Angela Mincemoyer
Structural Option
Advisor: Dr. Boothby
Peggy Ryan Williams Center
Ithaca, New York
18 October 2013

Peggy Ryan Williams Center

Technical Report 3

Angela Mincemoyer
Structural Option
October 18, 2013

Dr. Boothby
Advisor
Penn State University

Dear Dr. Boothby,

The following Technical Report 3 was prepared for AE 481W. Gravity spot checks were performed on the typical deck and one of each of the following: beam, girder, interior column, and exterior column. Following the spot checks, three alternative gravity framing systems were explored. The alternative designs consisted of non-composite steel, glulam, and a post-tensioned prestressed concrete slab with concrete beams. Possible lateral systems were also proposed for each of the alternative framing systems. The technical report concludes by comparing the designed alternative framing systems. Various hand calculations detail all of these calculations and designs. It is important to note that the following calculations are based on the adjusted gravity loads which may be seen on page 17.

Thank you in advance for taking the time to review the following report.

Sincerely,

Angela Mincemoyer

Enclosed: Technical Report 3

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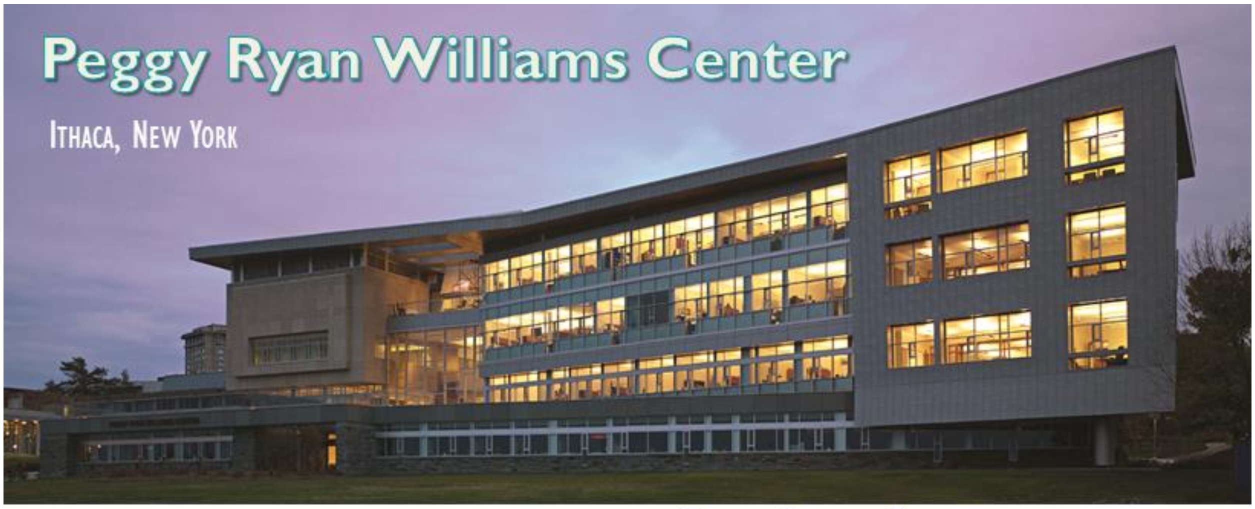


Primary Project Team:

Owner | Ithaca College
Architect | Holt Architects
Structural Engineer | Ryan-Biggs Associates
Mechanical & Electrical Engineer | Delta Engineers
General Contractor | Christa Construction

General Building Data:

Building Occupant | Ithaca College
Occupancy | Office Use
Size | 58,200 gross square feet
Stories | 4 stories above grade
Substantial Completion | March 2010
Cost of Construction | approx. $19.3 million
Project Delivery Method | Design-Bid-Build

Architecture:

- Various aspects were driven by desire to be eco-friendly
- Large areas of glass provide views of Cayuga Lake
- Façade consists of zinc panels, blue stone veneer, composite aluminum panels, and limestone panels
- Pedestrian bridge connects PRWC to adjacent building

Structure:

- *Foundation*
  - Slab-on-grade, foundation walls, footings, various grade beams, piers and drilled piers
- *Framing System*
  - All floors are composed of composite steel decking
  - Steel framing consists of wide flange beams, girders, and columns
- *Lateral System*
  - Concentrically braced structural steel frames in both the North-South and East-West directions

Sustainability:

- Awarded LEED Platinum
- "V" shaped roof aids in rain water collection
- Day lighting made possible by large areas of glass
- Intensive Green Roof
- Atrium promotes natural ventilation

MEP:

- *Mechanical*
  - Main heating and cooling source is geothermal via a closed loop system adjacent to the building
  - Two dedicated outdoor air units (DOA) will utilize water to water heat pumps
- *Electrical*
  - Primary Service: 12.5 KV primary fused switches, 500 KVA transformer, 480/277 Volt Distribution Switchboard
  - Secondary Distribution: 150 KVA, 480V to 120/208 Volt transformer and (1) 120/208 Volt Main power panel
- *Plumbing*
  - Collect and store rainwater for gray water use
  - (3) rainwater collections tanks

Angela Mincemoyer | Structural | http://www.engr.psu.edu/ae/thesis/portfolios/2014/ahm5066/index.html

Executive Summary

The Peggy Ryan Williams Center, formerly known as "The Gateway Building," is a four story office building located on the Ithaca College campus, Ithaca, New York. The building was originally known as "The Gateway Building" because the college saw the building as a gateway to the campus. At the time, the college was moving into a new era of sustainability and they wanted to show their prospective students, employees, and visitors the strides that they were making towards their goal.

Sustainability and a desire to connect with nature were both driving forces for the building's architectural features. The large areas of glass, offering vistas to Cayuga Lake, allow the occupants to feel like they are part of the nature around them. Other eco-friendly architectural features include the "V" shaped roof which aids in rainwater collection, and the large atrium which extends through the building to promote natural ventilation.

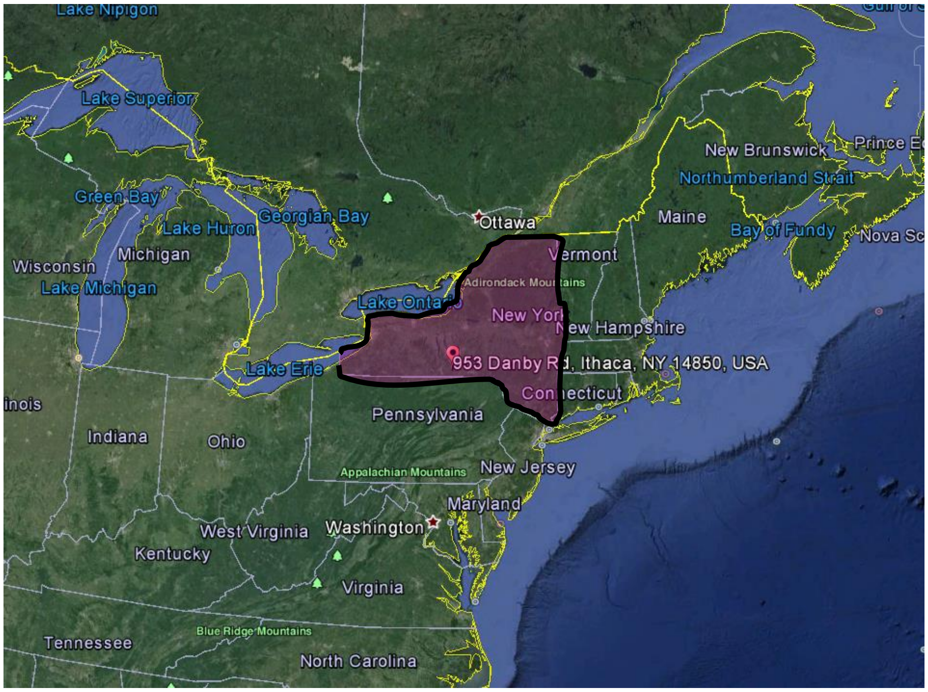
The structural system components are fairly common; however, their placement and size variations make the framing very irregular. The roof of the building is constructed of roof decking, which spans perpendicular to the beams, girders, and columns. The floor of Level 1 through Level 3 consists of composite decking and wide flanged beams, girders, and columns. Various beams and girders are provided with shear studs for composite action. Sizes and spans of the wide flanges vary greatly throughout the building and even throughout a single floor framing system. At locations where the building cantilevers, moment connections and larger beam/girder sizes make the cantilevers possible.

Columns, piers, and drilled piers support the foundation for the PRWC. The drilled piers range from resting on top of bedrock, to being drilled down 4'-0" below competent bedrock, depending on their location and loading.

Another distinctive feature of the Peggy Ryan Williams Center is the pedestrian bridge, which connects the building to the adjacent Dillingham Center. The bridge is a box truss supported in a double cantilever configuration with a 2" expansion joint on either end. I am eager to explore ways to improve the existing design for the bridge.

Due to its location, the PRWC was designed following the 2002 Building Code of New York State (BCNYS) which adopted the 2000 International Building Code (IBC). In addition to the BCNYS, additional loading and design requirements from American Society of Civil Engineering (ASCE) 7-98 are incorporated by reference into the IBC. In addition, various other codes were used in the design and are discussed in further detail in the following report.

Site Plan and Location Plan



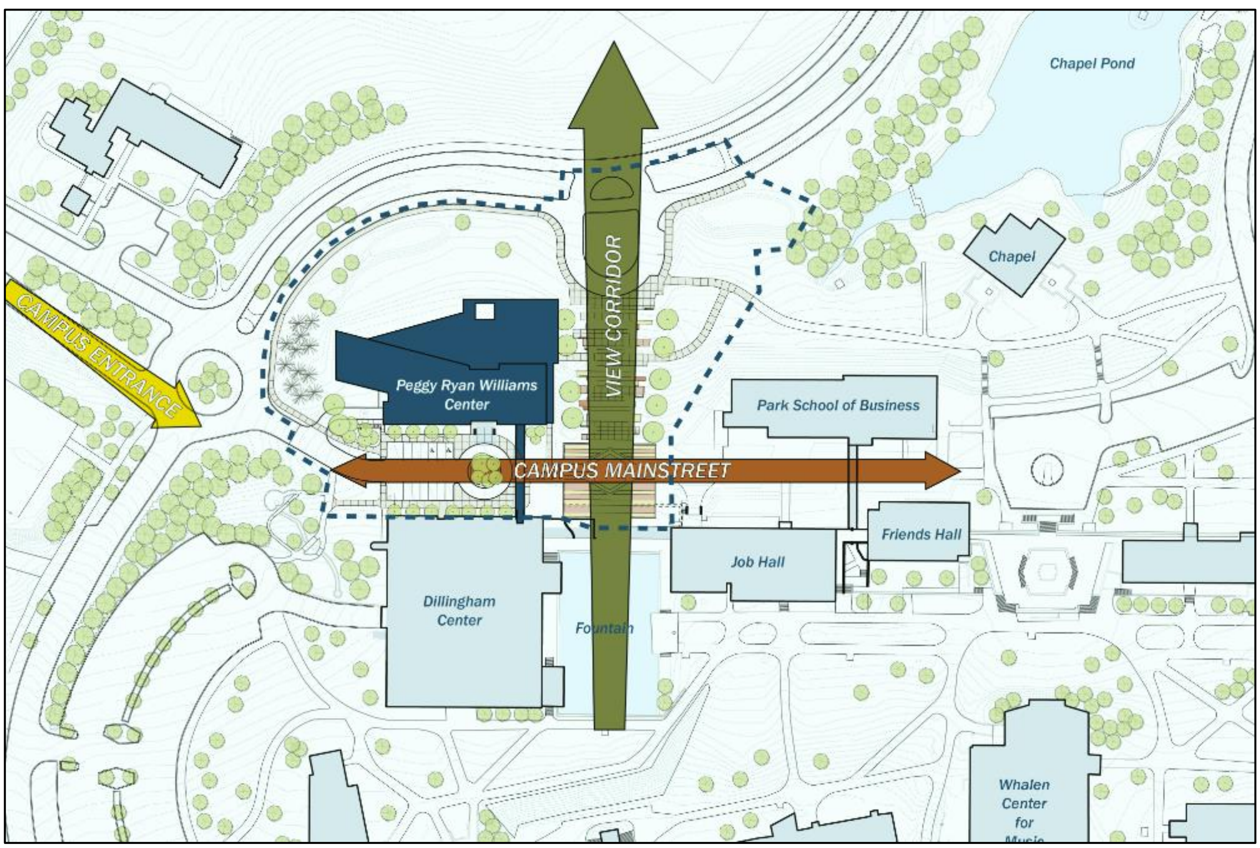


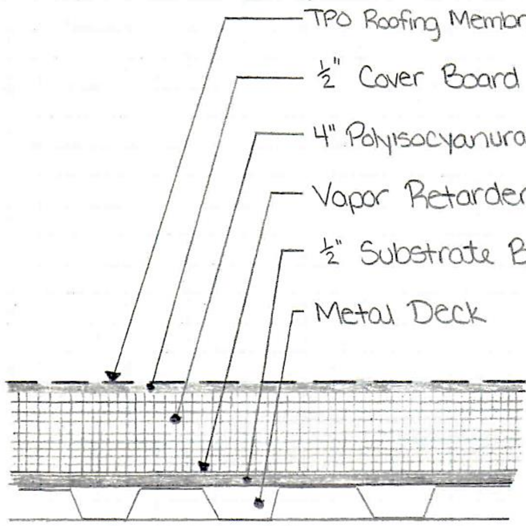
Photo provided courtesy of Holt Architects

