SSM – St. Clare Health Center: Fenton, Missouri

Technical Report 3

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Executive Summary

SSM St. Clare Health Center is a 420,000 square foot hospital located in a residential area of Fenton, Missouri. The building and parking areas sit on a 54 acre site, which was previously a 9-hole golf course with gently varying topography, large stands of trees, and a 3 acre pond. The hospital program contains a wide variety of medical use spaces, including 158 emergency supported inpatient beds, diagnostic and surgical services, administrative offices, dietary facilities, and pharmaceutical dispensaries. Budgeted at $226.8 million, the hospital was constructed with an Integrated Project Delivery method and came in well under budget at $223.5 million.

Structurally, the hospital is a composite steel frame building resting on massive concrete drilled piers which are connected by grade beams. The structure is broken up into three buildings (bed tower, surgery tower, and interventional care unit) isolated by expansion joints. These individual buildings each contain their own lateral force resisting systems which include special moment frames (SMF), special concentrically braced frames (SCBF), special reinforced concrete shear walls (SRCSW), and ordinary concentrically braced frames (OCBF).

HGA Architects and Engineers served as the primary architects and structural engineers on the project. They worked closely with the MEP engineers, KJWW, and the construction manager, Alberici Construction, through an integrated “Lean” project delivery contract that focused on improving coordination and quality by sharing project risks. The project began construction in September of 2006 and reached completion in March of 2009.

SSM St. Clare Health Center was designed in 2004 and uses the 2003 Edition of the International Building Code and ASCE 7-02 as a reference standard. Design loads were determined based on these codes, additional St. Louis County Codes and Ordinances, and practical engineering judgments.

SSM St. Clare Health Center

Fenton, Missouri: St. Louis County

General Information

Full Height:	90 feet
Number of Stories:	6
Size:	427,000 gross square feet
Cost:	$223.5 million
Date of Construction:	Sept. 2006 – March 2009
Project Delivery Method:	Integrated "Lean" Project Delivery

Project Team

Owner:	SSM Health Care, St. Louis
Owner's Program Manager:	Hammes Company
Architect of Record:	HGA Architects and Engineers
Associate Architect:	Mackey Mitchel Associates
Structural Engineers:	HGA Architects and Engineers
MEP Engineers:	KJWW Engineering
Construction Manager:	Alberici Construction
Elevator Consultants:	Lerch, Bates & Associates Inc.

Architecture

- 158 emergency supported inpatient beds
- Diagnostic and surgical services
- Dietary facilities and pharmaceutical dispensaries
- Floor plans developed using Lean process principles classically used in manufacturing facilities.

Structural Systems

- Framing
    - Steel framing, composite deck and lightweight concrete
    - Composite wide flange members
- Foundations
    - Slab on grade
    - Drilled concrete piers connected by grade beams
- Lateral System
    - special moment frames (SMF)
    - special concentrically braced frames (SCBF)
    - special reinforced concrete shear walls (SRCSW)
    - ordinary concentrically braced frames (OCBF)

Mechanical Systems

- Fan coil units in each patient room fed by central boiler and chiller system
- VAV dedicated outside air for ventilation.

Lighting and Electrical Systems

- Back up generators designed to power the entire hospital for >90 minutes
- Ultrasonic ceiling sensors and infrared wall switch sensors for energy savings.

Construction

- Special noise control procedures implemented to minimize disturbance to local residential neighborhoods.


Rendering of full hospital complex


Bed tower façade


Typical patient room


Ground floor atrium

Photos compliments of HGA Architects and Engineers

Christopher Brandmeier | Structural Option
https://www.engr.psu.edu/ae/thesis/portfolios/2015/aqb5205/index.html

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1 General Information

1.1 Purpose

This report contains spot checks of typical members in SSM St. Clare Health Center and evaluates several other alternate floor assembly designs for a typical interior bay. SSM St. Clare Health Center's original system is a composite steel framing system. The alternates are a non-composite system, a two way flat plate concrete slab system, a one way concrete slab system with intermediate beams, and a one way concrete slab system with full span.

1.2 Scope

The major sections of this document discuss dead loads, live loads, and building gravity systems that resists those loads. The building's location and relevant resource documents used in its design are also presented. The appendices to this document contain the original load calculations from HGA Architects and Engineers. The analysis focuses on the bed tower, labelled sections "A" and "B" on the record drawings.

1.3 Site Location and Plan

SSM St. Clare Health Center is located in Fenton, Missouri (St. Louis County) in a relatively open residential area. The site was previously a golf course, which provided open space and gently sloping terrain. Figure 1 shows the relative placement of the site in Missouri, while Figures 2 through 5 show the building's location on the site as dictated by zoning codes and city ordinances.

