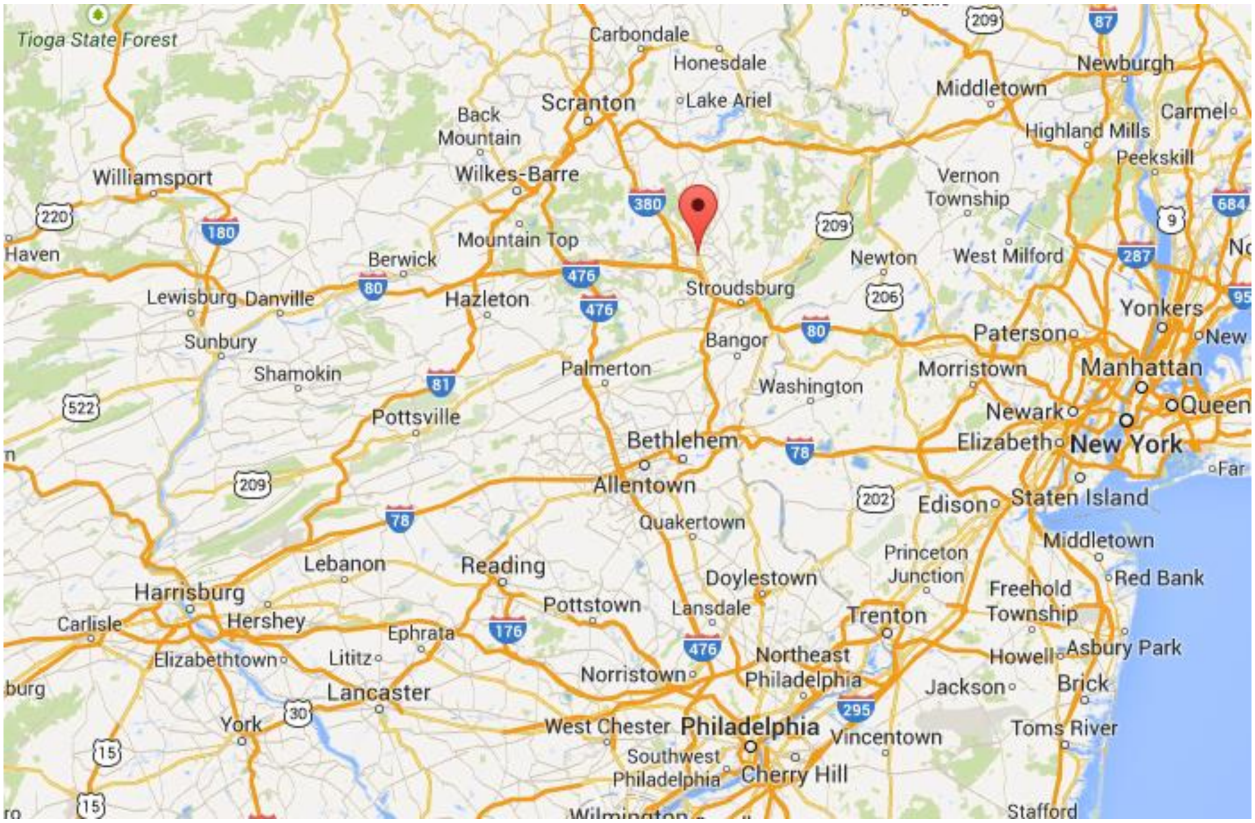
The foundation is consisted of cast in place concrete with footings and piers while north-west portion of building is partially unexcavated. The excavated portion of basement space is divided into usable rooms by concrete and masonry walls.

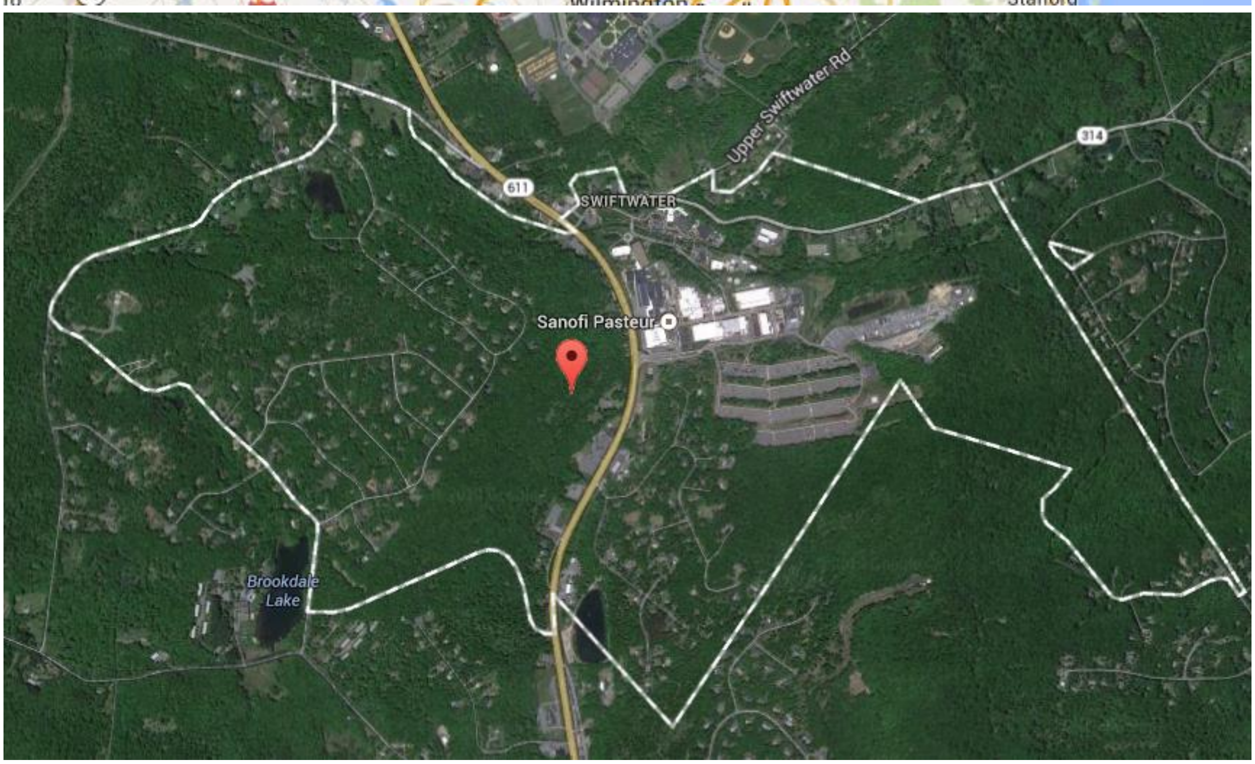
The typical elevated floor is 10” precast prestressed hollow core planks. At the excavated basement floor and first level floor above unexcavated foundation, a unique condition exists such that slab on grade concrete is used. The precast planks are supported by loadbearing masonry walls throughout the structure. However, in service areas like sauna, message and treatment on second floor, steel framing system is used to take advantage of opened frame system compared to solid shear wall that may block the view or pedestrian flow.

The nature of repetitive and typical hotel room floor layout allows the structural system to be simple and typical as well. The need of privacy also enabled the usage of masonry shear walls in between almost every room. Like mentioned earlier, these shear walls are supporting precast planks, therefore resisting gravity load.

In conclusion, while dominant structural system is masonry shear walls with precast planks, there are also structural wide flange steel framing in appropriate spaces, as well as reinforced concrete walls in lower levels. This usage of multiple structural systems will be analyzed throughout this report.

Building Site Information





Abstract

Hakuna Resort

Swiftwater, Pennsylvania

Project Team

Owner: LMN Development, LLC
Architect: Architectural Design Consultant
General Contractor: Kraemer Brothers, LLC
MEP/Structural: Harwood Engineering Consulants, LTD
Civil: Pennoni Associates, INC

General Building Data

Construction Dates: March 2014 - Summer of 2015
Building Cost: (Information Requested)
Delivery Method: Design Bid Build
Size: 395.938 SF

Architecture

The façade of hotel building has color tone of brown, red, and grey to give earth-like feeling. At the corners of building, architectural finish will be done to resemble ancient stone. Also little more distinctive color finishes will be used at the top of hotel façade to give tribal character to the building. The interior designs are also jungle theme. Most of the furniture in hotel have bark surface finishes.

Structural

Hakuna Resort is composed with three major components: indoor waterpark, hotel, and convention center. These components are connected by expansion joints, which allows each section to be looked at as separate independent buildings. As stated before, only hotel building will be described in this report due to its size. The main structural system used in this building is masonry shear walls and precast planks. There are also concrete piers, spread and strip footings, walls and masonry walls in the foundation and steel framing system in areas that require more flexible open spaces. The roof system is also precast hollow core planks.

Mechanical

(Not enough information - Material requested)

Lighting and Electrical

(Not enough information - Material requested)

Sustainability

(Not enough information - Material requested)

Young Jeon - Structural Option https://www.engr.psu.edu/ae/thesis/portfolios/2015/ybj5001/index.html

Documents Used to Create This Report

Masonry Standards Joint Committee

- Building Code Requirements and Specification for Masonry Structures
  - Building Code Requirements for Masonry Structures
    - TMS 402-11 / ACI 530-11 / ASCE 5-11
  - Specification for Masonry Structures
    - TMS 602-11 / ACI 530.1-11 / ASCE 6-11

Concrete Masonry Association of California and Nevada

- 2009 Design of Reinforced Masonry Structures

American Concrete Institute

- ACI 318-08 – Building Code Requirements for Structural Concrete and Commentary

Precast / Prestressed Concrete Institute

- PCI MNL 120-04 – PCI Design Handbook, Precast and Prestressed Concrete 6^{th} Edition
- PCI Manual for the Design of Hollow Core Slabs 2^{nd} Edition

American Institute of Steel Construction

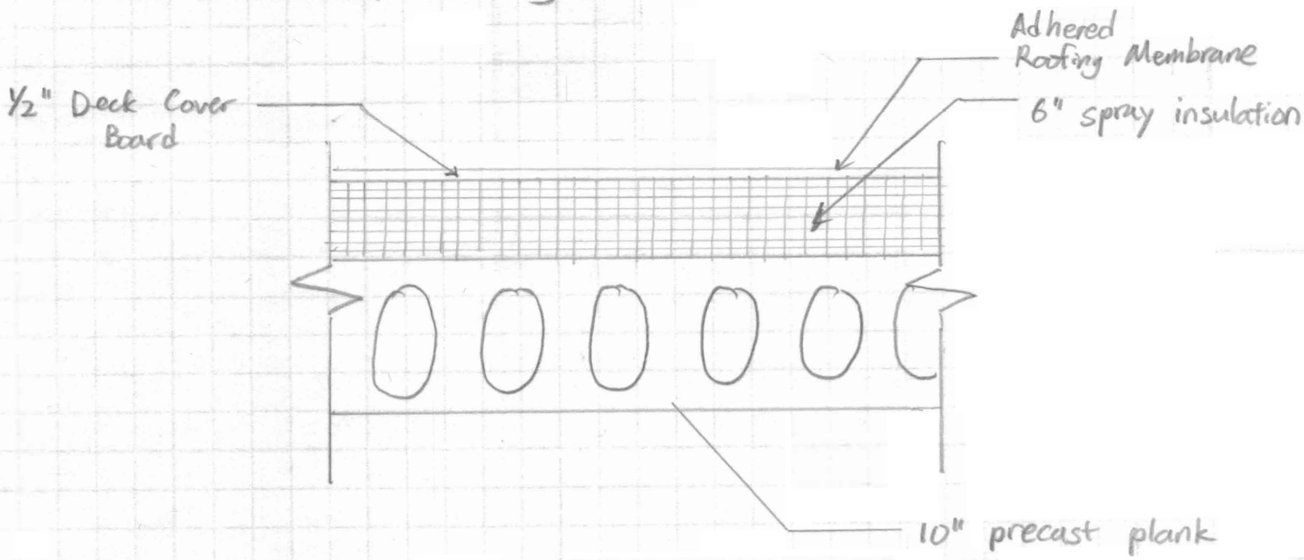
- Steel Construction Manual 14^{th} Edition

Hakuna Resort Construction Documents

- Architectural and Structural Sets

Gravity Load Calculations

Typical Roof Bay Loading



Roof dead load:

Adhered Roofing Membrane = 2 psf
1/2" Deck Cover Board = 2 psf.
* 6" Spray Insulation = 0.5 pcf × 0.5 ft = 0.25 psf.
* 10" precast plank = 68 psf.
Superimposed / misc. = 10 psf.