Documents Used in Preparation of this Report

- Building Code of New York State
  - 2002 BCNYS (IBC 2000 adopted)
- International Building Code
  - IBC 2009
- American Society of Civil Engineers
  - ASCE 7-98: Minimum Design Loads for Buildings and Other Structures
- Vulcraft Deck Catalog
- American Concrete Institute
  - ACI 318-11
- American Institute of Steel Construction
  - AISC 14^{th} edition
- American Wood Council
  - National Design Specification (NDS): Design Values for Wood Construction
- Boise Cascade
  - Engineered Wood Products: Boise Glulam Beam and Column Specifier Guide
- Reed Construction Data
  - RS Means: Square Foot Cost 2013
  - RS Means: Facilities, Maintenance, and Repair 2013
- UC Berkley's Industrial Engineering and Operations Research Center

TYPICAL ROOF BAY LOADING

Sketch of detail H5 Membrane Roofing RS-1 page A001



DEAD LOADS:

TPO Roofing membrane, adhered = 2 psf

½" cover board = 2 psf

4" Polyisocyanurate insulation = 6 psf

Vapor retarder = 1 psf

½" substrate board = 2 psf

Metal deck = 2.2 psf

Misc. & Superimposed:

mechanical equipment & piping = 5 psf
sprinklers = 10 psf
lighting = 5 psf
suspended ceiling = 3 psf
framing allowance = 10 psf

→ Total roof dead load = 48.2 psf

(25 psf roof dead load was used in design)

Angela Mincemoyer	Gravity Loads	Tech Report 2	8/43

LIVE LOADS:

per ASCE7-98 section 4.9 Minimum Roof Live Loads required the use of equation 4-2 ($L_r = 20R_1R_2$, where $12 \le L_r \le 20$).
The variable R_1 & R_2 rely on tributary areas. Because I am looking for a typical psf (I do not have a tributary area) I will be conservative and set $L_r = 20$ psf.

$\boxed{\text{(ASCE7-98) roof live load} = 20\text{ psf}}$

- No design roof live load was provided
  → roof snow load most likey controlled

SNOW LOADS:

uniform ground snow load (p_g) = 45 psf

$p_f = 0.7\, C_e C_t I p_g$

min $p_f = 20 \cdot I$

Exposure Factor (C_e) = 1.0
- partially exposed
- exposure B

Thermal Factor (C_t) = 1.0

Importance Factor (I) = 1.1
- category III

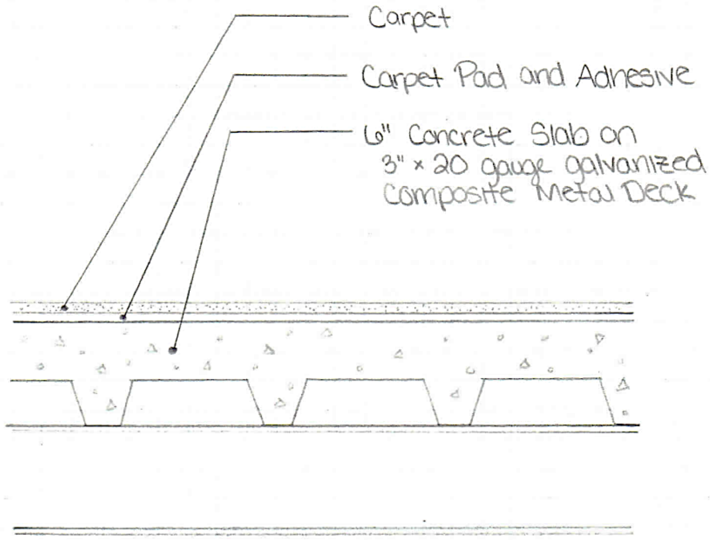
→ $p_f = 0.7(1.0)(1.0)(1.1)(45) = 34.65$ psf

check min p_f:
min $p_f = 20 \cdot I = 20(1.1) = 22$ psf < 34.65 psf ✓

→ $\boxed{p_f = 35\text{ psf}}$

→ design uniform flat-roof snow load = 35 psf
→ the design matches the code minimum

TYPICAL FLOOR BAY LOADING:



DEAD LOADS:

Carpet = 1 psf

carpet pad & adhesive = 1.5 psf

6" concrete slab on 3" × 20 gauge galvanized composite metal Deck = 57 psf

Misc & Superimposed:

mechanical equipment & piping = 5 psf
sprinklers = 10 psf
lighting = 5 psf
suspended ceiling = 3 psf
framing allowance = 10 psf

→ Total floor dead load = 92.5 psf

(for interior floors, a 80 psf dead load was used in design)

Angela Mincemoyer | Gravity Loads | Tech Report 2 | 10/43

LIVE LOADS:

$\boxed{\text{Live load} = 80\ \text{psf}}$

(corridors above first floor used for flexibility)

→ design interior floor live load = 80 psf

→ the design matches the code minimum

NON-TYPICAL LOADS:

Green Roof: (detail H7 - roof type RS-2)

DEAD LOADS: vegetation & planting medium = 70 psf
water retention composite = 2 psf
moisture mat = 2 psf
4" extruded polystyrene insulation = 1 psf
drainage composite = 2 psf
root barrier = 2 psf
hot fluid applied roofing = 2 psf
6" concrete slab on 3" × 20 ga galvanized composite metal deck = 57 psf (pg. 54 of Vulcraft catalog)
misc & superimposed:
same as typical roof bay = 33 psf

$\boxed{\text{Total Roof Dead Load = 171 psf}}$

(Design dead load = 120 psf)

LIVE LOADS: $\boxed{\text{Live Load = 100 psf}}$ (ASCE 7-98)

(Design live load = 100 psf)

Roof System RS-3 : used on first floor roof (deck area)

DEAD LOADS:

2" bluestone paver = 26 psf
→per Bluestone Guide from Braen Supply Inc)

pedestal system = 3 psf

4" extruded polystyrene insulation = 1 psf

drainage composite = 2 psf

hot liquid applied roofing = 2 psf

6" concrete slab on 3" × 20 ga = 57 psf (pg.54 of Vulcraft catalog)
galvanized composite metal deck

misc & superimposed:

same as typical roof bay = 33 psf

$\boxed{\text{Total Roof Dead Load} = 124 \text{ psf}}$

(Design dead load = 120 psf)

LIVE LOADS:

$\boxed{\text{Live Load} = 100 \text{ psf}}$ (ASCE 7-98)

(Design live load = 100 psf)

Misc. Floors:

first floor:

DEAD LOAD: 7" concrete slab on 3" × 20 ga. galvanized composite metal deck = 69 psf (pg. 54 of Vulcraft catalog)

* this composite deck weighs 12 psf more than the typical floor bay loading.

→ For areas of 7" concrete slab on 3" × 20 ga. galvanized composite metal deck, an addition 12 psf should be added to the typical floor bay loading.
→ This will result in a total dead load = 104.5 psf

(Design dead load = 80 psf)

Interior floor with Bluestone:

DEAD LOAD: 2" bluestone paver = 26 psf
→ per Bluestone Guide from Braen Supply, Inc)

pedestal system = 3 psf

in addition to typical floor bay loading = 90 psf
(carpet, carpet pad & adhesive are not included in this load)

Total floor dead load = 119 psf

(design dead load = 120 psf)

LIVE LOAD: In areas where bluestone flooring is present, an additional 20 psf live live should be added to the typical floor bay live load. Resulting in a total floor live load of 100 psf in those areas.

The additional 20 psf is added to account for any repairs that may be required in the future, such as broken/cracked sections needing to be replaced.

(design live load = 100 psf)

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Mechanical Room:

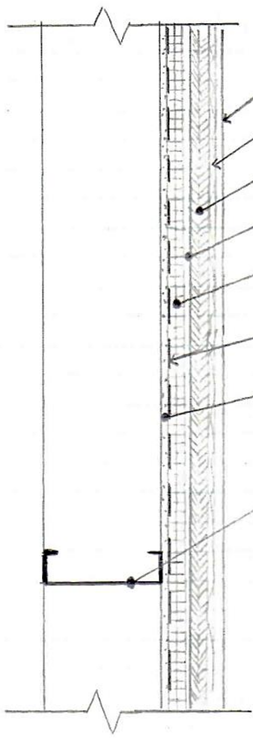
Live load = 150 psf

(industry standard)

Stairs:

Live load = 100 psf (ASCE7-98)

TYPICAL EXTERIOR WALL DETAIL: Zinc Panels (EW-4)



DEAD LOADS:

Flat Lock Zinc Panel = 2 psf
Separation sheet = 1 psf
3/4" Plywood Sheathing = 2.4 psf (ASCE 7-10)
3/4" Extruded Polystyrene Insulation = 0.5 psf
Weather Barrier = 1 psf
1/2" gypsum sheathing = 2 psf (ASCE 7-10)
6" metal studs @ 16" o.c. = 4 psf
Spray foam insulation = 1 psf

$\boxed{\text{Total dead load} = 13.9 \text{ psf}}$

Angela Mincemoyer	Gravity Load	Tech Report 2	16/43

LOAD PATH DESCRIPTION:

The exterior wall facade load is carried by a grid of 6" metal studs @ 16" O.C. The load is first transferred to the horizontal 6" metal studs and then into the vertical 6" metal studs. The metal studs then transfer the load into the foundation.

Angela Mincemoyer | Adjusted Gravity Load | Tech Report 2

Misc. & Superimposed:

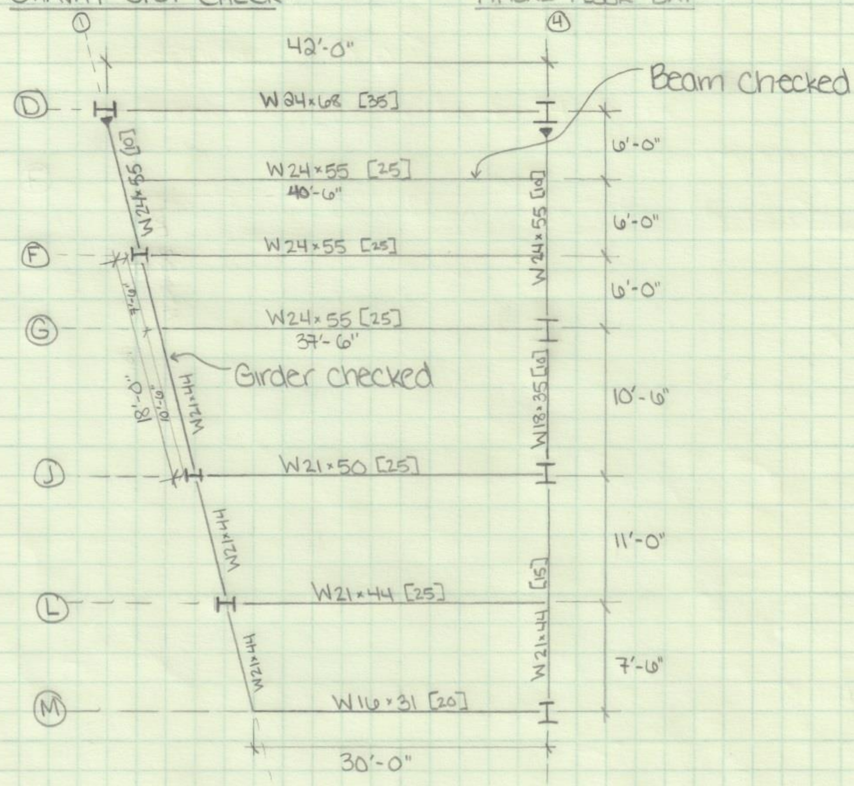
sprinklers = 5 psf

→ Total roof dead load = 43.2 psf (typical)

→ Total floor dead load = 87.5 psf (typical)

Angela Mincemoyer | Gravity Spot Check | Tech Report 3

GRAVITY SPOT CHECK: TYPICAL FLOOR BAY:



Note: This region of the framing was chosen because this layout is used on both level 2 and level 3 of the building.
It is important to note that the cantilever (which would be at the top of the page) has been left out of the "typical region" because most areas of the building are not cantilevered.