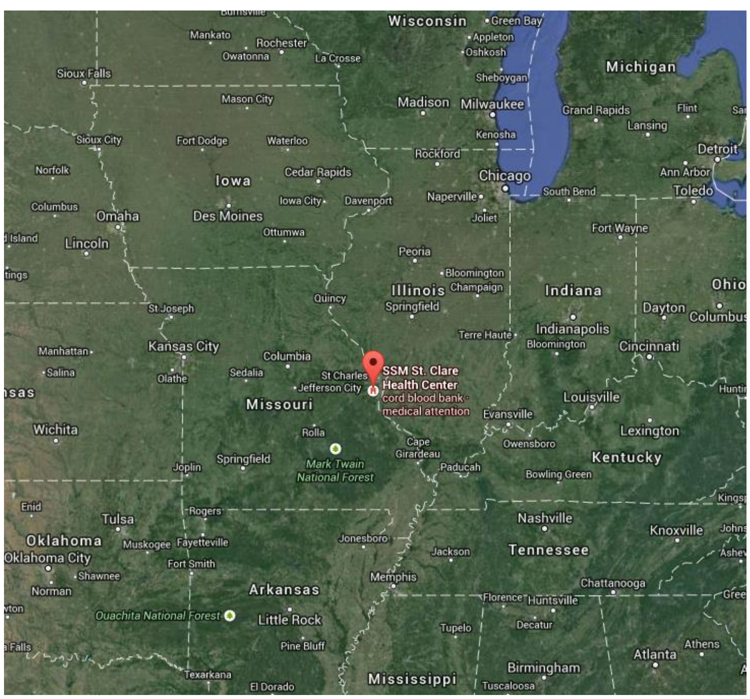


Figure 1: Building Location

Figure 2: Rendering of SSM Health Center Complex

Figure 3: Original Site, Golf Course

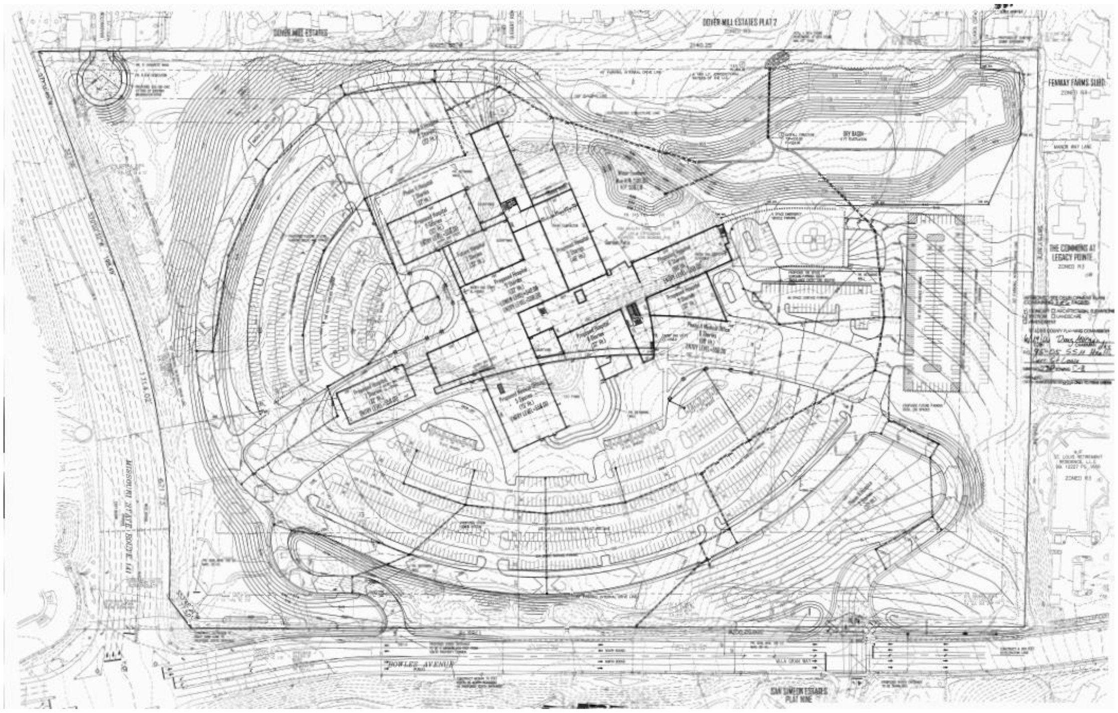


Figure 4: Building Orientation on Site

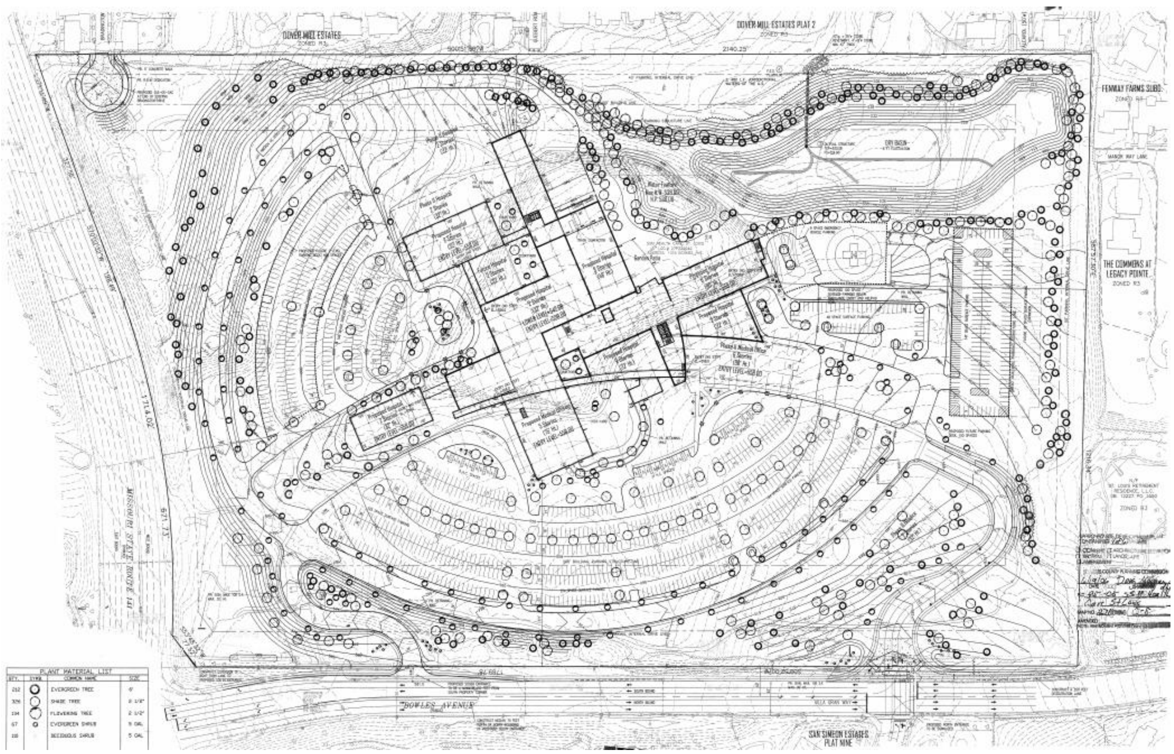


Figure 5: Building with Site Landscaping

1.4 List of Preparatory Documents

- SSM St. Clare Health Center Site Development Plan
    - Produced by Stock & Associates Consulting Engineers Inc.
- SSM St. Clare Health Center Replacement Hospital Project Manual
    - CP-11 E/T Document Issuance
- IBC
    - 2003 Edition (as reference)
    - 2012 Edition (for further design studies)
- ASCE 7
    - ASCE 7-02 (as reference)
    - ASCE 7-10 (for further design studies and load calculations)
- Vulcraft Steel Deck Catalogue, 2008 Edition
- AISC Steel Manual 14th Edition

2 GRAVITY LOADS

This section examines the dead, live, and snow loads used to design the building's gravity system. The original design calculations for gravity loads can be found in Appendix A. Dead loads are determined based on standard material weights, manufacturer data, and engineering experience. A full list of calculations for the values in Table 4 can be found in Appendix B. Future analyses of the building will focus on the bed tower. The majority of these loads are not present in the bed tower, but are listed here for comparison to the calculated loads, and as a reference.