Total = 82.25 psf

* All values from manufacturer.

Roof Live Load:

$L_r = 20$ psf (ASCE 7-05 Table 4-1)

★ Roof live load for the design not shown in the drawing.

Snow Load:

According to Figure 7-1 in ASCE 7-05, Case Studies are required. Therefore, used suggested load on the structural drawing.

$P_g = 40$ psf

$C_e = 1$

$I = 1$

$C_t = 1.1$

$P_f = P_g(0.7)(C_e)(I)(C_t) = 0.7(40)(1)(1)(1.1) = 30.8$ psf

★ The structural design document used 35 psf for conservativity.

$\gamma = 0.13\,p_g + 14 < 30$ pcf.

$\gamma = 0.13(40) + 14 = 19.2$ pcf < 30 pcf ✓

$h_b = \frac{P_f}{\gamma} = \frac{35}{19.2} = 1.82$ ft.

$h_r = 4$ ft.

$h_c = h_r - h_b = 4 - 1.8 = 2.18$ ft.

$$\frac{h_c}{h_b} = \frac{2.18}{1.82} = 1.19 > 0.2$$ ∴ Drift loads are required.

For Roof Projection Condition. (East-West)

$$h_d = 0.75\left((0.43)(L_{u_{lower}})^{1/3}(P_g+10)^{1/4} - 1.5\right)$$

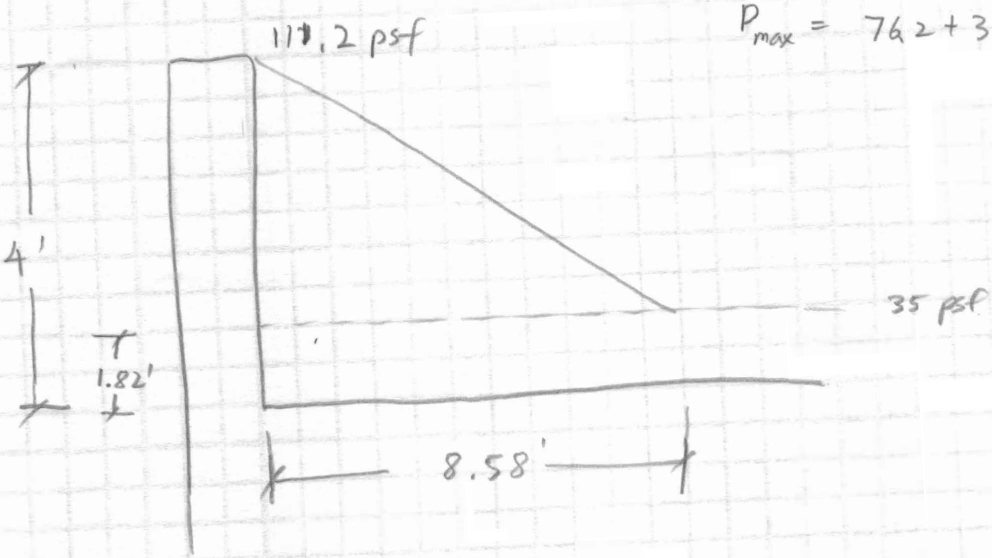
$$h_d = 0.75\left((0.43)(589')^{1/3}(40+10)^{1/4} - 1.5\right)$$

$$h_d = 5.69' > h_c \rightarrow h_d = h_c = 2.18.$$

$$w = \frac{4h_d^2}{h_c} = \frac{4(2.18)^2}{2.18} = 8.71\ ft \le 8h_c = 17.4\ ft.$$

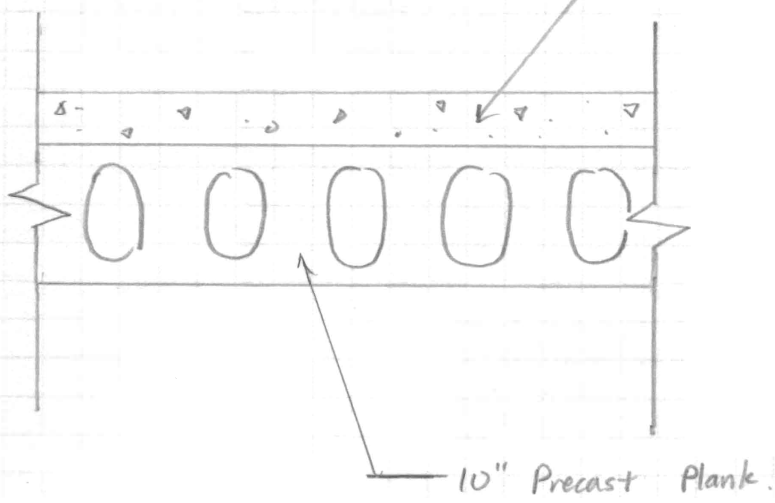
$$p_d = p_f\, h_d = 35(2.18) = 76.2\ psf$$

$$P_{max} = 76.2 + 35 = 111.2\ psf.$$



* Same snow drif load in North-South direction due to large $L_{u_{lower}}$.

Typical Floor Bay Loading



Floor Dead Load:

3" Composite Topping =

$(150\ pcf)(3/12'') =$ 37.5 psf.

10" Precast Plank = 68 psf

Superimposed / Misc = 10 psf

Total = 115.5 psf.

Floor bay live load:

ASCE 07-05 Table 4-1.

Lobbies	=	100 psf
Corridors	=	100 psf
Hotel room (private)	=	40 psf
Hotel rooms (public)	=	100 psf
Partition (minimum)	=	15 psf.

"Typical floor above grade including partition" from structural drawing = 55 psf

ASCE 7-05

Hotel Room (Private) Partition

40 psf + 15 psf = 55 psf. ✓

Non - Typical Dead Load

- Balcony roof. (Above balconies)
  - Plywood deck attachment - 5 psf
  - light gage steel truss - 15 psf

  20 psf.

- 12" precast plank w/ 3" Topping. (Luxury suite, longer floor span).

  12" plank =

  (150 pcf) (12/12) = 150 psf.

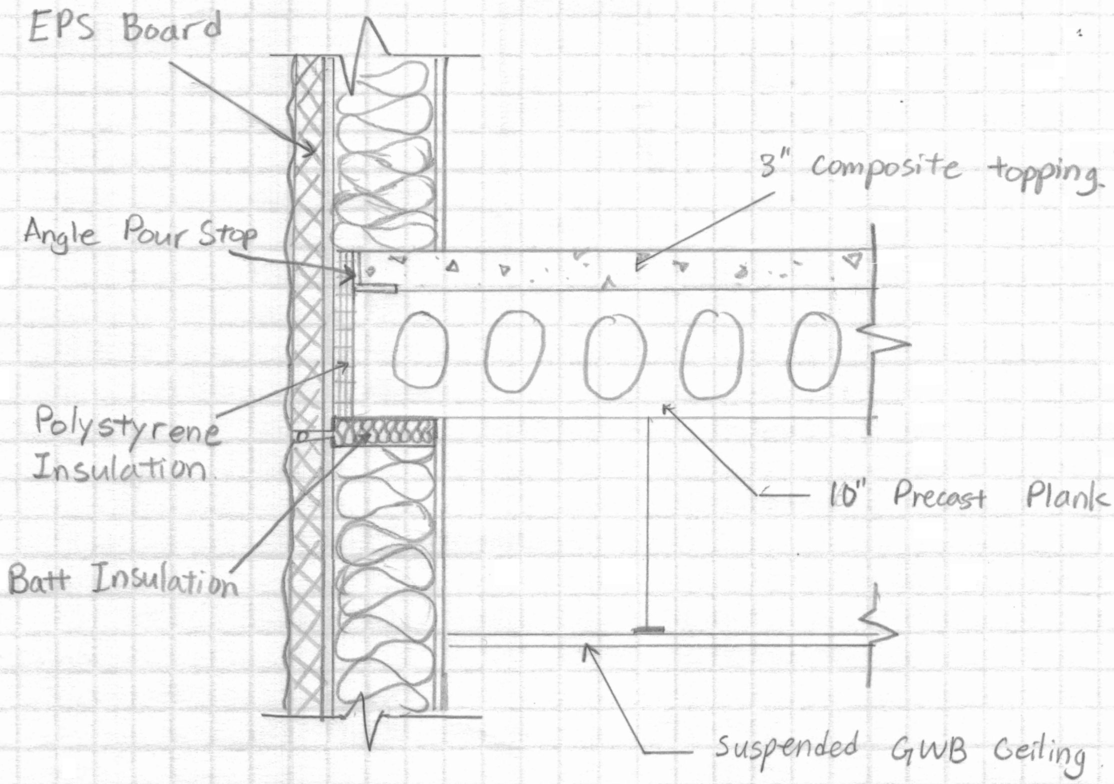
  3" Composite topping = 37.5 psf

  187.5 psf.

Non- Typical Live Load - from ASCE -05 T. 4-1

- Computer and file room = 100 psf
- Storage = 125 psf.

Typical Exterior Wall Load



Wall dead load:

EPS Board : 8 psf × 10.83' = 86.64 plf

Insulations : 5 psf × 10' = 50 plf

Wall Board : 6 psf × 4' = 24 plf

160.64 plf

Load path:

Typical exterior walls are not load bearing. The plank shown above spans in and out of the page, which is than supported by interior masonry shear walls that are normal to the shown exterior wall. Therefore, the weight of exterior wall is carried through floor planks to shear walls.

Existing Typical Member Spot Checks for Gravity Loads

Typical Bay and Pier Analyzed for Gravity Loads

Precast Prestressed Hollow core Concrete Plank

Plank : $f'_c = 6000$ psi
$E_{ci} = 3737$ ksi
$E_c = 4696$ ksi
Topping : $f'_c = 4000$ psi
$E_c = 3605$ psi
} By manufacturer.