A. Mincemoyer | Gravity Spot Check | Tech Report 3

Check Deck: 6" concrete slab on 3" x 20 ga galvanized composite metal deck

per Vulcraft Catalog → 3VLI20

Check if shoring is required:
(structural notes do not contain any mention of shoring)

worst case 3 span condition:
10'-6", 11'-0", 7'-6"

SDI Max Unshored Clear Span - 3 Span Condition

12'-3" > 11'-0" ✓

SDI Max Unshored Clear Span - 2 Span Condition

12'-4" > 11'-0" ✓

SDI Max Unshored Clear Span - 1 span condition

10'-1" < 11'-0" X

→ Deck may be unshored for 2 span and 3 span conditions.
If single span condition occurs, deck would need to be shored.

Check Strength:

Live Load = 80 psf

Superimposed Dead Load = 18 psf

mechanical equipment & piping = 5 psf
sprinklers = 5 psf
lighting = 5 psf
suspended ceiling = 3 psf

$W_{LL} + W_{misc\ DL} = 80 + 18 = 98$ psf

Clear Span = 11'-0"
Superimposed Live Load (Vulcraft Catalog) = 119 psf

119 psf > 98 psf ✓

→ Deck has sufficient strength to carry the loads ✓

⇒ Deck is adequate ✓

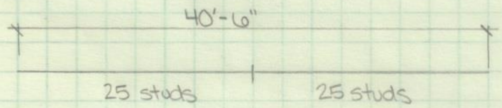
A. Mincemoyer | Gravity Spot Check | Tech Report 3

Check Beam: $W24\times55$ [25] span: 40'-6"

$w_{DL} = 87.5$ psf
$w_{LL} = 80$ psf

$w_u = 1.2w_{DL} + 1.6w_{LL} = 1.2(87.5) + 1.6(80)$
$\rightarrow w_u = 233$ psf $(6\text{ ft})(1/1000) = 1.40$ klf

$M_u = \frac{wl^2}{8} = \frac{(1.40)(40.5)^2}{8} \rightarrow \underline{M_u = 287.0 \text{ k}\cdot\text{ft}}$



~ 25 studs in 20'

5 ribs w/ 2 studs
15 ribs w/ 1 stud

deck perpendicular to beam
weak studs (conservative)
3/4" Φ studs (per S001)
$f'c = 3{,}500$ psi (per S001)

per Table 3-21:

$\Sigma Q_n = 5(14.6) + 15(17.2) \rightarrow \underline{\Sigma Q_n = 331\text{ k}}$
$\uparrow$ 2 stud/rib $\uparrow$ 1 stud/rib

$T_{s,max}$:

$T_{s,max} = A_s F_y$ $F_y = 50$ ksi (A992)
$A_s = 16.2\text{ in}^2$

$T_{s,max} = 16.2(50) \rightarrow \underline{T_{s,max} = 810^k}$

$C_{c,max}$:

$C_{c,max} = 0.85 f'c \cdot b_{eff} \cdot t$

$b_{eff} = 2 \times \min \begin{cases} \text{span}/8 = 40.5/8 = 5.06' \\ \tfrac{1}{2}\text{ dist. adj. beam} = \tfrac{1}{2}(6) = 3' \leftarrow \text{controls} \end{cases}$

$b_{eff} = 2\times3 \rightarrow b_{eff} = 6' = 72"$
$t = 3"$
$f'c = 3{,}500$ psi

$C_{c,max} = 0.85(3500)(72)(3)/1000 \rightarrow \underline{C_{c,max} = 642.6^k}$

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$\Sigma Q_n = 331^k < C_{cmax} = 642.6^k$
AND
$\Sigma Q_n = 331^k < T_{s\,max} = 810^k$
} partially composite

$a = \frac{\Sigma Q_n}{0.85 f'_c \cdot b_{eff}} = \frac{331}{0.85(3.5)(72)} \rightarrow a = 1.55$

$Y_2 = t - a/2 = 6 - 1.55/2 \rightarrow Y_2 = 5.23''$

Table 3-19: W24×55 $\Sigma Q_n = 331^k$ $Y_2 = 5.23''$

ΣQ_n	5	5.23	5.5
456	881	888	898
331		822	
329	816	821	828

(Y_2 above the 5 / 5.23 / 5.5 columns)

$\phi M_n = 822$ k·ft

822 k·ft $= \phi M_n > M_u = 287$ k·ft ✓
→ Beam strength is met

Check Unshored Strength:

$W_u = 1.4(57\text{ psf})(6\text{ ft}) + 1.4(55) \rightarrow W_u = 555.8$ plf

$W_u = 1.2[(57)(6) + 55] + 1.6[(20)(6)] \rightarrow W_u = 668.4$ plf ← controls

→ $W_u = 0.669$ klf

$M_u = \frac{wl^2}{8} = \frac{(.669)(40.5)^2}{8} \rightarrow M_u = 137.2$ k·ft

W24×55 → $\phi M_p = 503$ k·ft (Table 3-19)

$137.2'^k = M_u < \phi M_p = 503'^k$
→ No shoring is required for strength

A. Mincemoyer | Gravity Spot Check | Tech Report 3

Check wet concrete deflections:

$W_{wc} = 57(6) + 55 = 0.397$ klf

$\Delta_{wc} = \frac{5wl^4}{384EI} = \frac{5(0.397)(40.5)^4(1728)}{384(29000)(1350)} \rightarrow \Delta_{wc} = 0.614"$

$\Delta_{wc\,max} = \frac{l}{240} = \frac{40.5(12)}{240} \rightarrow \Delta_{wc\,max} = 2.025"$

$0.614" = \Delta_{wc} < \Delta_{wc\,max} = 2.025"$ ✓

→ wet concrete deflection is okay (no camber needed)

Check live load deflection:

$W_{LL} = 80(6) \rightarrow W_{LL} = 0.480$ klf

per Table 3-20 → $I_{LB} = 2670$ in^4 (conservative)

$\Delta_{LL} = \frac{5wl^4}{384EI} = \frac{5(.480)(40.5)^4(1728)}{384(29000)(2670)} \rightarrow \Delta_{LL} = 0.375"$

$\Delta_{LL\,max} = \frac{L}{360} = \frac{40.5(12)}{360} \rightarrow \Delta_{LL\,max} = 1.35"$

$0.375" = \Delta_{LL} < \Delta_{LL\,max} = 1.35"$ ✓

→ live load deflections are okay

⇒ Beam is adequate ✓

A. Mincemoyer | Gravity Spot Check | Tech Report 3

Check Girder: W21x44 (exterior)

* This girder was chosen because it is not affected by the cantilever (it is not a backspan for the cantilever). This girder is also common in this region of the building.

Determine point load on girder due to beam:

$W_{DL} = 87.5$ psf
$W_{LL} = 80$ psf

$W_u = 1.2 W_{DL} + 1.6 W_{LL} = 1.2(87.5) + 1.6(80)$
$\rightarrow W_u = 233 \text{ psf } (7.5/2 + 10.5/2) / 1000 = 2.10 \text{ klf} + \overbrace{1.2(.055)}^{\text{beam weight}}$
$\rightarrow W_u = 2.17$ klf

$P = \frac{2.17(37.5')}{2} \rightarrow P = 40.69^k \rightsquigarrow P \approx 41^k$

per Table 3-23:

Girder - $M_u = \frac{Pab}{l} = \frac{(41)(10.5)(7.5)}{18'}$

$\rightarrow M_u = 179.4'^k$

Exterior Wall load:

$W_D = (13.9 \text{ psf})(13.33')(1.2) \rightarrow W_D = 0.222$ klf

$M_u = \frac{wl^2}{8} = \frac{(.222)(18)^2}{8} \rightarrow M_u = 9.0'^k$

$M_{u,total} = 179.4 + 9.0 = 188.4'^k$

Per table 3-2:

W21x44: $\phi M_n = 358'^k$

$188.4'^k = M_u < \phi M_n = 358'^k$ ✓
→ Girder strength is met

A. Mincemoyer | Gravity Spot Check | Tech Report 3

Check Deflections:

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

Δ_{LL}:

beam: $W_{LL} = 80\text{ psf}\left(\frac{7.5}{2} + \frac{10.5}{2}\right)/1000 \rightarrow W_{LL} = 0.72\text{ klf}$

point load on girder:

$$P = \frac{0.72(37.5)}{2} \rightarrow P = 13.5^k \rightsquigarrow P \approx 14^k$$

per Table 3-23:

Girder: $$\Delta_{max} = \frac{Pab(a+2b)\sqrt{3a(a+2b)}}{27EI\ell}$$

$$= \frac{(14)(10.5)(7.5)(10.5+2(7.5))\sqrt{3(10.5)(10.5+2(7.5))}\,(1728)}{27(29000)(843)(18)}$$

$$\rightarrow \Delta_{max} = 0.116"$$

$$L/360 = \frac{18(12)}{360} \rightarrow L/360 = 0.6"$$

$$\boxed{0.116" = \Delta_{max} < L/360 = 0.6" \checkmark}$$

→ Girder deflections are met

⇒ Girder is adequate

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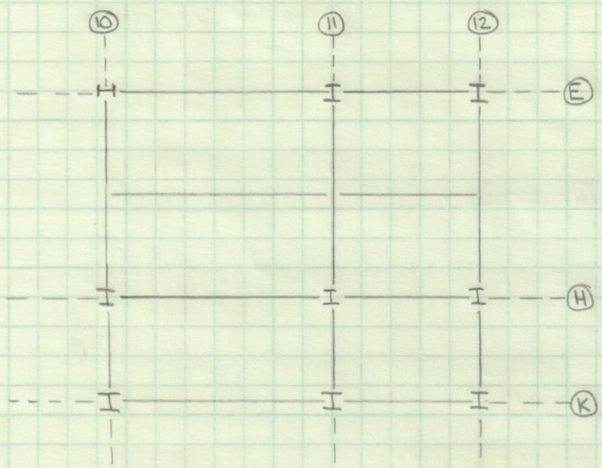
Interior Column: Column H11 (from Garden Level to Roof)



Sketch of Level 2 Framing Plan

Tributary Areas:

Level 1: $(18/2 + 11.25/2)(9/2 + 16.5/2) = 186.5\ ft^2$

Level 2: $(18/2 + 11.25/2)(9/2 + 16.5/2) = 186.5\ ft^2$

Level 3: $(18/2 + 11.25/2)(9/2 + 3) = 109.7\ ft^2$

Roof: $(18/2 + 11.25/2)(9/2 + 37.5/2) = 340.0\ ft^2$

Loading:

Level	Dead Load (psf)	Live Load (psf)
1	87.5	80
2	87.5	80
3	87.5	80
Roof	43.2	S = 35 psf (L_R = 20 psf)

Worst load combination:

$1.2D + 1.6L + 0.5S$

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Check Live Load Reduction:

Is influence area ≥ 400 ft² ?