2.1 DEAD AND LIVE LOADS

Table 1: Typical Live Loads

Live Load	Value (psf)	Code Minimum (psf)
Operating Room	60	60
Offices	50	50
Private Rooms	40	40
Corridors (1st Floor)	100	100
Corridors (other)	80	80
Stairs and Exits	100	100
Equipment Rooms	125	125

Table 2: Non-Typical Floor Dead Loads

Dead Load	Original Design Values (psf)	Thesis Calculated Values (psf)
Hospital Floor	60	64
Hospital Roof	78	70
Power Plant Roof	133	N/A
Penthouse Floor	60	N/A
Penthouse Roof	28	N/A
Rooftop Mech. Unit Supp.	75	N/A
Piping Zone	115	N/A
MRI Zone	78	N/A
Piping and MRI Zone	103	95
Exterior Brick Wall	50	51
Exterior Curtain Wall	20	N/A
Exterior Metal Panel	15	N/A

Note in Figure 6 on the next page that the bed tower's floor plan is congested with corridors. This means that conservatively, a live load of 80 psf can be assumed for the entire floor area unless a higher load occurs. The highest load to occur in the hospital outside of a corridor is an operating room with movable partitions; however, 60 psf + 20 psf returns the load conservatively to 80psf. The entire floor slab is the same 64 psf "Hospital Floor" assembly.