Prestressed steel : 6 - ½" dia. strands
$f_{pu} = 270$ ksi.
0.153 in² / strand
4' wide plank.
web width = 13.125"

Plank Span : 27' - 10"

Non. Comp.
$I = 3196 \text{ in}^4$
$A = 261.6 \text{ in}^2$
$y = 5''$

composite section properties — from manufacturer.

$A_c = 336.912 \text{ in}^2$
$I_c = 5332.07 \text{ in}^4$
$Y_{bc} = 6.335$ in

Loads:
plank s.w = 68 psf
topping = 37.5 psf
S.I. DL = 10 psf
LL = 40 psf (Hotel private)
15 psf (partitions)

Prestress Loss

* Calculation referenced from PCI Manual for the Design of Hollow Core Slabs

** initial stress = 60% f_{pu}.

$A_{ps} f_{pu} = 0.153\,(270) = 41.3^k$ / strand.

$M_g = \frac{(27.83)^2}{8}(0.068)(4) = 316$ k-in.

$P_i = 0.6(6)(41.3) = 148.68^k$.

$$f_{cir} = K_{cir}\left(\frac{P_i}{A} + \frac{P_i e^2}{I}\right) - \frac{M_g e}{I} = 0.9\left(\frac{148.68}{336.912} + \frac{148.68(4.585)^2}{3196}\right) - \frac{316(4.585)}{3196.}$$

$f_{cir} = 0.82$ ksi.

* error : $e = y - d_{st}$.
$= 5 - 1.75$
$= 3.25''$

$E_s = 29{,}000$ ksi

$$ES = K_{es}\frac{E_s}{E_{ci}} f_{cir} = (1.0)\frac{29000}{3737}(0.82) = \boxed{6.36 \text{ ksi}}$$

Concrete Creep:

$$M_{sd} = \frac{(27.83)^2}{8}(0.0375 + 0.01)(4) = 18.4^{1k} = 220.7 \text{ k-in}$$

$$f_{cds} = \frac{M_{sd}\, e}{I} = \frac{(220.7)(4.585)}{3196} = 0.317 \text{ ksi}$$

Prestress loss due to creep:

$$CR = K_{cr}\frac{E_s}{E_c}(f_{cir} - f_{cds}) = 2.0\,\frac{29000}{4696}(0.82 - 0.317) = 6.21 \text{ ksi.}$$

Shrinkage of Concrete

$$\frac{V}{S} = \frac{Area}{Perimeter} = \frac{261.6\,in^2}{48 + 46.125 + 2\times10} = 2.29$$

use RH = 70% (Fig. 2.2.3.1)

$$SH = 8.2\times10^{-6} K_{sh} E_s \left(1 - 0.06\frac{V}{S}\right)(100 - RH)$$

$$= 8.2\times10^{-6}(1.0)(29000)(1 - 0.06(2.29))(100-70)$$

$$= 6.15 \text{ ksi.}$$

Steel Relaxation

$K_{re} = 5000$ $J = 0.04$ (Table 2.2.3.1)

$$\frac{f_{si}}{f_{pu}} = 60\% \rightarrow C = 0.33$$

$$RE = \left[K_{re} - J(SH + CR + ES)\right]C$$

$$= \left[\frac{5000}{1000} - 0.04(6.15 + 6.21 + 6.36)\right]0.33$$

$$RE = 1.4 \text{ ksi.}$$

Total Loss at Midspan.

$$Loss = 6.15 + 6.21 + 6.36 + 1.4 = 20.12 \text{ ksi}$$

$$\frac{20.12}{(0.6)(270)} = \boxed{12.4\%}$$

Service Load Stress

$$A_{ps} f_{se} = 0.6(6)(41.3)(1 - 0.124) = 130^{k}$$

$$M_{non-comp.} = \frac{27.83^2}{8}(0.068 + 0.0375)12 = 122.6 \text{ in-k/ft}$$

$$M_{comp} = \frac{27.83^2}{8}(0.01 + 0.055)12 = 75.5 \text{ in-k/ft}$$

At top of topping.

$$f_{top} = \frac{75.5(4)(13-6.335)}{5332.07}\left(\frac{3605}{4696}\right) = 0.42 \text{ ksi.}$$

At top of plank

$$f_{top} = \frac{130}{261.6} - \frac{130(3.25)(5)}{3196} + \frac{122.6(4)(5)}{3196} + \frac{75.5(4)(10-6.335)}{5332.07}$$

$$= 0.811 \text{ ksi.}$$

At bottom of plank

$$f_{bot} = \frac{130}{261.6} + \frac{130(3.25)(5)}{3196} - \frac{122.6(4)(5)}{3196} - \frac{75.5(4)(6.335)}{5332.07}$$

$$= 0.032 \text{ ksi}$$

Permissible Compression.

$0.45 f'c = 0.45(6000) = 2.7 \text{ ksi} > 0.42 \text{ ksi}$ ✓

$0.6 f'c = 0.6(6000) = 3.6 \text{ ksi} > 0.811 \text{ ksi}$ ✓

Permissible Tension

$7.5\sqrt{f'c} = 7.5\sqrt{6000} = 0.581 \text{ ksi} > 0.032 \text{ ksi}$ ✓

Flexural Strength.

$w_u = 1.2(0.068 + 0.0375 + 0.01) + 1.6(0.055)$

$= 0.2266 \text{ ksf}$

$M_u = 87.75^{\prime k}$.

$$\rho_p = \frac{A_{ps}}{b d_p} = \frac{6(0.153)}{48(10-1.75)} = 0.0023$$

$$\beta_1 = 0.85 - \left(\frac{6000-4000}{1000}\right)0.05 = 0.75$$

$\gamma_p = 0.28$ for low relax. strands

$$f_{ps} = 270\left[1 - \frac{0.28}{0.75}\left(0.0023\,\frac{270}{6}\right)\right] = 259.6 \text{ ksi}$$

$$\omega_p = \frac{\rho_p f_{ps}}{f'_c} = \frac{0.0023(259.6)}{6} = 0.0995$$

$$0.36\beta_1 = 0.27 > 0.0995 \quad \checkmark \quad \underline{\text{OK}}$$

$$a = \frac{A_{ps} f_{ps}}{0.85 f'_c b} = \frac{6(0.153)(259.6)}{0.85(6)(48)} = 0.974 \text{ in.}$$

Top flange thickness $= 1.563'' > 0.974''$ $\checkmark$ $\underline{\text{ok}}$.

$$\phi M_n = 0.9(6)(0.153)(259.6)\left(11.25 - \frac{0.974}{2}\right)$$

$$= 2308.5 \text{ k-in.} = 192.4 \text{ 'k.}$$

Check $1.2 M_{cr}$.

$$f_{bot} = \frac{130}{261.6} + \frac{130(3.25)(5)}{3196} = 1.158 \text{ ksi}$$

$$M_{cr} = \frac{5332.07}{6.335}\left(1.158 + \frac{7.5\sqrt{6000}}{1000}\right) = 1463 \text{ k-in} = 121.97 \text{ 'k}$$

$$\frac{\phi M_n}{M_{cr}} = \frac{192.4}{121.97} = 1.58 > 1.2 \quad \checkmark \quad \underline{\text{OK}}$$

Check Horizontal Shear.

$\phi V_{nh} = \phi\, 80\, b_v d = 0.85(80)(48)(11.25) = 36.7^k$

at $h/2$,

$V_u = \left(\frac{27.83}{2} - \frac{13}{2(12)}\right)(0.2266)(4) = 12.1^k < 36.7^k$ ✓ ok

Web shear @ $h/2$ + 3" bearing.

transfer length = 25"

$A_{ps} f_{se} = 130\left(\frac{10}{25}\right) = 52^k$

$f_{pc} = \frac{52}{261.6} - \frac{52(3.25)(6.335-5)}{3196} = 0.128$ ksi

$\phi V_{cw} = 0.85\left[3.5\sqrt{f'_c} + 0.3 f_{pc}\right] b_w d$

$= 0.85\left[3.5\sqrt{6000} + 0.3(0.128)\right](13.125)(11.25)$

$= 34.0^k > 12.1^k$ ✓ ok

Initial Camber

$\frac{P_e \ell^2}{8E_c I} - \frac{5w\ell^4}{384EI} = \frac{(1-0.124)(148.68)(4.585)(27.83 \times 12)^2}{8(3737)(5332)}$

$- \frac{5(4)(0.068+0.0375)(27.83)^4(1728)}{384(3737)(5332)}$

$= 0.418" - 0.286"$

$= 0.132"$

Long Term Camber – Long term multiplier from Table 2.4.1

$2.2(0.418) - 2.4(0.286)$

$= 0.2332''$ ∴ approx. 1/4" final camber.

Deflection.

$P = A_{topping}\,(\text{strain})(\text{modulus})$ differential shrinkage = .2

$= 48''(3'')(0.00025)(3605)$

$= 129.78^{k}.$

$P = \dfrac{129.78}{2.3} = 56.4^{k}.$ ← Reduction due to conc. creep. (factor from Table 2.4.1)

$e = 13'' - 5'' = 8''$

$M = Pe = (56.4)(8) = 451.2^{\,k\text{-}in}$

Downward deflection $= \dfrac{Ml^2}{8EI} = \dfrac{451.2\,(27.83\times 12)^2}{8\,(4696)(5332)}$

$= 0.251''.$

$\dfrac{L}{360} = \dfrac{27.83\times 12}{360} = 0.93'' > 0.251''$ ✓

OK

Masonry Pier Load Summary

Tributary area of typical masonry pier 3 = $172.64\ ft^2$.

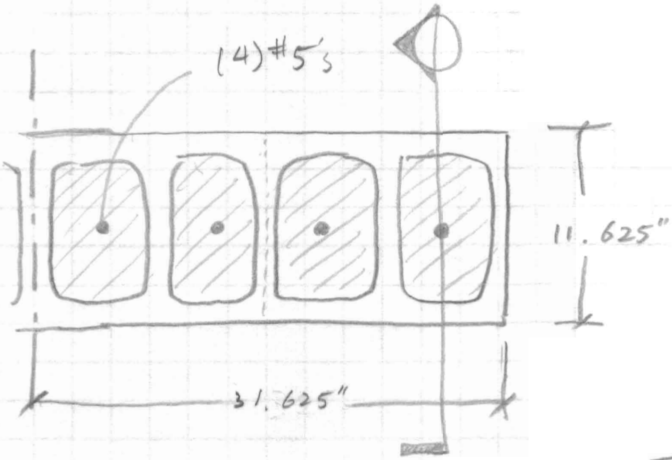
78% private hotel room (40psf) + partitions (15 psf)

22% Corridor (100 psf)

	DL	LL	Non-factored Load
Roof	82.25 psf	35 psf	$= 6.125^k$
8th floor	115.5 psf	$(.78)(55)+(0.22)(100)$	$= 31.144^k$.
7th			
6th			
5th			
4th			
3rd			
2nd	115.5	100 psf	37.2^k

Total $= 230.2^k$.

Masonry Pier #3 Interaction Diagram.