Level 1: $(29.25')(25.5') = 746\ ft^2$

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

$= 80\left(0.25 + \frac{15}{\sqrt{746}}\right)$ → $L = 64\ psf$

$L \geq 0.40 L_o = 0.40(80)$

$L = 64 > 32$ ✓

Level 2: same as Level 1 → $L = 64\ psf$

Level 3: $(29.25')(12') = 351\ ft^2 < 400\ ft^2$

→ can't reduce live load → $L = 80\ psf$

$P_u = 1.2D + 1.6L + 0.5S$

$= 1.2[(87.5)(186.5) + (87.5)(186.5) + (87.5)(109.7) + (43.2)(340.0)]$

$+ 1.6[(80)(186.5) + (80)(186.5) + (80)(109.7)]$

$+ 0.5[(35)(340)]$

→ $P_u = 136.1^k$

* assuming column is C2 (W10×49) per S100

per Table 4-1:

$KL = 13.33'$

KL	13	492
	13.33	485^k
	14	471

$\phi P_n = 485^k$

$\boxed{136.1^k = P_u < \phi P_n = 485^k \checkmark}$
→ Column H11 is adequate

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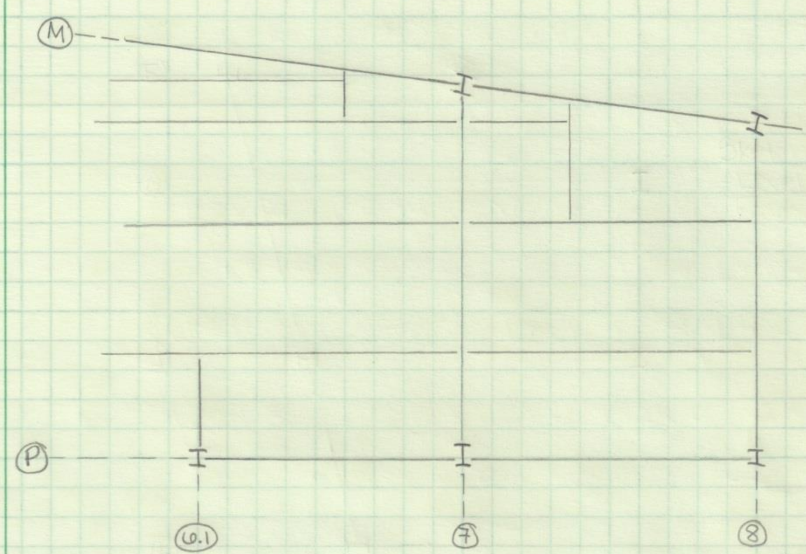
Exterior Column: Column P7 (from Level 1 to Roof)



Tributary Areas:

Level 1,2,3, & roof: $(20.25/2 + 27/2)(27/2) = 318.9 \text{ ft}^2$

Loading:

Level	Dead Load (psf)	Live Load (psf)
2	87.5	80
3	87.5	80
Roof	166	$L_R = 100$ (S = 78 psf)

exterior wall load:

$P_D = (13.9 \text{ psf})(13.33)(20.25/2 + 27/2) = 4.4^k$

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* Due to irregular framing layout, live load reduction will not be used (this is conservative).

$P_D = 4.4^k + [87.5(318.9) + 87.5(318.9) + 166(318.9)]/1000$
$\rightarrow P_D = 114^k$

$P_L = [80(318.9) + 80(318.9)]/1000$
$\rightarrow P_L = 52^k$

$P_{LR} = [100(318.9)]/1000$
$\rightarrow P_{LR} = 32^k$

worst load combination:

$1.2D + 1.6L + 0.5L_R$

$P_u = 1.2(114) + 1.6(52) + 0.5(32)$
$\rightarrow P_u = 236^k$

Column C3 (W10×39) per S101

per Table 4-1:

$KL = 13.33'$

KL	
13	329
13.33	321^k
14	306

$\phi P_n = 321^k$

$\boxed{236^k = P_u < \phi P_n = 321^k \checkmark \rightarrow \text{Column P7 is adequate}}$

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- The region that I will use for my typical bay includes a cantilever on the north side. However, for my alternate gravity framing system, I will "remove" the cantilever and simply end the framing at column line D. By removing the cantilever, the region will be more typical of the rest of the building.
During my redesigns, I will briefly explore how to design for the cantilever with the alternate systems.

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NON-COMPOSITE-STEEL:
Determine which beam to redesign: Find $M_{u,max}$

W24x55 - west end of building - between D & F
(same as beam checked for gravity spot check)

From gravity spot check: $M_u = 287.0$ k·ft

W21x50 - west end of building - along J
(see pg. 45 for sketch)

$w_D = 87.5$ psf (this includes a 10 psf framing allowance)
$w_L = 80$ psf

$w_u = 1.2 w_{DL} + 1.6 w_{LL} = 1.2(87.5) + 1.6(80)$
$w_u = 233 \text{ psf} \left(\frac{11.25}{2} + \frac{10.5}{2}\right)\left(\frac{1}{1000}\right)$

$\rightarrow w_u = 2.54$ klf

$M_u = \frac{wl^2}{8} = \frac{(2.54)(34.5)^2}{8} \rightarrow M_u = 378$ k·ft

→ The W21x50 along column line J is the worst case. For the beam design, a maximum moment of 378 k·ft and a span of 34'-6" will be used.

Note:
→ The beam/girder layout from the original design will be used in the redesign.
→ Existing deck (3VLI20) will be used in this redesign.

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Strength:
Using table 3-2: and choosing an economic section

→ W21×48

$\phi M_n = 398'^k > 378'^k = M_u$ ✓

Deflections:

$\Delta_{max} = \frac{5wl^4}{384EI}$

$W = 80\,psf\,(11.25/2 + 10.5/2)(1/1000) \rightarrow W = 0.87\,klf$

$l = 34.5'$
$E = 29000\,ksi$
$I = 959\,in^4$

$\Delta_{max} = \frac{5(.87)(34.5)^4(1728)}{384(29000)(959)} = 0.997''$

$\frac{L}{360} = \frac{34.5(12)}{360} = 1.15'' > \Delta_{max}$ ✓

Longest span:

$\Delta_{max} = \frac{5wl^4}{384EI}$

$W = 80\,psf\,(6)(1/1000) \rightarrow w = 0.48\,klf$

$l = 40.5'$
$E = 29000\,ksi$
$I = 959\,in^4$

$\Delta_{max} = \frac{5(.48)(40.5)^4(1728)}{384(29000)(959)} = 1.05''$

$\frac{L}{360} = \frac{40.5(12)}{360} = 1.35'' > \Delta_{max}$ ✓

Check Self Weight:

$\frac{48\,plf}{6\,ft} = 8\,psf < 10\,psf$ allowance ✓

min spacing

→ $\boxed{\text{Typical Beam: W21×48}}$

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Girder:

* Start by keeping current girder locations & spans so that columns do not need to be changed.

Interior Girder:

The longest interior girder span is 19'-6".

Determine point load on girder due to beams:

$W_{DL} = 87.5 - 10 = 77.5$ psf (without a framing allowance)

$P_{DL} = [(77.5 \text{ psf})(6 \text{ ft}) + (48 \text{ plf})] \times 40.5/2$

$\rightarrow P_{DL,1} = 10.4^k$

$P_{LL,1} = (80 \text{ psf})(6)(40.5/2)$

$\rightarrow P_{LL1} = 9.72^k$

$P_{u,1} = 1.2 P_{DL,1} + 1.6 P_{LL,1} = 1.2(10.4) + 1.6(9.72)$

$\rightarrow \underline{P_{u1} = 28.1^k}$

$P_{DL2} = [(77.5 \text{ psf})(9.75') + 48 \text{ plf}] \times 33/2$

$\rightarrow P_{DL2} = 13.3^k$

$P_{LL2} = (80 \text{ psf})(9.75')(33/2)$

$\rightarrow P_{LL2} = 12.9^k$

$P_{u2} = 1.2(13.3) + 1.6(12.9)$

$\rightarrow \underline{P_{u2} = 36.6^k}$

Self weight allowance:

$W_u = (60 \text{ plf}) 1.2 = 72 \text{ plf} = 0.072$ klf

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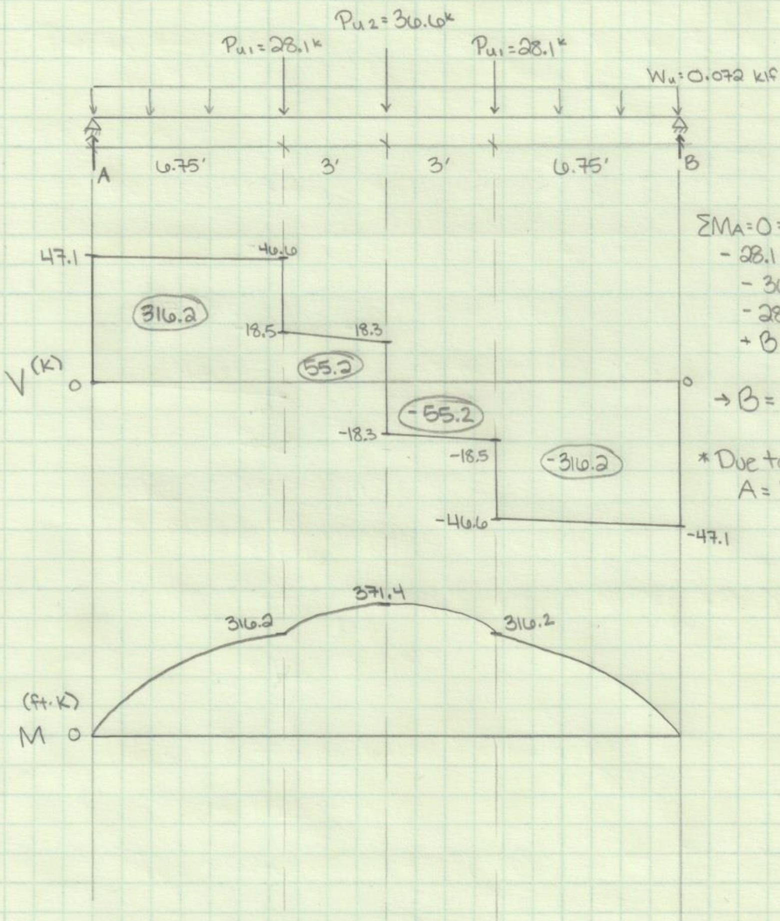

$\Sigma M_A = 0 = -(.072)(19.5)(19.5/2)$
$- 28.1(6.75)$
$- 36.6(9.75)$
$- 28.1(12.75)$
$+ B(19.5)$

$\rightarrow B = 47.1^k$

* Due to symmetry
$A = 47.1^k$

$\rightarrow M_u = 371.4$ ft·K

Strength:

using Table 3-2 : and choosing an economic section

$\rightarrow$ W 21×48

$\phi M_n = 398'^k > 372'^k = M_u$ ✓

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Deflections :

using RISA: $\Delta_{max} = -0.287''$

$\frac{L}{360} = \frac{19.5(12)}{360} = 0.65'' > \Delta_{max}$ ✓

Check Self-weight:

48 plf < 60 plf allowance ✓

→ $\boxed{\text{Typical Interior Girder W21×48}}$

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Exterior Girder:

-worst exterior girder span is between column lines F & J.

span: 18'-0"

Determine point load on girder due to beam:

W_{DL} = 77.5 psf (no framing allowance)

$P_{DL} = [(77.5)(10.5/2 + 7.5/2) + (48\ plf)] \times 37.5/2$

$\rightarrow P_{DL} = 14^k$

$P_{LL} = (80)(10.5/2 + 7.5/2)(37.5/2)$

$\rightarrow P_{LL} = 13.5^k$

$P_u = 1.2(14) + 1.6(13.5) \rightarrow P_u = 38.4^k$

Exterior wall load:

$W_D = (13.9\ psf)(13.33')(1.2) \rightarrow W_u = 0.222\ klf$

Self weight allowable:

$W_u = (50\ plf)(1.2) = 60\ plf = 0.06\ klf$

Determine max moment:

per table 3-23:

$$M_u = \frac{Pab}{\ell} + \frac{w\ell^2}{8}$$

$$= \frac{(38.4)(10.5)(7.5)}{18} + \frac{(.222+.06)(18^2)}{8}$$

M_u = 180 k·ft

Strength:

using Table 3-2: and choosing an economic section

$\rightarrow$ W16×31

$\phi M_n = 203'^k > 180'^k = M_u$ ✓

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Deflections: using Table 3-23

due to point load:

$$\Delta_{max} = \frac{Pab(a+2b)\sqrt{3a(a+2b)}}{27 EI \ell}$$

$$= \frac{(13.5)(10.5)(7.5)(10.5+2(7.5))\sqrt{3(10.5)(10.5+2(7.5))}(1728)}{27(29000)(375)(18)}$$

$\Delta_{max} = 0.251''$

$$\frac{L}{360} = \frac{18(12)}{360} = 0.60'' > \Delta_{max} \checkmark$$

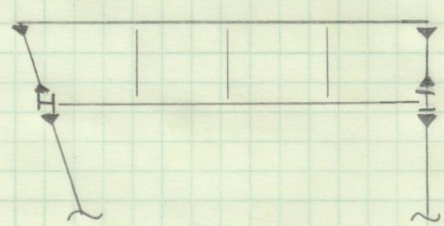
Check Self-weight:

31 plf < 50 plf allowance ✓

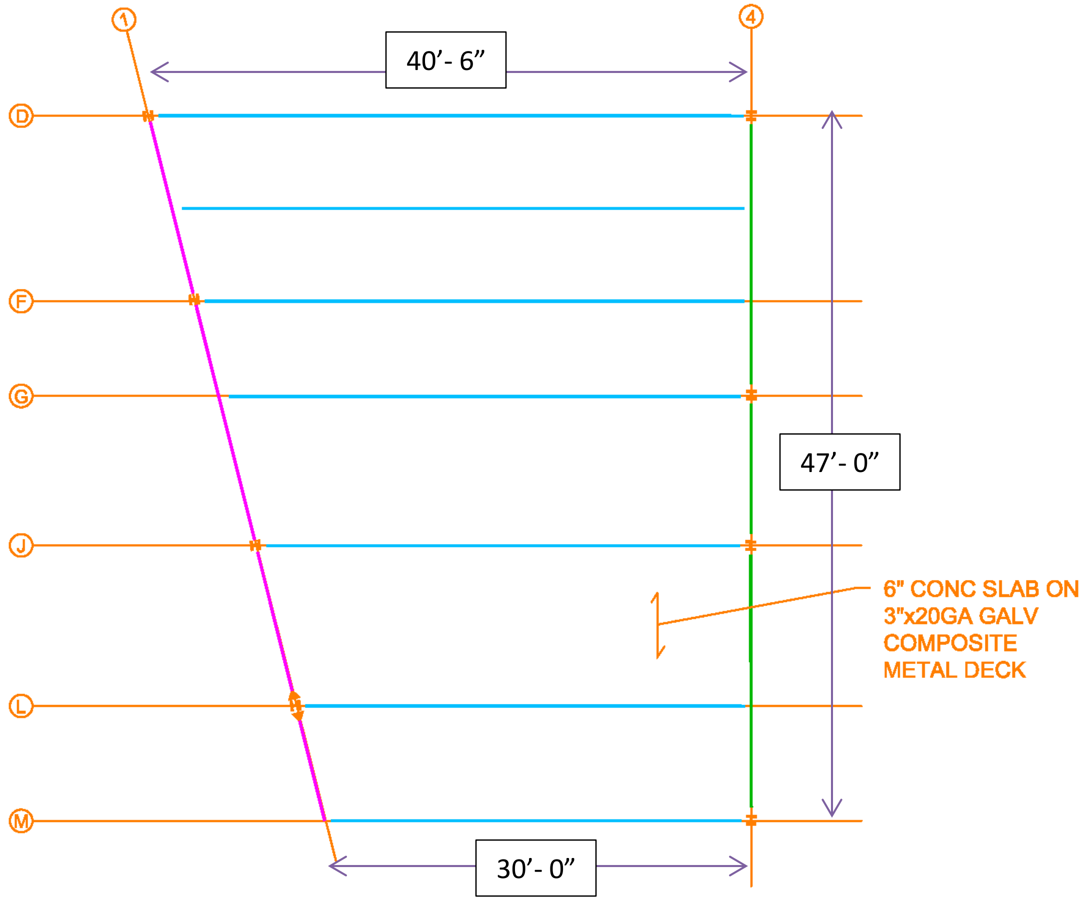
→ $\boxed{\text{Typical Exterior Girder W16×31}}$

Design for Cantilever:

- I would design for the cantilever by specifying the girder-to-column connections to be moment connections.



NON-COMPOSITE STEEL SYSTEM



TYPICAL BEAM: W21x48
TYPICAL INTERIOR GIRDER: W21x48
TYPICAL EXTERIOR GIRDER: W16x31

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GLU-LAM RESIGN:

Decking:

3" Tongue & Groove Decking
covered with 1" nominal dimension tongue-and-groove flooring
* per IBC 2009 Section 602.4.4 *

per Western Lumber Product Use Manual - Table 12:

Surfaced size = 12" x 2.5"

Section Modulus (S) = 12.50 in³ (per foot of width)

per NDS Supplement 2012 - Table 4E

SPF 2-4" nominal thickness → $F_b = 1200$ psi

Calculate max moment it can handle:

$f_b = \frac{M}{S} \rightarrow M = Sf_b$
$= (12.50)(1200)/1200 \rightarrow \underline{M_{max} = 1.25 \text{ k-ft}}$

Load on decking (per foot of width):

Dead Load:
$\underset{\text{typical}}{87.5} - \underset{\text{deck}}{57} - \underset{\text{steel allowance}}{10} + \underset{\text{self weight}}{2} = 22.5$ psf

Live Load:
80 psf

Total Load:
(22.5 + 80)(1 ft of width) = 102.5 plf

→ W = 0.1025 klf

Solve for maximum allowable deck span:

factor of safety → $\frac{M}{F.S.} = \frac{w\ell^2}{8}$

$\frac{1.25}{1.67} = \frac{0.1025\,\ell^2}{8} \rightarrow \ell = 7.64$ ft

→ max deck span is 7 ft (conservative)

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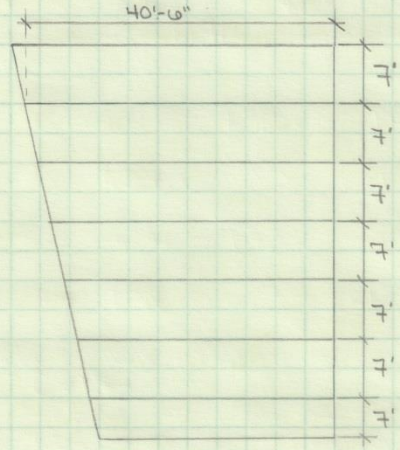
Beam Design:

*Since the max deck span allowed is 7 ft, this redesign will consist of (7) beams spaced at 7 ft on-center and (1) beam at 5 ft. O.C.

*max beam span is 40'-6"



* per IBC 2009 section 602.4.2 Glulam beams & girders shall have a nominal width not less than 6"

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Solve for moment on the beam:

Dead Load = 22.5 + 15 = 37.5 psf

load on decking

Glulam allowance

Live Load = 80 psf

Total Load = 37.5 + 80 = 117.5 psf

$w = (117.5 \text{ psf})(7 \text{ ft}) = \sim 823 \text{ plf} \rightarrow w = 0.823 \text{ klf}$

$M = \frac{wl^2}{8}$ $M = \frac{(0.823)(40.5)^2}{8}$

$\rightarrow M_u = 169{,}000 \text{ ft}\cdot\text{lbs}$

per Boise Glulam Product Guide:

$M_u = 169{,}000$ ft·lbs is not feasible

Consider spacing beams closer:

(16) beams @ 3' o.c. and (1) beam @ 2' o.c.

$w = (117.5)(3) = 353$ psf

$M = \frac{(353)(40.5)^2}{8}$ $\rightarrow M = 72{,}376 \text{ lb}\cdot\text{ft}$

per Boise Glulam Product Guide: 8¾ × 24 24F-V4

allowable moment $= 148585\left(\frac{21}{40.5}\right) = 77{,}044 \text{ lb}\cdot\text{ft}$

adjustment per footnote

$M_{max} = 72{,}376 \text{ lb}\cdot\text{ft} < M_{allow} = 77{,}044 \text{ lb}\cdot\text{ft}$ ✓

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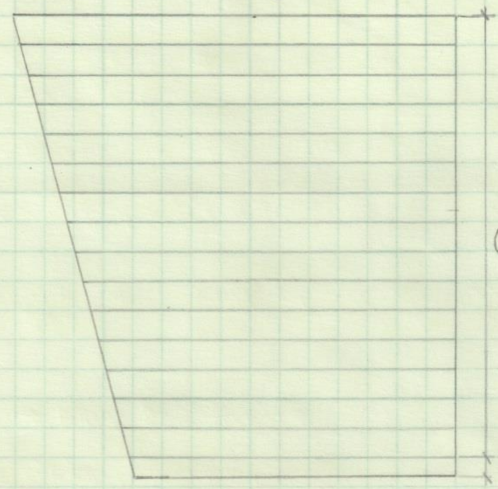
Typical Bay Beam Design:



(16) 8 3/4" × 24"
24F-V4
@ 3'-0" o.c.

(1) 8 3/4" × 24"
24F-V4
@ 2'-0" o.c.

check depth:

per IBC 2009 Section 602.4.2

Glulam beams shall not have a nominal depth less than 10"
→ this is met ✓

*Note: the minimum dimensions in IBC are provided to ensure there is sufficient material in the case of a fire

$\boxed{\text{Typical Beam: } 8\tfrac{3}{4}" \times 24" \text{ 24F-V4}}$

Deflections: $\Delta_{max} = \dfrac{5wl^4}{384EI} = \dfrac{5(80)(40.5)^4(1728)}{384(1.7\times10^6)(10080)} \rightarrow \Delta_{max} = 0.283"$

$L/360 = 40.5(12)/360 = 1.35" > \Delta_{max}$ ✓

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Girder Design:

* Start by keeping current girder locations & spans so that columns do not need to be changed.

Interior Girder:

The longest interior girder span is 19'-6". This girder will support 6 beams. Therefore, I can apply the floor load as a distributed load instead of points loads (from beams).

Moment on Girder:

Dead Load = 37.5 psf (includes Glulam allowance)
Live Load = 80 psf

Total Load = 117.5 psf

$W = 117.5\,(40.5/2 + 33/2) \rightarrow W = 4319$ plf

$M = \frac{wl^2}{8} = \frac{(4319)(19.5)^2}{8} \rightarrow \underline{M_{max} = 205{,}288 \text{ lb·ft}}$

per Boise Glulam Commercial Guide: 10¾" × 27" 24F-V4

allowable moment = $\underline{223{,}679 \text{ ft·lb}}$

$M_{max} = 205{,}288 \text{ lb·ft} < M_{allow} = 223{,}679 \text{ ft·lb}$ ✓

$\boxed{\text{Typical Interior Girder: } 10\,3/4'' \times 27''\ \ 24F\text{-}V4}$

per IBC 2009 Section 602.4.2

width: 10¾" > 6" ✓ } dimensional requirements
depth: 27" > 10" ✓ } are met

Deflections: $\Delta_{max} = \frac{5wl^4}{384EI} = \frac{5(80)(19.5)^4(1728)}{384(1.7\times10^6)(17632.7)} \rightarrow \Delta_{max} = 0.00869''$

$L/360 = 19.5(12)/360 = 0.65'' > \Delta_{max}$ ✓

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Exterior Girder:

* Start by keeping current girder locations & spans so that columns do not need to be changed.

The longest exterior girder span is 17'-3". This girder will support 5 beams. Therefore, I can apply the floor load as a distributed load instead of point loads (from beams).