Compression Limit =

$P_a = (0.25 A_n f'_m + 0.65 A_{st} F_s) R$

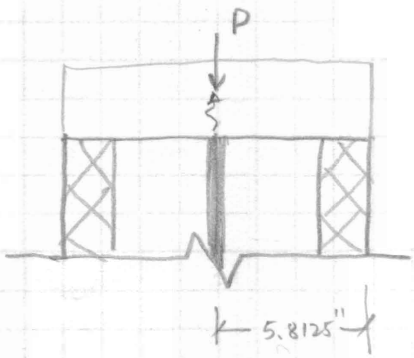
Lateral tied reinforcement.

★ No ties for pier #3.

For solid grout, $r = \frac{d}{\sqrt{12}} = \frac{11.625}{\sqrt{12}} = 3.356"$

Effective Height = 97'-4" = 1168"

① Pure Compression.



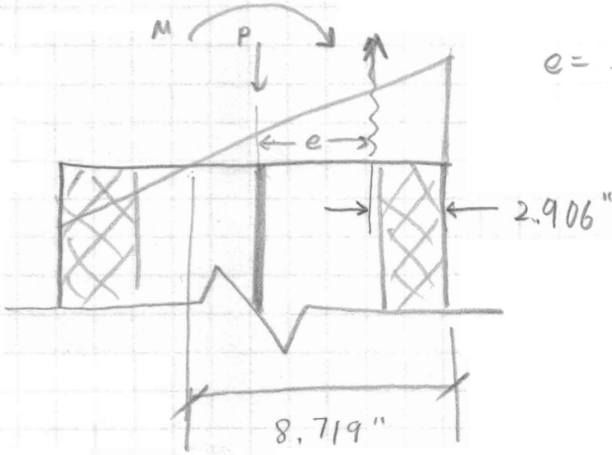
$R \Rightarrow \frac{h}{r} = \frac{1168}{3.356} = 348 > 99$

$R = \left(\frac{70r}{h}\right)^2 = \left(\frac{70(3.356)}{1168}\right)^2 = 0.04$

$P_a = 5.5^k$.

★ Note: Due to very long height of pier compared to its size, the compression limit is reduced greatly due to R.

② No tension in the steel.

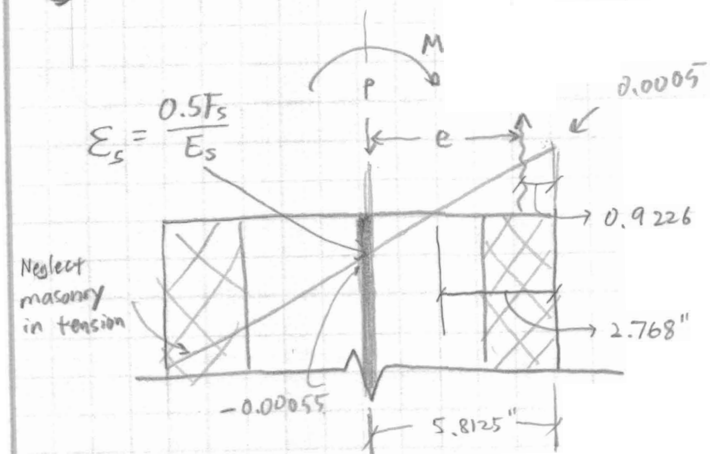


$e = \frac{11.625}{2} - 2.906 = 2.9065$

③ Tension Control ($\frac{1}{2} F_s$)

Grade 60 steel

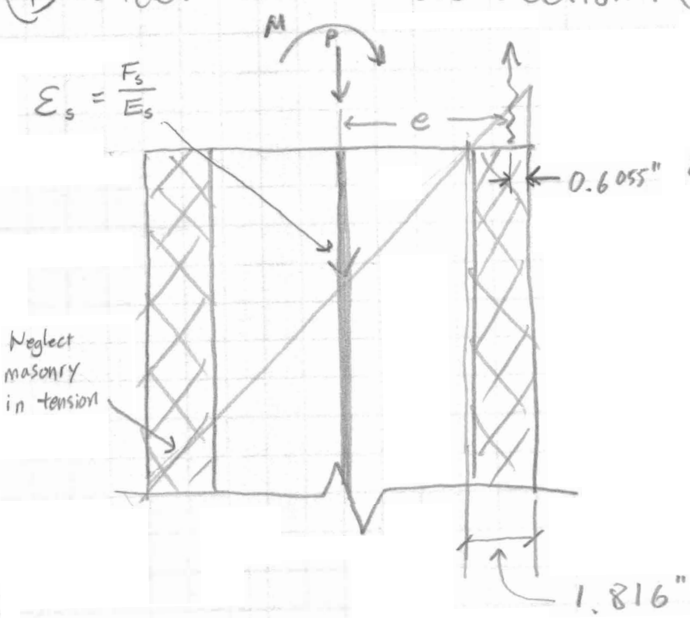
$F_s = 32\ ksi$



$$\varepsilon_s = \frac{0.5F_s}{E_s} = \frac{0.5(32)}{29{,}000} = -0.00055$$

$$e = 5.8125 - 0.9226 = 4.8899$$

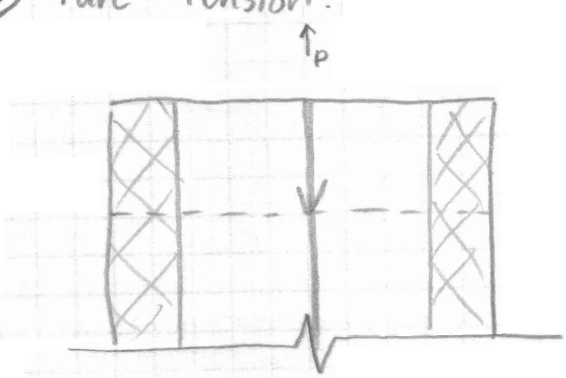
④ Steel Controls Cross Section. ($f_s = F_s$)



$$\varepsilon_s = \frac{F_s}{E_s} = \frac{32}{29{,}000} = -0.00110$$

$$e = 5.8125 - 0.6055 = 5.2070''$$

⑤ Pure Tension.



Masonry Pier 3 Interaction Diagram

f'm 1500
Em 1350
Es 29000
grade 60 steel
12" mominal thickness

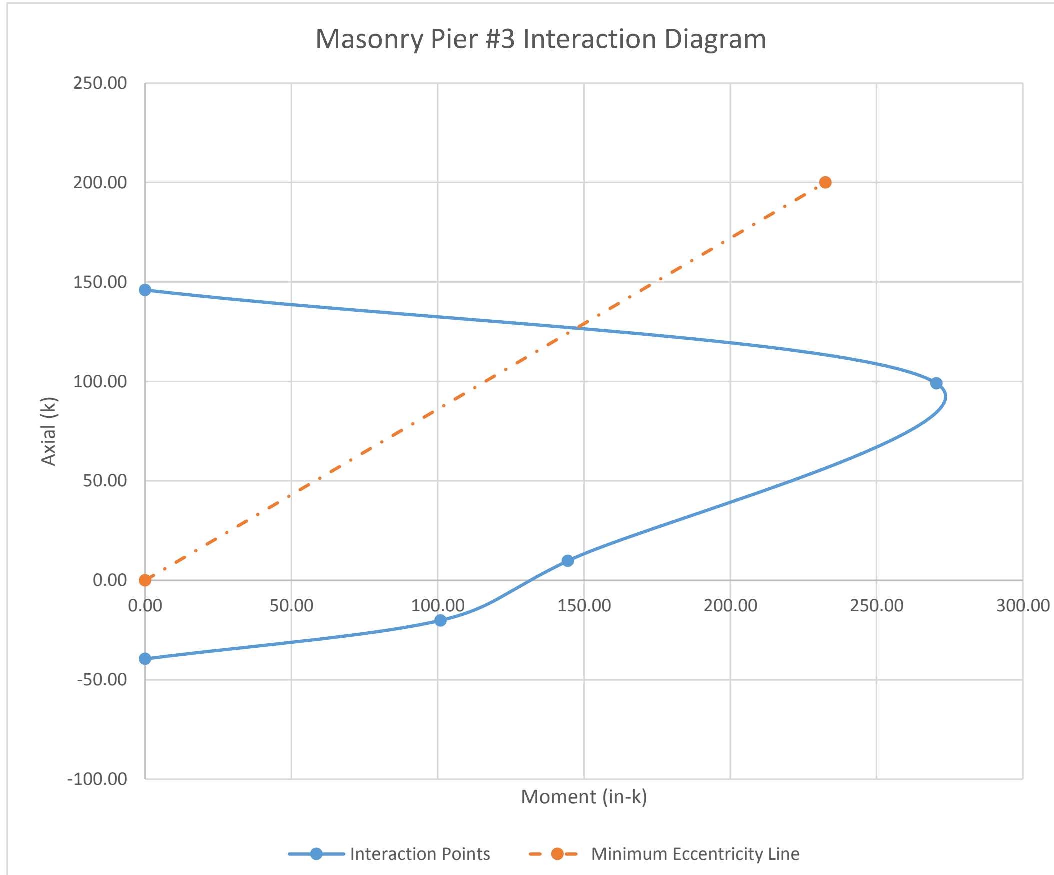
Point 1	Pure Compression (neglecting slenderness)					
	d (in)	Area (in^2)	Strain (in/in)	Force (k)	e (in)	Moment (in-k/ft)
Masonry	5.8125	379.2656	0.0005	128.00	0.0000	0.00
Steel Layer	5.8125	1.2400	0.0005	17.98	0.0000	0.00
			P1 =	145.98	M1 =	0.00

Point 2	No net tension at outside face of masonry					
	d (in)	Area (in^2)	Strain (in/in)	Force (k)	e (in)	Moment (in-k/ft)
Masonry	2.9063	275.7400	0.0005	93.06	2.9063	270.46
Steel Layer	5.8125	1.2400	0.0002	5.99	0.0000	0.00
			P2 =	99.06	M2 =	270.46

Point 3	Tension Control					
	d (in)	Area (in^2)	Strain (in/in)	Force (k)	e (in)	Moment (in-k/ft)
Masonry	0.9226	87.5335	0.00050	29.54	4.8899	144.46
Steel Layer	5.8125	1.2400	-0.00055	-19.78	0.0000	0.00
			P3 =	9.76	M3 =	144.46

Point 4	Steel Controls Section					
	d (in)	Area (in^2)	Strain (in/in)	Force (k)	e (in)	Moment (in-k/ft)
Masonry	0.6055	57.4438	0.0005	19.39	5.2070	100.95
Steel Layer	5.8125	1.2400	-0.0011	-39.56	0.0000	0.00
			P4 =	-20.17	M4 =	100.95

Point 5	Pure Tension					
	d (in)	Area (in^2)	Strain (in/in)	Force (k)	e (in)	Moment (in-k/ft)
Masonry	5.8125	379.2656	0.0000	0.00	0.0000	0.00
Steel Layer	5.8125	1.2400	-0.0011	-39.56	0.0000	0.00
			P5 =	-39.56	M5 =	0.00



*Note: The compression limit line is not shown due to error which produces very low value of compression limit.
** Due to an error, the gravity load in the masonry pier 3 exceeds the interaction diagram boundry.