Moment on Girder:

Dead Load = 37.5 psf (includes Glulam allowance) + (13.9 psf × 13.33 ft) ← exterior wall load

= (37.5)(40.5/2) + (13.9)(13.33)
= 945 plf

Live Load = 80 psf (40.5/2) = 1620 plf

Total Load = 945 + 1620 = 2565 plf

$$M = \frac{wl^2}{8} = \frac{(2565)(17.25)^2}{8} \rightarrow M = 95{,}406 \text{ lb·ft}$$

per Boise Glulam Commercial Guide: 8 3/4" × 19 1/2" 24F-V4

allowable moment = 100,147 ft·lb

M_{max} = 95,406 lb·ft < M_{allow} = 100,147 ft·lb ✓

$\boxed{\text{Typical Exterior Girder: } 8\tfrac{3}{4}" \times 19\tfrac{1}{2}" \text{ 24F-V4}}$

per IBC 2009 section 602.4.2

width: 8 3/4" > 6" ✓
depth: 19 1/2" > 10" ✓
} dimensional requirements are met

Deflections:

$$\Delta_{max} = \frac{5wl^4}{384\,EI} = \frac{5(80)(17.25)^4(1728)}{384\,(1.7\times10^6)(5406.7)} \rightarrow \Delta_{max} = 0.0174"$$

$$L/360 = 17.25(12)/360 = 0.575" > \Delta_{max} \checkmark$$

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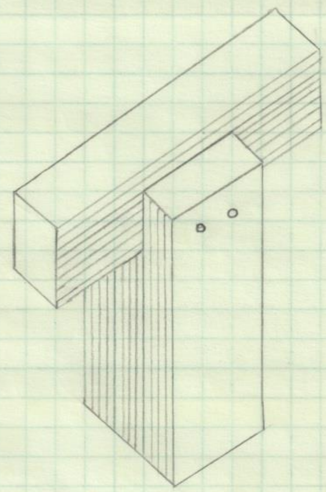
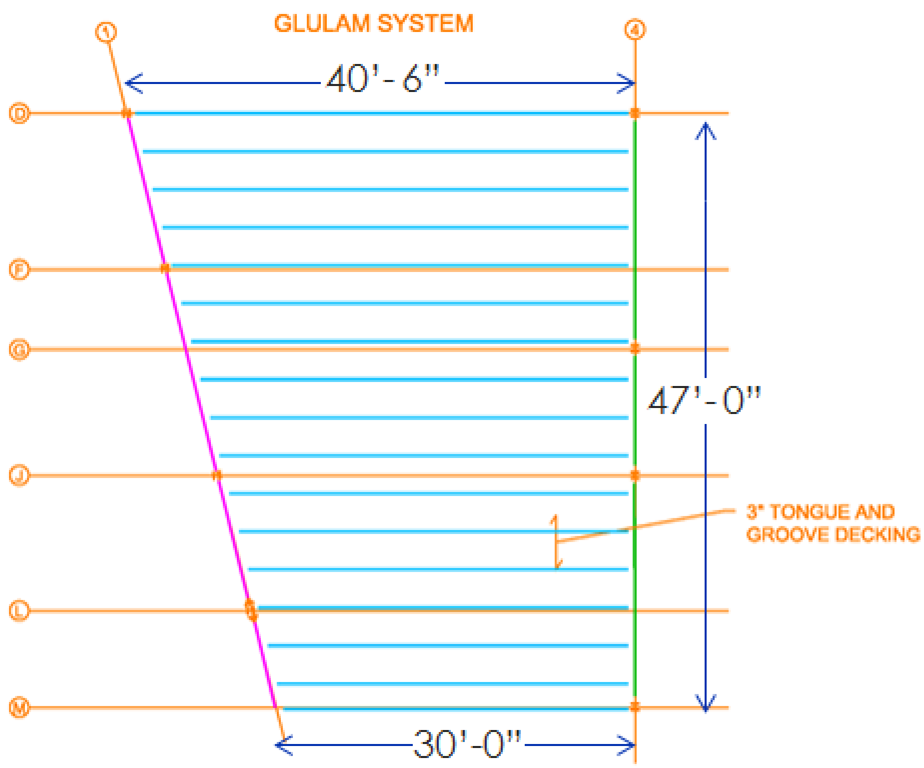
Design for Cantilever:

- I would design for the cantilever by simply continuing the girders from the previous span out to the cantilevered section.





TYPICAL BEAM: 8¾" x 24" 24F-V4
TYPICAL INTERIOR GIRDER: 10¾" x 27" 24F-V4
TYPICAL EXTERIOR GIRDER: 8¾" x 19½" 24F-V4

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* For this redesign, I will design a post-tensioned slab that runs from east to west in my typical bay. Therefore, the longest span is 40'-6".
I will then design a reinforced concrete beam which will support the slab and be positioned where the pre-existing steel girders were. By placing the beams where the steel girders were, I can hopefully avoid the need to move columns.

Span = 40.5'
live load = 80 psf
superimposed dead load:
carpet = 1 psf
carpet pad & adhesive = 1.5 psf
mechanical equipment & piping = 5 psf
sprinklers = 5 psf
lighting = 5 psf
suspended ceiling = 3 psf

→ superimposed dead load = 20.5 psf

Try 12" post-tensioned slab

150 pcf (1 ft) = 150 psf self weight

$f'c = 4000$ psi

→ work with a one foot strip

1' (80 + 20.5 + 150) = 250.5 plf/ft

midspan moment:

$$M = \frac{wl^2}{8} = \frac{(250.5)(40.5^2)}{8} \div 1000 \rightarrow M = 51.4 \text{ k-ft/ft}$$

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try 0.6" diameter lo-lax strands @ 11" o.c.:

initial prestress = $0.9(0.217 \text{ in}^2)(270 \text{ k/in}^2) \div \frac{11}{12}$ (ft/strand)
= 57.5 k/ft

effective prestress = $0.55(0.217 \text{ in}^2)(270 \text{ k/in}^2) \div \frac{11}{12}$ (ft/strand)
= 35.2 k/ft

Midspan Stresses - service loads:

$A = 12" \times 12" = 144 \text{ in}^2/\text{ft}$

$I = \frac{1}{12} \times 12" \times (12")^3 = 1728 \text{ in}^4/\text{ft}$

$S_t = S_b = \frac{12" \times (12")^2}{6} \rightarrow S_t = S_b = 288 \text{ in}^3/\text{ft}$

$e_{midspan} = +5 \text{ in}$ (strands at midspan depressed below neutral axis)

- try 2 strands

$$f_t = -\frac{M}{S_t} - \frac{P_e}{A} + \frac{P_e \cdot e}{S_t}$$

$$= -\frac{51.4 \frac{k\cdot ft}{ft}(12 \frac{in}{ft})}{288 \text{ in}^3/\text{ft}} - \frac{(2)(35.2 \text{ k/ft})}{144 \text{ in}^2/\text{ft}} + \frac{(2)(35.2 \text{ k/ft})(5 \text{ in})}{288 \text{ in}^3/\text{ft}}$$

$\rightarrow f_t = -1.408 \text{ k/in}^2 = -1408 \text{ lb/in}^2 < 2400 \text{ lb/in}^2$ ✓

$0.6 f'c = 0.6(4000) = 2400 \text{ lb/in}^2$

$$f_b = \frac{M}{S_b} - \frac{P_e}{A} - \frac{P_e \cdot e}{S_b}$$

$$= \frac{51.4(12)}{288} - \frac{(2)(35.2)}{144} - \frac{(2)(35.2)(5)}{288}$$

$\rightarrow f_b = 0.431 \text{ k/in}^2 = 431 \text{ lb/in}^2 < 474 \text{ lb/in}^2$ ✓

Class U Check: $7.5\sqrt{f'c} = 7.5\sqrt{4000} = 474 \text{ lb/in}^2$

→ design is good for midspan stresses under service load

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Midspan Stresses - at transfer:
(using initial prestress force)

initial prestress $(P_i) = 57.5$ k/ft

$$f_t = -\frac{M}{S_t} - \frac{P_i}{A} + \frac{P_i e}{S_t}$$

$$= -\frac{51.4(12)}{288} - \frac{(2)57.5}{144} + \frac{(2)(57.5)(5)}{288}$$

$\rightarrow f_t = -0.944\ \text{k/in}^2 = -944\ \text{lb/in}^2$

$0.60 f'_{ci}$ *using $f'_{ci} = 3500$ psi

$0.60(3500) = 2100\ \text{lb/in}^2$

$f_t = -944\ \text{lb/in}^2 < 0.60 f'_{ci} = 2100\ \text{lb/in}^2$ ✓

$$f_b = \frac{M}{S_b} - \frac{P_i}{A} - \frac{P_i \cdot e}{S_b}$$

$$= \frac{51.4(12)}{288} - \frac{(2)(57.5)}{144} - \frac{(2)(57.5)(5)}{288}$$

$\rightarrow f_b = -0.653\ \text{k/in}^2 = -653\ \text{lb/in}^2$

$f_b = -653\ \text{lb/in}^2 < 0.60 f'_{ci} = 2100\ \text{lb/in}^2$ ✓

→ design is good for midspan stresses at transfer.

Slab Thickness: 12"
Tendons:
lo relaxation 7-wire strand
0.6" diameter
(2) strands @ 11" o.c.
run east to west
$f'_c = 4000$ psi
$f'_{ci} = 3500$ psi

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Concrete Beam Design:

Interior Beam: former W24×55 along column line 4
this is the worst case (longest span)

Loads:

superimposed dead load = 20.5 psf

slab weight = 150 pcf (1 ft) = 150 psf

live load = 80 psf

$W_u = 1.2\,W_D + 1.6\,W_L$
$= 1.2(20.5+150) + 1.6(80)$

$\rightarrow W_u = 332.6 \text{ psf } (40.5/2 + 33/2)/1000 \rightarrow W_u = 12.3 \text{ klf}$

$M_u = \left(\dfrac{12.3(19.5^2)}{8}\right) \times 1.1$ ← beam self-weight allowance

$\rightarrow M_u = 643 \text{ k}\cdot\text{ft}$

Determination of beam size:

$bd^2 \geq 20 M_u$ try $b = 4/5\,d$

$(4/5\,d)d^2 \geq 20(643)$

$d = 25.24"$

$b = 4/5\,d = 4/5(25.24) \rightarrow b = 20.20"$

$h = d + 2.5 = 25.24 + 2.5 \rightarrow h = 27.74"$

$h = 28"$ $d = 25.5"$
$b = 21"$

Compute self-weight effects:

$W_{sw} = \dfrac{(28)(21)}{144}(150) = 613 \text{ plf}$

$W_u = 12.3 \text{ klf} + 1.2(0.613) \rightarrow W_u = 13.1 \text{ klf}$

$M_u = \dfrac{wl^2}{8} = \dfrac{(13.1)(19.5^2)}{8} \rightarrow \underline{M_u = 623 \text{ k}\cdot\text{ft}}$

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Required Steel:

$$A_s = \frac{M_u}{4d} = \frac{623}{4(25.5)}$$

assuming:
$\rho = 1.25$
$f'c = 4000$ psi
$f_y = 60$ ksi

$$A_s = 6.11 \text{ in}^2$$

→ (5) #10

$$A_s = (5)(1.27) = 6.35 \text{ in}^2$$

$$\rho = \frac{A_s}{b \cdot d} = \frac{6.35}{(21)(25.5)} = 1.19\% < 1.25\% \checkmark$$

⇒ equation above may be used for estimating

Check Flexural Strength:

• assume steel is yielding $(\varepsilon_s > \varepsilon_y)$

$$a = \frac{A_s f_y}{0.85 f'c \cdot b} = \frac{(6.35)(60)}{0.85(4)(21)} \rightarrow a = 5.34"$$

$$d = h - C_{cl} - d_{TR} - \tfrac{1}{2}d_b$$
$$= 28 - 1.5 - .5 - \tfrac{1}{2}(1.27) \rightarrow d = 25.3"$$

$$f'c = 4000_{psi} \rightarrow \beta_1 = 0.85$$

$$c = a/\beta_1 = 5.34/0.85 \rightarrow c = 6.28"$$

$$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{6.28}(25.3 - 6.28) \rightarrow \varepsilon_s = 0.009086$$

$$\varepsilon_s > \varepsilon_y = 0.00207 \checkmark$$

$$M_n = A_s \cdot f_y (d - a/2)$$
$$= (6.35)(60)(25.3 - 5.34/2)/12 \rightarrow M_n = 718.5 \text{ k} \cdot \text{ft}$$

$$\varepsilon_s = \varepsilon_T = 0.009086 > .005 \rightarrow \Phi = 0.90$$

$$\Phi M_n = (0.90)(718.5) \rightarrow \Phi M_n = 646 \text{ k} \cdot \text{ft}$$

$$646 \text{ k} \cdot \text{ft} = \Phi M_n > M_u = 623 \text{ k} \cdot \text{ft} \checkmark$$

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Check $A_{s\,min}$ & $A_{s\,max}$:

$$A_{s,min} = \frac{3\sqrt{f'_c}}{f_y} b_w d \geq \frac{200 b_w d}{f_y}$$

$$= \frac{3\sqrt{4000}}{60000}(21)(25.3) \quad \frac{200(21)(25.3)}{60000}$$

$$= 1.68 > 1.77 \times$$

$$A_{s,min} = 1.77 \text{ in}^2 < 6.35 \text{ in}^2 \checkmark$$

$$\rho_{max} = 0.85 \beta_1 \frac{f'_c}{f_y}\left(\frac{.003}{.003 + .004}\right)$$

$$= .85(.85)\left(\frac{4}{60}\right)\left(\frac{.003}{.007}\right) \rightarrow \rho_{max} = 0.0206$$

$$A_s = \rho \cdot b_w \cdot d = (0.0206)(21)(25.3)$$

$$A_{s\,max} = 10.9 \text{ in}^2 > 6.35 \text{ in}^2 \checkmark$$

Check spacing:

using Table A.7 adopted from Ref. 3.8 (ACI)

b = 21"
#10 bars

from table b = 20" #10 bars → 6 bars in single layer max

Therefore, (5) #10 with b = 21" is okay ✓
(spacing requirement met)

Check Crack Control:

Using Table A.8 from ACI 318-11 §10.6.4

2" clear cover (#4 stirrup)
b = 21"
#10 bars

from table b = 20" #10 bars → 3 bars min in a single layer

5 > 3 ✓
→ Crack control is met

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Check cover:

$d_{bms} = h - \text{clear cover} - d_{stirr} - d_b/2$

$25.3 = 28 - C_{CL} - .5 - 1.27/2$

$C_{CL} = 1.565" > 1.5"$ ✓ clear cover is met

Check Shear Capacity:

$V_u = 13.1\text{ klf}\,(19.5/2) = 128^k$

$V_u@d = 128 - (13.1)(25.3)/12 = 101^k$

$V_c = 2\lambda\sqrt{f'c}\,b_w \cdot d$
$= 2(1)\sqrt{4000}\,(21)(25.3) \rightarrow V_c = 67.2^k$

$\phi V_n = 0.5\,\phi V_c$
$= 0.5(.75)(67.2) \rightarrow \phi V_n = 25.2^k < 128^k$

→ need shear reinforcing (stirrups)

Shear strength required by shear reinf:

$V_s = V_u/\phi - V_c \leq 8\sqrt{f'c}\,b_w \cdot d$

$= 101/.75 - 67.2 \leq 8\sqrt{4000}\,(21)(25.3)$

$= 67.5 < 268.8$ ✓

Maximum spacing of shear reinforcing:

$V_s \leq 4\sqrt{f'c} \cdot b_w \cdot d = 4\sqrt{4000}\,(21)(25.3) = 134.4^k$

$V_s \leq 134.4^k$

$S_{max} = \min \begin{cases} d/2 = 25.3/2 = 12.65 \rightarrow 12" \\ 24" \end{cases}$

$\rightarrow S_{max} = 12"$

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Minimum shear reinforcement:

$A_{v\,min} = \max \begin{cases} 0.75\sqrt{f'c}\ b_w \cdot s / f_{yt} = 0.75\sqrt{4000}\,(21)(12)/60000 = .199 \\ 50\ b_w\ s / f_{yt} = 50\,(21)(12)/60000 = .21 \end{cases}$

$A_{v\,min} = 0.21\ in^2$

→ use 2 legs of #3 stirrup @ 12"

$2(.11) = .22 > .21$ ✓

Design of shear reinforcement:

$s = A_v\, f_{yt}\, \dfrac{d}{V_s} \le s_{max}$

$= \dfrac{(0.22)(60)(25.3)}{67.5} \le 12"$

$= 4.95" \le 12"$ ✓

→ $s = 4"$

Stirrup Layout:

terminate stirrups when $V_u \le 0.5\phi V_c = 25.2^k$

$25.2 = 128 - 13.1(x)$

$x = 7.85'$

number of stirrups needed:

$2" + (n-1)(4") \ge 7.85\,(12)$

$n = 24.05$ → 25 stirrups

⇒ (25) #3 × ⊔ stirrups @ 4"
starting 2" from face of support

Beam Design: (Interior)

21" × 28" normal weight concrete ($f'c = 4000$ psi)
(5) #10 longitudinal reinforcing bars
↳ located at $d = 25.3"$
(25) #3 × ⊔ stirrups @ 4"
↳ starting 2" from face of support

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Exterior Beam: for W21×44 along column line 1
this is the worst case (longest span)
(between column lines J & F)

Loads:

superimposed dead load = 20.5 psf

slab weight = 150 pcf (1 ft) = 150 psf

exterior wall load = 0.222 klf (factored)

live load = 80 psf

$W_u = 0.222 + 1.2\left[(20.5+150)(40.5/2)/1000\right] + 1.6(80)(40.5/2)/1000$

$\rightarrow W_u = 7.0$ klf

$M_u = \left(\frac{7.0(18)^2}{8}\right) \times 1.1$

beam self-weight allowance

$\rightarrow M_u = 312$ k·ft

Determination of beam size:

$bd^2 \geq 20 M_u$ try $b = 4/5 d$

$(4/5 d) d^2 \geq 20(312)$

$d = 19.83"$

$b = 4/5 d = 4/5(19.83) \rightarrow b = 15.87"$

$h = d + 2.5 = 19.83 + 2.5 \rightarrow h = 22.33"$

$h = 23"$ $d = 20.5"$
$b = 16"$

Compute self-weight effects:

$W_{sw} = \frac{(23)(16)}{144}(150) = 384$ plf

$W_u = 7.0$ klf $+ 1.2(0.384) \rightarrow W_u = 7.46$ klf

$M_u = \frac{wl^2}{8} = \frac{(7.46)(18^2)}{8} \rightarrow \underline{M_u = 303 \text{ k·ft}}$

A. Mincemoyer | Structural Redesign - Prestressed Conc. | Tech Report 3

Required Steel:

$$A_s = \frac{M_u}{4d} = \frac{303}{4(20.5)}$$

assuming:
$\rho = 1.25$
$f'c = 4000$ psi
$f_y = 60$ ksi

$A_s = 3.70 \text{ in}^2$

→ (5) #8

$A_s = (5)(0.79) = 3.95 \text{ in}^2$

$\rho = \frac{A_s}{b \cdot d} = \frac{3.95}{(16)(20.5)} = 1.21\% < 1.25\%$ ✓ → equation above may be used for estimating

Check Flexural Strength:

* assume steel is yielding ($\varepsilon_s > \varepsilon_y$)

$a = \frac{A_s f_y}{0.85 f'c \cdot b} = \frac{(3.95)(60)}{(.85)(4)(16)}$ → $a = 4.36"$

$f'c = 4000$ psi → $\beta_1 = 0.85$

$d = h - C_{CL} - d_{TR} - \frac{1}{2} d_b$
$= 23 - 1.5 - .5 - \frac{1}{2}(1)$ → $d = 20.5"$

$c = a/\beta_1 = 4.36/.85$ → $c = 5.13"$

$\varepsilon_s = \frac{\varepsilon_u}{c}(d - c) = \frac{0.003}{5.13}(20.5 - 5.13)$ → $\varepsilon_s = 0.00899$

$\varepsilon_s > \varepsilon_y = 0.00207$ ✓

$M_n = A_s \cdot f_y (d - a/2)$
$= 3.96(60)(20.5 - 4.36/2)/12$ → $M_n = 362.7$ k·ft

$\varepsilon_s = \varepsilon_T = 0.00899 > 0.005$ → $\phi = 0.90$

$\phi M_n = (0.90)(362.7)$
→ $\phi M_n = 326$ k·ft

326 k·ft $= \phi M_n > M_u = 303$ k·ft ✓

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Check As min & As max:

$$A_{s,min} = \frac{3\sqrt{f'c}}{f_y} b_w \cdot d \geq \frac{200 b_w \cdot d}{f_y}$$

$$= \frac{3\sqrt{4000}(16)(20.5)}{60000} \quad - \quad \frac{200(16)(20.5)}{60000}$$

$$1.04 \geq 1.09 \times$$

$$A_{s,min} = 1.09 \text{ in}^2 < 3.95 \text{ in}^2 \checkmark$$

$$\rho_{max} = 0.85 \beta_1 \frac{f'c}{f_y} \left(\frac{.003}{.003 + .004}\right)$$

$$= 0.85(0.85)\left(\frac{4}{60}\right)\left(\frac{.003}{.007}\right) \rightarrow \rho_{max} = 0.0206$$

$$A_s = \rho \cdot b_w \cdot d = (0.0206)(16)(20.5)$$

$$A_{s,max} = 6.75 \text{ in}^2 > 3.95 \text{ in}^2 \checkmark$$

Check Spacing:

using Table A.7 adapted from Ref. 3.8 (ACI)

b = 16"
#8 bars } from table: 6 bars in single layer max

Therefore, (5) #8 with b = 16" is okay ✓
(spacing requirement met)

Check Crack Control:

using Table A.8 from ACI 318-11 § 10.6.4

2" clear cover (#4 stirrup)
b = 16"
#8 bars

from table: b = 16" #8 bars → 3 bars min in a single layer

5 > 3 ✓

→ crack control is met

A. Mincemoyer | Structural Redesign - Prestressed Conc. | Tech Report 3

Check cover:

$d_{bms} = h - \text{clear cover} - d_{stirr} - d_b/2$

$20.5 = 23 - C_{CL} - .5 - \frac{1}{2}$

$C_{CL} = 1.5" \geq 1.5"$ ✓ → Clear cover is met

Check Shear Capacity:

$V_u = 7.46\text{ klf}\,(18/2) = 67.2^k$

$V_u @ d = 67.2 - (7.46)(20.5)/12 = 54.5^k$

$V_c = 2\lambda\sqrt{f'_c}\, b_w \cdot d$
$= 2(1)\sqrt{4000} \cdot 16 \cdot 20.5$ → $V_c = 41.5^k$

$\phi V_n = 0.5\,\phi V_c$
$= 0.5(.75)(41.5)$ → $\phi V_n = 15.6^k < 54.5^k$

→ need shear reinforcing (stirrups)

Shear strength required by shear reinf:

$V_s = V_u/\phi - V_c \leq 8\sqrt{f'_c}\, b_w \cdot d$

$= 54.5/.75 - 41.5 \leq 8\sqrt{4000}\,(16)(20.5)$

$31.2 \leq 165.96$ ✓

Max spacing of shear reinforcing:

$V_s \leq 4\sqrt{f'_c} \cdot b_w \cdot d = 4\sqrt{4000}\,(16)(20.5) = 83.0^k$

$V_s \leq 83.0^k$

$S_{max} = \min \begin{cases} d/2 = 20.5/2 = 10.25" \\ 24" \end{cases}$

→ $S_{max} = 10"$

A. Mincemoyer | Structural Redesign - Prestressed Conc. | Tech Report 3

Minimum shear reinforcement:

$$A_{v\,min} = \max \begin{cases} 0.75\sqrt{f'_c}\, b_w \cdot s / f_{yt} = 0.75\sqrt{4000}\,(16)(10)/60000 = 0.126 \\ 50\, b_w \cdot s / f_{yt} = 50(16)(10)/60000 = 0.133 \end{cases}$$

$A_{v\,min} = 0.133 \text{ in}^2$

→ use 2 legs of #3 stirrup @ 10"

$2(.11) = .22 > .133$ ✓

Design of shear reinforcement:

$$s = A_v f_{yt} \frac{d}{V_s} \leq s_{max}$$

$$= \frac{.22(60)(20.5)}{31.2} \leq 10"$$

$s = 8.67 \leq 10"$ ✓

→ $s = 8"$

Stirrup Layout:

terminate stirrups when $V_u \leq 0.5\phi V_c = 15.6^k$

$15.6 = 67.2 - 7.46x$

$x = 6.92'$

number of stirrups needed:

$2" + (n-1)(8) \geq 6.92(12)$

$n = 11.13$ → 12 stirrups

⇒ (12) #3 × ⊔ stirrups @ 8"
starting 2" from face of support

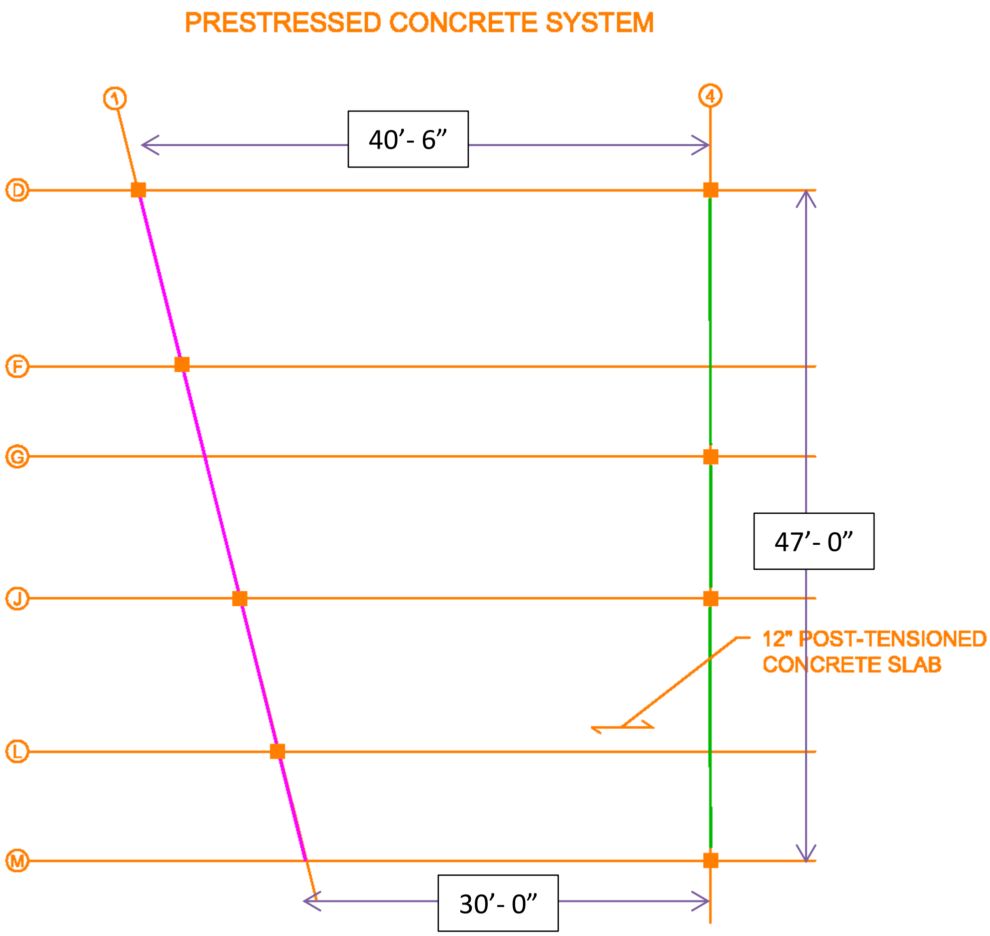
Beam Design: (Exterior)

16" × 23" normal weight concrete ($f'_c = 4000$ psi)
(5) #8 longitudinal reinforcing bars
↳ located at $d = 20.5"$
(12) #3 × ⊔ stirrups @ 8"
starting 2" from face of support

A. Mincemoyer | Structural Redesign - Prestressed Conc. | Tech Report 3

Design for cantilever:

- I would design for the cantilever by extending the two end/perimeter beams out to the cantilever. The reinforcement in that portion of the beams would need to be located near the top of the beam (to provide tensile strength). The post-tensioned slab would then span between the two extended beams. The main difference with the slab in this region is that e would be located above the neutral axis, meaning that the tendons would be near the top of the slab to support the cantilever.



TYPICAL INTERIOR BEAM: 21"x28" NWC
(5) #10 LONGITUDINAL REINFORCING BARS
(25) #3 x 2 LEGS STIRRUPS @ 4"

TYPICAL EXTERIOR BEAM: 16"x23" NWC
(5) #8 LONGITUDINAL REINFORCING BARS
(12) #3 x 2 LEGS STIRRUPS @ 8"

SLAB TENDONS:
LO RELAXATION 7-WIRE STRAND
(2) 0.6" Ø STRANDS @ 11" o.c.

A. Mincemoyer | System Comparison - Cost | Tech Report 3

Composite Steel:

*estimate bay size to 35' × 49'

Superimposed load = superimposed dead load + live load
= 18 psf + 80 psf
= 98 psf

total load = 87.5 + 80
= 168 psf

Bay Size	Total Cost Per S.F.
35×35	$25.70
35×40	$26.25
35×49	x

* based on total load

x = $27.24

* used pg. 283 of R.S. Means Square Foot Cost 2013

Location modifier: 0.95 (Elmira)

$27.24 (0.95) = 25.88

$\boxed{\text{Composite Steel} = \$25.88/\text{SF}}$

A. Mincemoyer | System Comparison - Cost | Tech Report 3

Non-Composite Steel:

* estimate bay size to 35' x 49'

Superimposed load = 98 psf

total load = 168 psf

Bay Size	Total Cost Per S.F.
30 x 35	$33.20
35 x 35	$34.24
49 x 35	x

* based on total load

x = $37.16

* used pg. 282 of R.S. Means Square Foot Cost 2013

Location Mod = 0.95 (Elmira)

$37.16 (.95) = $35.31

$\boxed{\text{Non-composite Steel} = \$35.31/SF}$

A. Mincemoyer | System Comparison - cost | Tech Report 3

Glulam: * using UC Berkley's Industrial Engineering & Operations Research Center data
* estimate bay size to 35' × 49'

Beams: (17) 8¾" × 24"

$17.92 / lineal foot

$17.92 (17 × 35') = $10,662.40
↑ quantity

Interior Girder: 10¾" × 27" 49 ft

Use interpolation: $25.35 / lineal foot

$25.35 (49') = $1,242.15

Deck: 3" Tongue & Groove Decking

* using RS Means: Facilities, Maintenance, and Repair 2013

$465 / 1000 board feet

determine cubic inches of deck:

$(35 \times 12)(49 \times 12)(3") = 740{,}880 \text{ in}^3$

$740{,}880 \text{ in}^3 \left(\frac{1 \text{ brd ft}}{144 \text{ in}^3}\right) = 5145$ board feet

$5145 \text{ board feet} \left(\frac{\$465}{1000 \text{ bd ft}}\right) = \$2{,}392.43$

Exterior Girder: 8¾" × 19½" 49 ft

$14.07 / lineal foot

$14.07 (49') = $689.43

Total: 10662.40 + 1242.15 + 2392.43 + 689.43
= 14,986.41

$14,986.41 / (35 × 49)

$\boxed{\text{Glulam} = \$8.74/SF}$

A. Mincemoyer | System Comparison - cost | Tech Report 3

Prestressed: (Post-tensioned slab w/concrete beams)

* estimate bay size to 35' × 49'

total load = 168 psf

Slab Thickness	Total Cost Per S.F.
8	$19.03
9	$19.95
12	x

x = $22.71

*used pg. 266 of RS Means Square Foot Cost 2013

* add an additional $2/SF to account for tendons (national average)

Location modifier (Elmira)

($22.71 + 2)(.95) = $23.48/SF

$\boxed{\text{Prestressed} = \$23.48/\text{SF}}$

A. Mincemoyer | System Comparison - weight | Tech Report 3

Composite Steel:

* estimate bay size to 35' × 49'

- Composite deck - 3VLI20 = 57 psf

$(57\text{ psf})(35')(49') = 97.76\text{ k}$

- beams:

$(68+55+55+55+50+44+31)\text{plf} \times 35\text{ ft}$

$= 12.53\text{ k}$

- girders:

$(55+44+44+44+55+35+44)\text{plf} \times 49\text{ ft}$

$= 15.73\text{ k}$

- shear studs:

$(35+25+25+25+25+25+20+10+15+10+10)10\#/\text{stud}$

$= 2.25^{k}$

Total: $(97.76 + 12.53 + 15.73 + 2.25) = 128.27^{k}$

$\frac{128.27^{k}}{(35 \times 49)} = 74.79\text{ psf}$

$\boxed{\text{Composite Steel} = 74.79\text{ psf}}$

A. Mincemoyer | System Comparison - weight | Tech Report 3

Non-Composite Steel:

* estimate bay size to 35' × 49'

• composite deck - 3VLI20 = 57 psf

(57 psf)(35')(49') = 97.76^k

• beams:

(48 plf)(35 ft)(7 beams) = 11.76 k

• girders:

(48 plf)(49 ft) + (31 plf)(49 ft) = 3.88 k

Total: (97.76 + 11.76 + 3.88) = 113.4^k

$\frac{113.4}{(35 \times 49)} = 66.13$ psf

$\boxed{\text{Non-composite Steel} = 66.13 \text{ psf}}$

A. Mincemoyer | System Comparison -weight | Tech Report 3

Glulam:

* estimate bay size to 35' × 49'

- Tongue & Groove Deck

$(2\ psf)(35' \times 49') = 3.43\ k$

- Beams:

$(51\ plf)(35')(17) = 30.35\ k$

- Girders

$(41.5\ plf)(49') + (70.5\ plf)(49') = 5.49^k$

Total: $3.43 + 30.35 + 5.49 = 39.27^k$

$\frac{39.27^k}{(35 \times 49)} = 22.90\ psf$

$\boxed{\text{Glulam} = 22.90\ psf}$

A. Mincemoyer | System Comparison - weight | Tech Report 3

Prestressed:

* estimate bay size to 35' × 49'

- Tendons
  - using "American Society of Professional Estimators"

    0.62 plf / tendon

    $(0.62\ \text{plf/tendon})(2\ \text{tendons})(35\ \text{ft})\left(\frac{49'}{11''/12}\right)$

    $= 2.32\ k$

- Slab:

  $(150\ \text{pcf})(12''/12)(35')(49') = 257.25^k$

- Beams:

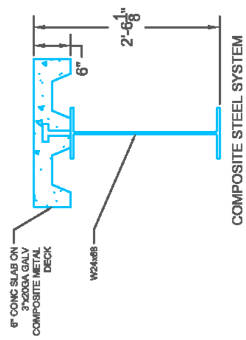
  $(150\ \text{pcf})(24/12)(28/12)(49) + (150\ \text{pcf})(16/12)(23/12)(49')$

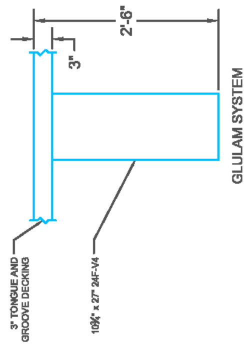
  $= 48.80\ k$

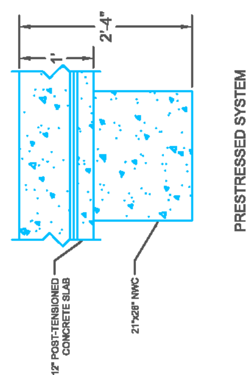
Total: $2.32 + 257.25 + 48.80 = 308.37^k$

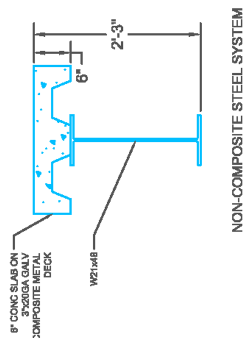
$\frac{308.37^k}{(35 \times 49)} = 179.81\ \text{psf}$

$\boxed{\text{Prestressed} = 179.81\ \text{psf}}$


2'-6 1/8"
6"
6" CONC SLAB ON 3"x20GA GALV COMPOSITE METAL DECK
W24x68
COMPOSITE STEEL SYSTEM


2'-6"
3"
3" TONGUE AND GROOVE DECKING
10 3/4" x 27" 24F-V4
GLULAM SYSTEM


2'-4"
1'
12" POST-TENSIONED CONCRETE SLAB
21"x28" NWC
PRESTRESSED SYSTEM


2'-3"
6"
6" CONC SLAB ON 3"x20GA GALV COMPOSITE METAL DECK
W21x48
NON-COMPOSITE STEEL SYSTEM

Considerations	System							
	Composite Steel (Existing System)	Rank	Non-Composite Steel	Rank	Glulam	Rank	Prestressed Concrete (Post-tensioned Slab)	Rank
General								
Cost ($/SF)	$25.88	3	$35.31	4	$8.74	1	$23.48	2
System Weight (psf)	74.79	3	66.13	2	22.90	1	179.81	4
Durability	acceptable	1	acceptable	1	acceptable	1	acceptable	1
Architectural								
System Depth (in)	30.125	4	27	1	30	3	28	2
Additional Fire Proofing Req'd	none		none		none		none	
Fire Rating (hours)	0	3	0	3	1**	1	2	2
Conclusion								
Viable?	yes	14	yes (not recommened)	11	yes	7	yes	11
Future Investigation?	n/a		no		yes		yes	

*NOTE: PRWC is Type IIB Construction Type → According to IBC 2009 Section 601 (Table 601), no additional fire proofing is required on structural elements.
** Glulam is rated at 1 hour. After this 1 hour, 1.5" have been charred and the structural element is no longer reliable.

Appendix A

Second Floor Framing Plan