Alternative Framing System for Gravity Load:

Alternative 1 – One Way Concrete Slab with Beams and Girders

Alternate System #1 : One way concrete slab with beams and girders.

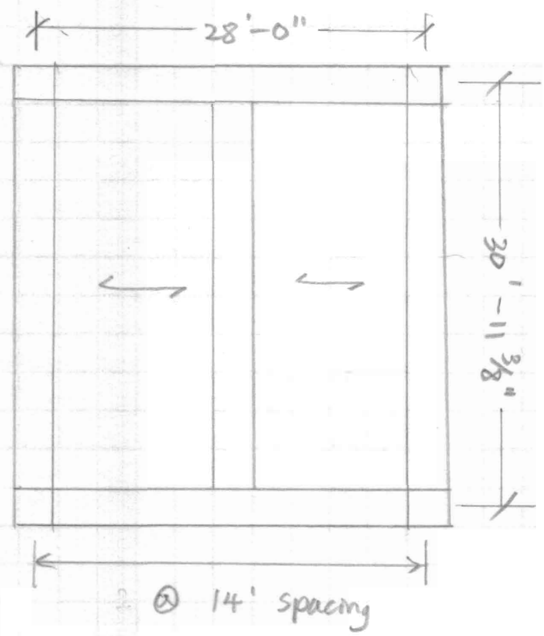
Live Load

Residential - Hotel Private Room - 40 psf

Partitions - 15 psf.

Dead Load

Superimposed dead load - 10 psf.



Assume: $f'_c = 6000\,psi$

$f_y = 60\,ksi$.

Minimum thickness of slab (ACI 318-08 Table 9.5 (a))

$$h_{min} = \frac{l}{28} = \frac{14 \times 12}{28} = 6\,in.$$

Let $d = 5"$

Live load = 40 + 15 = 55 psf.

Dead load = $10 + (150\ pcf)(6''/12) = 85$ psf.

$w_u = 1.2(85) + 1.6(55) = 190$ psf.

Design Moment

$M_u^- = \frac{w_u l^2}{11} = \frac{190(14)^2}{11} = 3.385^{1k}$

$M_u^+ = \frac{w_u l^2}{16} = \frac{190(14)^2}{16} = 2.327^{1k}$

Estimate A_s.

$A_s = \frac{M_u}{4d} = \frac{3.103}{4(5)} = 0.155\ in^2/ft$

Use #4 bar @ 12" o.c. = $0.2\ in^2/ft$

$d = 6'' - 1'' - \frac{0.5}{2} = 4.5''$

Check $\phi M_n > M_u$

Assume $\varepsilon_s > \varepsilon_y$

$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{0.2(60)}{0.85(6)(12)} = 0.196''$

$\beta_1 = 0.85 - \frac{0.05(6000-4000)}{1000} = 0.75$

$c = \frac{a}{\beta_1} = \frac{0.196}{0.75} = 0.261''$

$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{0.261}(4.5 - 0.261) = 0.0487 > 0.005$

$\therefore \phi = 0.9$

$\phi M_n = \phi A_s f_y (d - \frac{a}{2}) = 0.9(0.2)(60)(4.5 - \frac{0.196}{2})/12 = 3.96^{1k}$

$3.96^{1k} > 3.385^{1k}$ $\therefore$ Slab OK for flexural strength.

Check one-way shear

$q_u = 190 \text{ psf} = 0.19 \text{ ksf}$

$V_u = 0.19\left(30.95 \times \left(14 - \frac{4.5}{12}\right)\right) = 80.1^k$

$V_c = 2\lambda\sqrt{f'_c}\, b_w d$

$2(1.0)\sqrt{6000}\,(30.95 \times 12)(4.5) = 259^k$

$\phi V_c = 0.75(259) = 194.25^k$

$194.25^k > 80.1^k$

$\therefore$ Slab is OK for one way shear

Crack Control

$S \le 15\left(\frac{40{,}000}{f_s}\right) - 2.5\, C_c$

$\le 15\left(\frac{40{,}000}{2/3(60{,}000)}\right) - 2.5(0.75)$

$\le 13.1''$

$\therefore$ 12" spacing OK.

Temperature & Shrinkage Reinforcement.

$A_t = 0.0018\, b \cdot h = 0.0018(12)(6) = 0.13 \text{ in}^2$

use #4 @ 12" O.C. $= 0.2 \text{ in}^2 > 0.13 \text{ in}$ ✓

Beam

Assume : $f'_c = 6000$ psi
$f_y = 60{,}000$ psi

$$w_u = [1.2(85) + 1.6(55)]\ 7' = 1.33\ klf$$

$$M_u = \frac{w_u l^2}{8} = \frac{1.33\,(30.95)^2}{8} = 159.25^{\,1k}$$

$$159.25^{k} \times 1.1 = 175.18^{\,1k}$$

estimate of self weight

Estimate size

$$bd^2 = 20\,M_u$$

Try $b = \frac{2}{3}d \Rightarrow \frac{2}{3}d^3 = 20(175.18)$

$$d = 17.4''$$

$$h = d + 2.5 = 17.4 + 2.5 = 19.9'' \Rightarrow 20''$$

use $d = 18'' \Rightarrow b = \frac{2}{3}d = \frac{2}{3}(18) = 12''$

Compute Self Weight Effect.

$$w_{sw} = \frac{12(20)}{144} \times 150 = 250\ plf$$

$$w_u = 1.33 + 1.2(0.25) = 1.63\ klf$$

$$M_u = \frac{1.63\,(30.95)^2}{8} = 195.2^{\,1k}$$

Required steel

$$A_s = \frac{M_u}{4d} = \frac{195.2}{4(18)} = 2.71 \Rightarrow \text{use } (5)\ \#7 = 3\ in^2$$

Check Flexural Strength.

$$a = \frac{3(60)}{0.85(6)(12)} = 2.94 \qquad c = \frac{a}{\beta_1} = \frac{2.94}{0.75} = 3.92$$

$$M_n = \frac{3(60)(18 - \frac{2.94}{2})}{12} = 247.95^{k}$$

$$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{3.92}(18 - 3.92) = 0.0108 > 0.00207$$

$\therefore$ OK

$$\phi M_n = 0.9(247.95) = 223.16^{k} > 195.2^{k}$$

$\therefore$ Beam is ok for flexural strength.

Check Minimum Reinforcement

$$A_{s,min} = \max \begin{cases} \dfrac{3\sqrt{f'_c}}{f_y} b_w d = \dfrac{3\sqrt{6000}}{60{,}000}(12)(18) = 0.837 \\ \dfrac{200\, b_w d}{f_y} = \dfrac{200(12)(18)}{60{,}000} = 0.72 \end{cases}$$

$$A_s = 3\ in^2 > 0.837\ in^2 \checkmark$$

Check Maximum Reinforcement.

$$A_{s,max} = 0.85\beta \frac{f'_c}{60}\left(\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_y}\right) b_w d = 0.85(0.75)\frac{6}{60}\left(\frac{0.003}{0.003 + 0.004}\right)(12)(18)$$

$$= 5.9\ in^2$$

$$A_s = 3\ in^2 < 5.9\ in^2 \checkmark$$

Check Minimum Spacing.

min. clear distance b/w bars = max $\begin{cases} d_s = 0.875'' \\ 1'' = 1'' \\ 4/3\ S_u = 4/3'' \end{cases}$

Actual spacing

$$s = \frac{12 - 1.5(2) - 0.5(2) - 0.875(5)}{4} = 0.9'' \not> 4/3'' \; X$$

Try again

Change A_s to (4) #8 = $3.16\ in^2$ ⇒ $0.837 < 3.16 < 5.9$ ✓

$$S = \frac{12 - 1.5(2) - 0.5(2) - 1(4)}{3} = 1.33'' > 4/3'' \checkmark$$

Shear Strength

$$V_u = (1.63)\left(\frac{30.95}{2} - \frac{18}{12}\right) = 22.8^k$$

$$V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2(1.0)\sqrt{6000}\,(12)(18) = 33.5^k$$

$$0.5\,\phi V_c = 0.5(0.75)(33.5) = 12.56^k < 22.8^k$$

∴ Stirrups required.

Shear strength required by reinforcements

$$V_s = \frac{V_u}{\phi} - V_c < 8\sqrt{f'_c}\, b_w d$$

$$= \frac{22.8}{0.75} - 33.5 < 8\sqrt{6000}\,(12)(18)$$

$$= -3.1 < 133.9^k \checkmark$$

Max spacing of shear reinforcement

$$V_s \le 4\sqrt{f'_c}\, b_w d = 4\sqrt{6000}\,(12)(18) = 66.9^k$$

$$\Rightarrow s_{max} = \min \begin{cases} d/2 = 18/2 = 9'' \\ 24'' \end{cases}$$

Min Shear Reinforcing

$$A_v = \max \begin{cases} \dfrac{0.75\sqrt{6000}\,(12)(9)}{60{,}000} = 0.104 \text{ in}^2 \\ \dfrac{50\,(12)(9)}{60{,}000} = 0.09 \text{ in}^2 \end{cases}$$

use 2 legs of #3 stirrups @ 9''

$$0.11 \times 2 = 0.22 > 0.104 \text{ in}^2$$

Design Shear Reinforcement

$$s = \frac{A_v f_{ye}\, d}{V_s} = \frac{0.22(60)(18)}{3.1} = 76.6'' > s_{max}$$

use $s = @\ 9''$.

Stirrup layout

Terminate stirrups at $V_u \le 0.5\phi V_c = 12.5^k$

$$12.5^k = \frac{1.63\,(30.95)}{2} - 1.63\,(d)$$

$$d = 2.13 \text{ ft}$$

Number of stirrups:

$$2'' + (n-1)(9'') \ge 2.13(12)$$

$n = 3.62 \Rightarrow$ use 4 stirrups

Girder Design - Interior

Design Load

$w_n = 1.63$ klf

Beam above corridor.

$$w_n = \frac{[1.2(80) + 1.6(100)]}{1000} 7 + 1.2(0.25)$$

$$w_{n_{cor.}} = 2.134 \text{ klf}$$

$$P_u = \frac{(1.63)(30.95)}{2} + \frac{2.134(8.33)}{2} = 34.1^k$$

Design Moment

$$M_u = 0.5(34.1)(28) = 477.4 \times 1.1 = 525.14^{1k}$$

Estimate size.

$$bd^2 = 20M_u \Rightarrow b = 2/3 d \Rightarrow 2/3 d^3 = 20(525.14)$$

$$d = 25.1" \quad h = d + 2.5 = 27.6" \Rightarrow 28"$$

$$\hookrightarrow d = 25.5" \quad b = 17"$$

Compute Self weight Effect.

$$W_{sw} = \frac{17(28)}{144} \times 150 = 0.496 \text{ klf}$$

$$M_u = 525.14 + \frac{(0.496)(28)^2}{8} = 573.75^{1k}$$

Required Steel

$$A_s = \frac{M_u}{4d} = \frac{573.75}{4(25.5)} = 5.625 \text{ in}^2$$

use (4) #11 = 6.24 in² > 5.625 in² ✓

Check Flexural Strength

$a = \frac{(6.24)(60)}{0.85(6)(17)} = 4.32$ $c = \frac{4.32}{0.75} = 5.76$

$M_n = 6.24(60)(25.5 - \frac{5.76}{2})\frac{1}{12} = 705.7^{1k}$

$\varepsilon_s = \frac{0.003}{6.24}(25.5 - 6.24) = 0.0093 > 0.00207$ ✓

$\phi M_n = 0.9(705.7) = 635.13^{1k} > 573.75^{1k}$ ✓

Check Min. Reinf.

$A_{s,min} = \max \left| \begin{array}{l} \frac{3\sqrt{f'_c}}{f_y} b_w d = \frac{3\sqrt{6000}}{60{,}000}(17)(25.5) = 1.68 \ in^2 \\ \frac{200\, b_w d}{f_y} = \frac{200(17)(25.5)}{60{,}000} = 1.445 \ in^2 \end{array} \right.$

$A_s = 6.24 > 1.68 \ in^2$ ✓

Check Max. Reinf.

$A_{s\,max} = 0.85\beta_1 \frac{f'_c}{f_y}\left(\frac{0.003}{0.003+0.004}\right) b_w d$

$= 0.85(0.75)\frac{6}{60}\left(\frac{0.003}{0.007}\right)(17)(25.5)$

$= 11.84 \ in^2$

$A_s = 6.24 \ in^2 < 11.84 \ in^2$ ✓

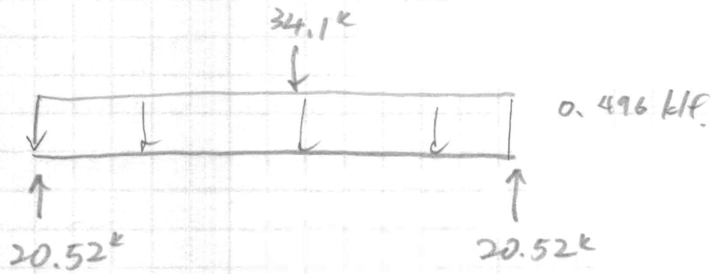
Check Min. Spacing.

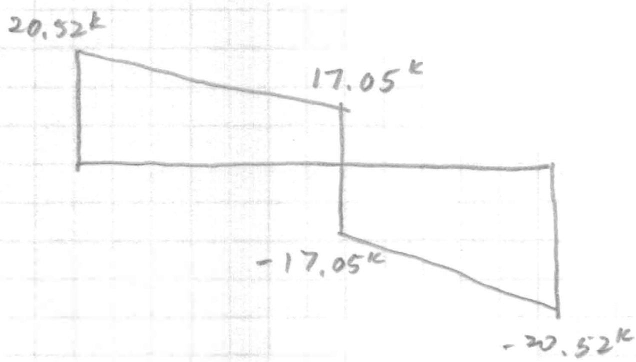
$$\max \left| \begin{array}{lcl} d_s & = & 1.41'' \\ 1'' & = & 1.0'' \\ 4/3'' \, S_a & = & 1.33'' \end{array} \right.$$

Actual Spacing.

$$S = \frac{17 - 1.5(2) - 0.5(2) - 1.56(4)}{3} = 2.25'' > 1.41'' \checkmark$$

Check Shear Strength.





$$V_u @ d = 20.52 - 0.496(25.5)\frac{1}{12} = 19.47^k$$

$$V_c = \frac{2\sqrt{6000}(17)(25.5)}{1000} = 67.16^k$$

$$0.5\phi V_c = 0.5(0.75)(67.16) = 25.185 > 19.47^k$$

$\therefore$ Stirrups not required

Girder Design - Exterior

Design Load

$w_u = 1.63$ klf

$P_u = \frac{(1.63)(30.95)}{2} = 25.2^k$

Design Moment.

$M_u = 0.5(25.2)(28) = 352.8^{\prime k} \times 1.1 = 388.1^{\prime k}$

Estimate size

$bd^2 = 20 M_u$

$\Rightarrow b = 2/3\, d = \quad 2/3\, d^3 = 20(388.1)$

$d = 22.7''$

$h = d + 2.5 = 25.2 \Rightarrow h = 26''$, $d = 23.5''$

$b = 16''$

Compute Self weight Effect

$W_{sw} = \frac{16(26)}{144} \times 150 = 433.3$ plf $= 0.4333$ klf

$M_u = 388.1 + \frac{(0.4333)(28)^2}{8} = 430.6^{\prime k}$

Required Steel.

$A_s = \frac{M_u}{4d} = \frac{430.6}{4(23.5)} = 4.58\ in^2 \Rightarrow$ use (4) #10 $= 5.08\ in^2$

$4.58 < 5.08\ in^2$

✓

Check Flexural strength

$$a = \frac{(5.08)(60)}{0.85(6)(16)} = 3.74 \qquad c = \frac{3.74}{0.75} = 4.98$$

$$M_n = 5.08(60)(23.5 - \frac{4.98}{2})\frac{1}{12} = 533.7^{\,'k}$$

$$\varepsilon_s = \frac{0.003}{5.08}(23.5 - 5.08) = 0.0126 > 0.00207 \checkmark$$

$$\phi M_n = 0.9(533.7) = 480.3^{\,'k} > 430.6^{\,'k} \checkmark$$

Check Minimum reinforcement.

$$A_{s,min} = \max \begin{cases} \frac{3\sqrt{f'_c}}{f_y} b_w d = \frac{3\sqrt{6000}}{60{,}000}(16)(23.5) = 1.46 \text{ in}^2 \\ \frac{200 b_w d}{f_y} = \frac{200(16)(23.5)}{60{,}000} = 1.25 \text{ in}^2 \end{cases}$$

$$A_s = 5.08 \text{ in}^2 > A_{s\,min} = 1.46 \text{ in}^2 \checkmark$$

Check Maximum reinforcement

$$A_{s\,max} = 0.85 \beta_1 \frac{f'_c}{f_y}\left(\frac{0.003}{0.003+0.004}\right) b_w d$$

$$= 0.85(0.75)\frac{6}{60}\left(\frac{0.003}{0.007}\right)(16)(23.5)$$

$$= 10.27 \text{ in}^2$$

$$A_s = 5.08 \text{ in}^2 < A_{s\,max} = 10.27 \text{ in}^2 \checkmark$$

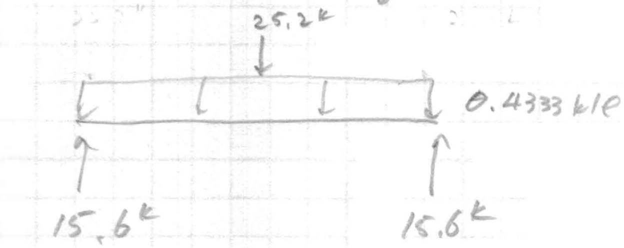
Check Min. Spacing

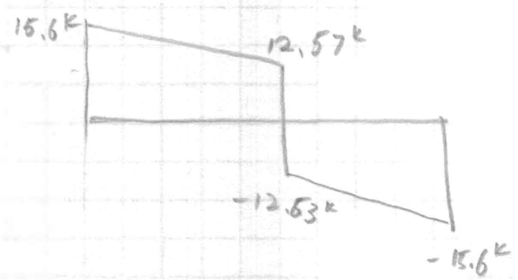
$$\max \begin{cases} d_s = 1.27'' \\ 1'' = 1'' \\ 4/3\, S_a = 1.33'' \end{cases}$$

Actual spacing.

$$S = \frac{18 - 1.5(2) - 0.5(2) - 1.27(4)}{3} = 2.31'' > 1.33'' \checkmark$$

Check Shear Strength.





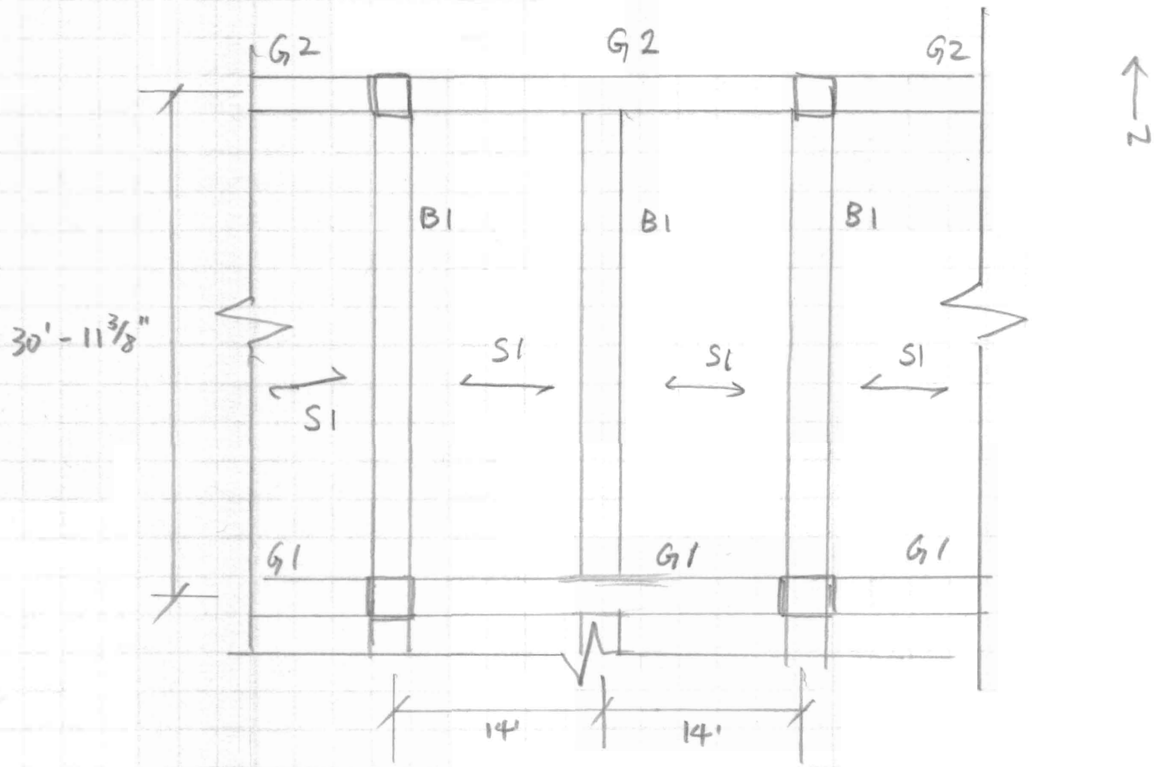
$$V_u \ @\ d = 15.6^k - 0.433(23.5)\frac{1}{12} = 14.75^k$$

$$V_c = \frac{2\sqrt{6000}\,(16)(23.5)}{1000} = 58.25^k$$

$$0.5\,\phi V_c = 0.5(0.75)58.25 = 21.8^k > 14.75^k$$

∴ Stirrup not required.

One-way concrete slab w/ Beams & Girder
Design Summary



S1: Slab
Thickness = 6"

E-W direction
Top Reinf: #4 @ 12"
Bottom Reinf: #4 @ 12"

N-S direction
#4 @ 12"
↳ For temperature and shrinkage.

B1: Beam
Size: 12" x 20"
NW Concrete
$f'_c = 6000$ psi
Longitudinal Reinf: (4) #8
Transverse Reinf.: (4) #3 stirrups @ 9"

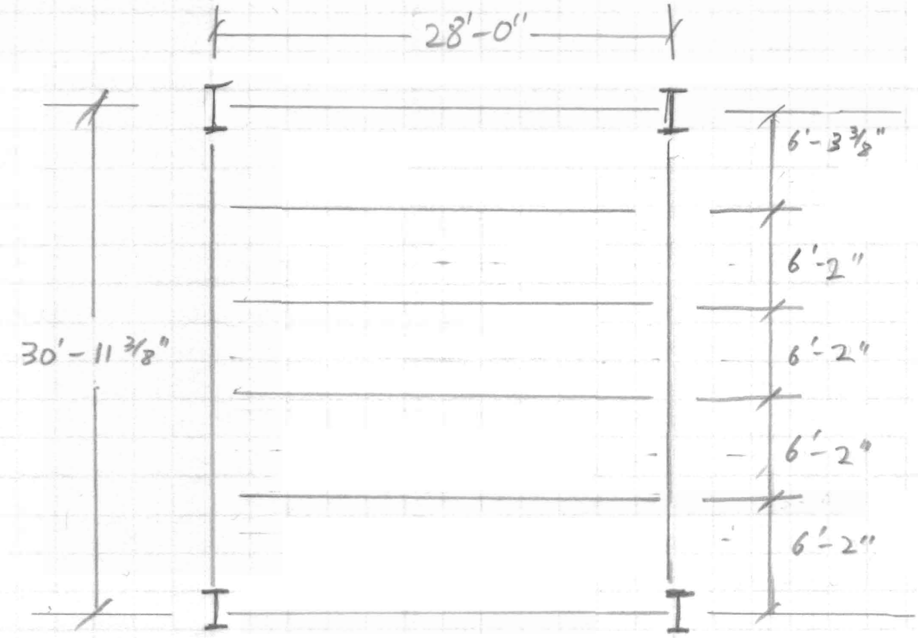
Girders
G1
Size: 17"x28"
NW Concrete
$f'_c = 6000$ psi
Longitudinal Reinf.: (4) #11
Transverse Reinf N/A

G2
Size: 16"x26"
NW Concrete
$f'_c = 6000$ psi
Longitudinal Reinf: (4) #10
Transverse Reinf: N/A.

Alternative Framing System for Gravity Load:

Alternative 2 – Non-composite Steel Framing

Alternate System #2: Non-Composite Steel



Determine Deck

1.5 VLR 22 w/ 3" topping Lightweight Concrete. 2hr rating.

3 span unshored clear span = 7'-9" > 6'-2"

Design Loadings

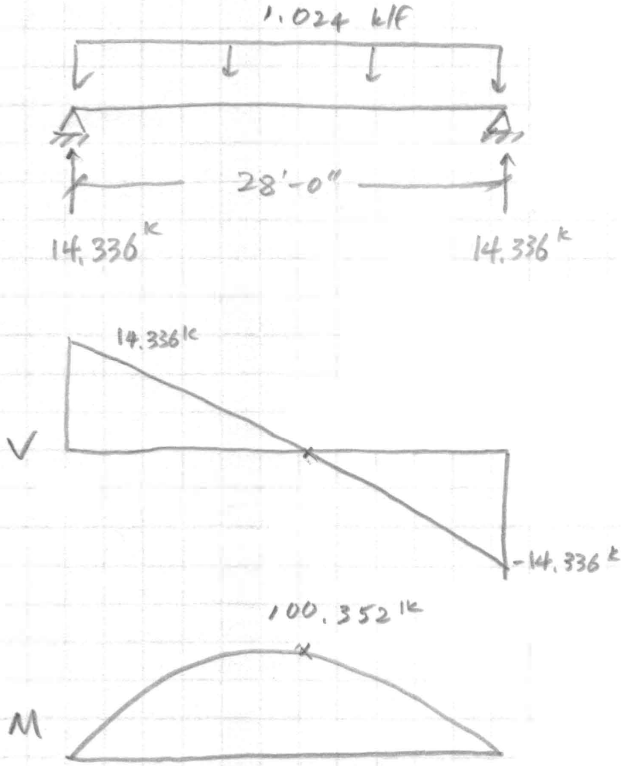
Live Load: Residential Hotel (Private) = 40 psf
partitions = 15 psf
55 psf

Dead Load:
Deck self weight = 50 psf
Superimposed/misc = 10 psf
Beam self weight = 5 psf
65 psf.

$$W_u = [1.2(65) + 1.6(55)]\, 6.17 = 1.024 \text{ klf}$$

Determine Beam Size

Shear & Moment Diagrams



Check Bending

From AISC Steel Manual Table 3-2,

	$\phi_b M_{px}$	
W12 x 22	110 'k	> 100.352 'k
W14 x 22	125 'k	

Check Shear

	$\phi_v V_{nx}$	
W12 x 22	95.9 k	> 14.336 k
W14 x 22	94.5 k	

Check Deflection.

$\Delta_{LL} \leq \frac{L}{360}$ $\quad W_{LL} = \frac{55 \times 6.17}{1000} = 0.339$ klf

W12x22 $\Delta_{LL} = \frac{5 w_{LL} L^4}{384 E I_x} = \frac{5(0.339)(28)^4(1728)}{384(29000)156} = 1.04''$

$\frac{L}{360} = \frac{28(12)}{360} = 0.933 < 1.04''$ N.G.

W14x22 $\Delta_{LL} = \frac{5(0.339)(28)^4(1728)}{384(29000)(199)} = 0.812'' < 0.933$ ✓

Check Beam Self weight Assumption.

$\frac{22 \text{ plf}}{6.17 \text{ ft}} = 3.57$ psf < 5 psf ✓

Check Shear & Moment

$DL = 50 + 10 + 3.57 = 63.57$ psf

$w_u = [1.2(63.57) + 1.6(55)]\, 6.17/1000 = 1.014$ klf

$V_u = 14.196^k$

$M_u = 99.372^{\prime k}$

From Table 3-2,

W 14 x 22 $\quad \phi_b M_{px} = 125^{\prime k} > 99.372^{\prime k}$ ✓

$\phi_v V_{nx} = 94.5^k > 14.196^k$ ✓

$\boxed{\text{Use W 14 x 22 beams}}$

★ Note: The second beam from the north may require bigger size beam due to larger tributary area.

Determine Girder

Design Load

Live load = 55 psf

Dead load =

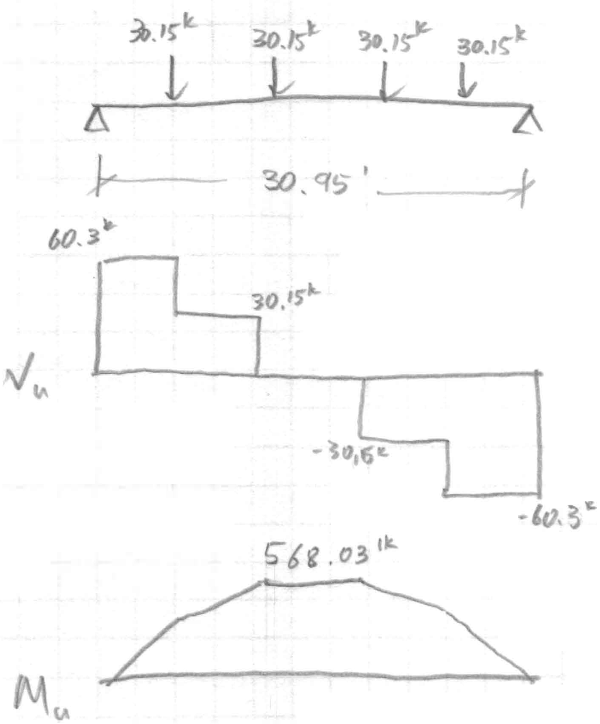
Superimposed/misc.	= 10 psf
Beam selfweight	= 3.57 psf
Deck	= 50 psf
Assumed Girder s.w.	= 6 psf
	= 69.57 psf

$W_u = \left[1.2(69.57) + 1.6(55)\right] = 171.48\ psf$

$P_u = 171.48\ psf(28)(6.28) = 30.15^k$

use larger tributary area for the top beam for conservative design.

Shear & Moment Diagram



From T. 3-2,

		$\phi_b M_{px}$	
Try	W21 x 68	600 1k	> 568.03 1k ✓
	W24 x 68	664 1k	

Shear		$\phi_v V_{nx}$	
	W21 x 68	272 k	> 60.3 k ✓
	W24 x 68	295 k	

Check Deflection

$P_{LL} = \frac{55(6.28)}{1000}(28) = 9.67^k$

$\Delta_{LL} \leq \frac{L}{360}$

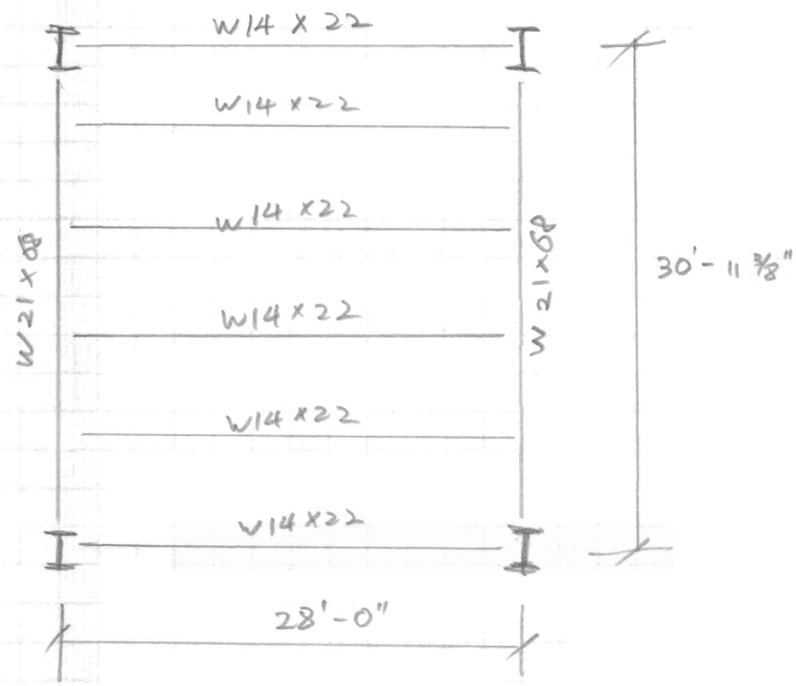
W21×68 $\Delta_{LL} = \frac{P_{LL} L^3}{28 E I_x} = \frac{(9.67^k)(30.95)^3(1728)}{28(29000)(1480)} = 0.412''$

$\frac{L}{360} = \frac{30.95}{360}(12) = 1.032'' > 0.412''$ ✓

Check Girder Self Weight Allowance

$\frac{68}{28} = 2.43$ psf < 6 psf ✓

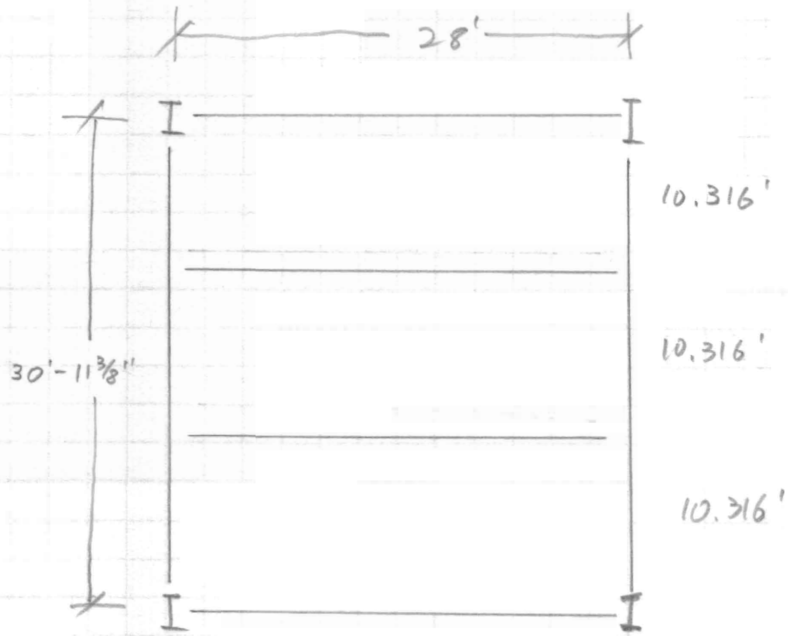
Use W21×68 for girder



Alternative Framing System for Gravity Load:

Alternative 3 – Composite Steel Framing

Alternate System #3 : Composite Steel.



LL = 55 psf

DL =

SDL = 10 psf

Beam S.W. = 5 psf

Deck = 42 psf

57 psf

Determine Deck.

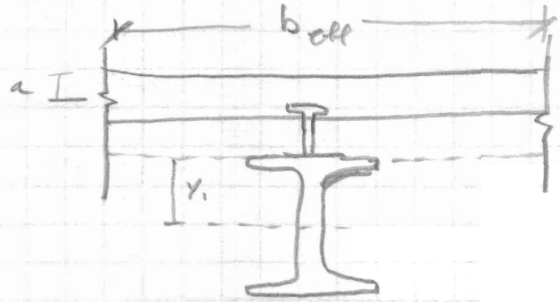
2 VLI 20 w/ 3.25" Light weight concrete

Max. unshored clear span = 10'-11" > 10.316' ✓

Allowable Superimposed Live load = 130 psf > 55 psf.

For composite design, assume 3/4" diameter shear studs.

Beam



$w_u = [1.2(57) + 1.6(55)]\ 10.316 = 1.613$ klf.

$b_{eff} = 2 \times \min \begin{cases} 10.316/2 \times 12 = 123.8'' \\ 28 \times 12/8 = 84'' \leftarrow \end{cases}$

$M_u = \dfrac{w_u l^2}{8} = \dfrac{1.613\ (28)^2}{8} = 158.07^{\text{'k}}$

$V_{cmax} = 0.85\ (3\ \text{ksi})(84'')(3.25'') = 696.15$ k.

Stud strength = 17.1 k/stud.

Let $a = 1.5''$ $Y_2 = 4.5''$

From Table 3-19 (AISC)

Try W16 x 36   Flex strength = $240^{\text{'k}} > 158.07^{\text{'k}}$ ✓

$\Sigma Q_n = 181^{k} \Rightarrow 164/17.1 = 9.6 \rightarrow$ 10 studs on each side.

$a_{rel} = \dfrac{\Sigma Q_n}{0.85\ f'_c\ b_{eff}} = \dfrac{181}{0.85(3)(84)} = 0.845'' < 1.5''$ ✓

Check unshored Strength

$\phi M_p = 240^{\text{'k}}$.

$w_u = 1.2(52)(10.316) + 1.2(36) + 1.6(55)(10.316) = 1.595$ klf

$M_u = \dfrac{(1.595)(28)^2}{8} = 156.31 < 158.07^{\text{'k}}$ ✓

Check wet concrete deflection.

$$\Delta_{wc} = \frac{L}{240} = \frac{28 \times 12}{240} = 1.4''$$

$$W_{wc} = 42(10) + 36 = 456 \text{ plf.}$$

$$\Delta_{wc} = \frac{5wL^4}{384EI_x} = \frac{5(.456)(28)^4(1728)}{384(29000)(448)} = 0.485'' < 1.4''$$ ✓

Check Live Load Deflection.

$$W_{LL} = 55 \times 10.316 = 0.567 \text{ klf.}$$

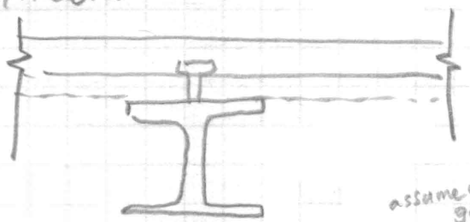
$$I = 906 \text{ in}^4$$

$$\Delta_{LL} = \frac{5(0.567)(28)^4(1728)}{384(29000)(906)} = 0.298'' < 0.933''$$ ✓

$$\Delta_{LL} = \frac{L}{360} = \frac{28 \times 12}{360} = 0.933''$$

$$\boxed{\text{Use W 16} \times \text{36 (20) for Beam}}$$

Girder.



assumed girder s.w.

$P_u = [1.2(42 + 10 + 6)(10.316) + 1.2(36) + 1.6(55)(10.316)]\,28$

$P_u = 46.7^k$

$b_{eff} = 2 \;\min\begin{cases} 28/2 \times 2 = 168'' \\ 30.95 \times 12/8 = 46.425'' \end{cases}$

$M_u = \frac{PL}{4} = \frac{46.7(30.95)}{4} = 361.3^{\prime k}$

$V_{cmax} = 0.85(3)(46.425)(3.25) = 384.7\ k.$

Stud strength $= 17.1^k$/stud.

Assume $a = 2''$, $Y_2 = 4''$

From AISC Table 3-19,

Try W16×40 Flex. Strength $= 409^{\prime k} > 361.3^{\prime k}$ ✓

$\Sigma Q_n = 192^k \Rightarrow \frac{192}{17.1} = 11.2$

↳ use (12) studs on each side

$a_{net} = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{192}{0.85(3)(46.425)} = 1.62'' < 2''$ ✓

Check unshored strength

$\phi M_p = 240^{\prime k}$

$P_u = [1.2(42+10)(10.316) + 1.2(40) + 1.6(15)(10.316)](28)$

$P_u = 26.3^k.$

$M_u = \frac{PL}{4} = \frac{26.3(30.95)}{4} = 203.5^{\prime k} < 240^{\prime k}$ ✓

Check wet concrete deflection

$$\Delta_{wc} = \frac{L}{240} = \frac{30.95 \times 12}{240} = 1.5'' \qquad w_{wc} = 40 \text{ plf}$$

$$P_{wc} = 40(28) + 42(10.316)(28) = 13.25^k$$

$$\Delta W_c = \frac{PL^3}{48EI_x} + \frac{5wl^4}{384EI_x} = \frac{13.25(30.95)^3(1728)}{48(29000)(518)} + \frac{5(40)(30.95)^4}{384(29000)(518)}$$

$$= 0.97'' < 1.5'' \quad \checkmark$$

Check Live Load Deflection

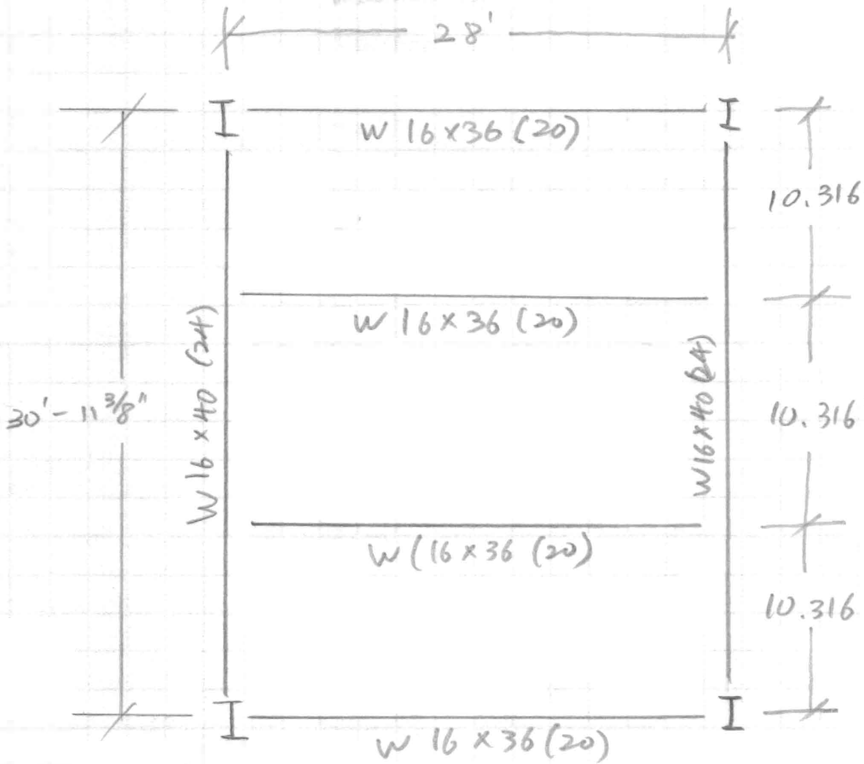
$$\Delta_{LL} = L/360 = \frac{30.95 \times 12}{360} = 1.032''$$

$$P_{LL} = 55(10.316)(30.95) = 17.6^k$$

$$I = 971$$

$$\Delta_{LL} = \frac{PL^3}{48EI} = \frac{17.6(30.95)^3(1728)}{48(29000)(971)} = 0.67'' < 1.032'' \quad \checkmark$$

$\boxed{\text{Use W16x40 (24) for Girder.}}$


28'
W 16x36 (20)
W 16x36 (20)
W 16x36 (20)
W 16x36 (20)
W 16 x 40 (24)
W16x40 (24)
30'-11 3/8"
10.316'
10.316'
10.316

Comparison and Comment

Through this technical assignment, four different framing system was incorporated for a same typical bay. With the previous experience with the precast plank manufacturer during last summer, that fact that most of upper level in the residential hotel floors were quite conservatively designed was taken into account when approaching the alternative framing systems. Because the company is a precast concrete manufacturer, production sequences are one of items that requires a close look. Designing members most optimally to their strength and capacity and to be used most efficiently may be beneficial to save materials or others. But the amount of material being saved while making different size members for an efficient design may not be that beneficial considering the difficulties and labor that needs to be spent for the extra work to manufacture different member sizes.

Because of this, the alternate systems may have come out to be more noticeably efficient than the original. The one way concrete slab was able to be reduced down to 6 inches from total of 13 inches of plank with composite topping. Although both non-composite and composite steel framing systems have higher floor depth due to member size but steel can make the whole structure lighter and more flexible than concrete.

Each system has its own benefits and disadvantages compared to others. It is the engineers who need to make the right call to make a most reasonable and yet intricate and long lasting structure.