Figure 6: Architectural Plan of Bed Tower (typical 30'x30' bays in red)

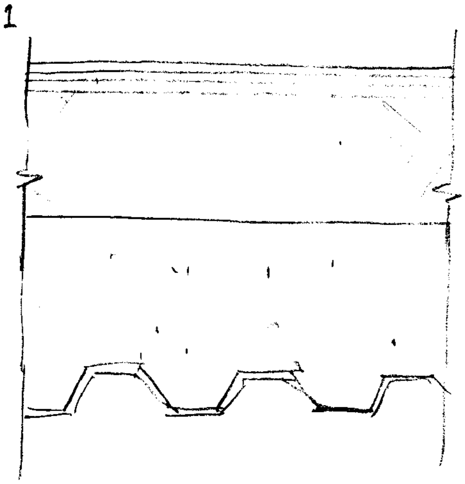
2.2 Snow Loads

The following section contains example calculations of snow loads and snow drift loads on SSM St. Clare Health Center.

Gravity Loads

ROOF CONSTRUCTION DEAD:
SEE A431

MBIT-1



PENTHOUSE ROOF

3PLY: 1 psf
5 3/4" Ins: 1.5(5.75) = 8.625
3.5" Conc: } 48 psf
3" Deck }
FRAMING: 6 psf
MISC: 6 psf
70 psf

BEAMS	26 × 4 × 30' =	3120
GIRDERS	76 × 1 × 30' =	2280
AREA		900 sf
		= 6 psf

LOW ROOF

3PLY: 1 psf
11" Ins: 1.5(11) = 16.5 psf
3.5" Conc: 48 psf
3" Deck:
FRAMING: 6 psf
MISC: 6 psf
78 psf

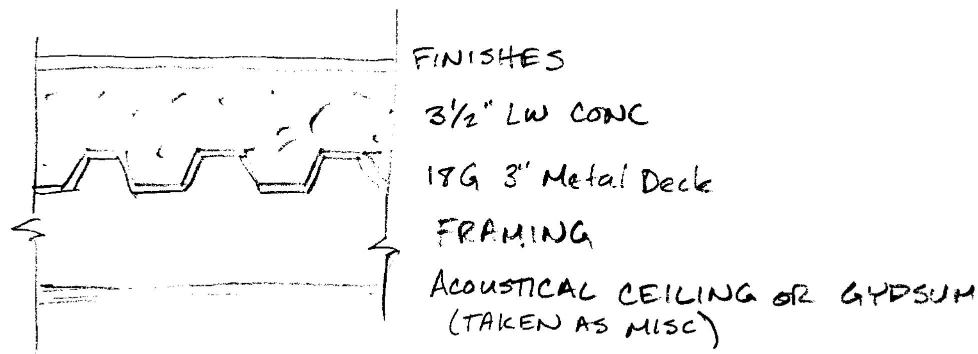
ROOF LIVE:

20 PSF (REQUIRED BY ASCE 7-10)

NOTE: LESS THAN SNOW LOAD

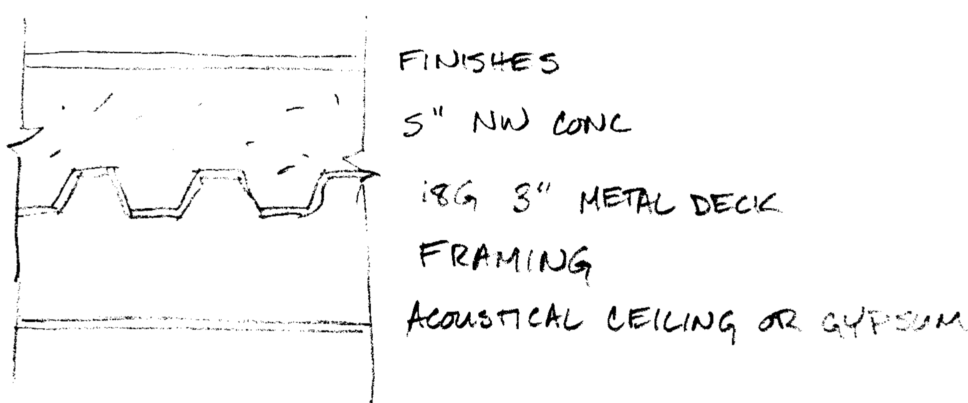
FLOOR CONSTRUCTION DEAD:

DECK 1:



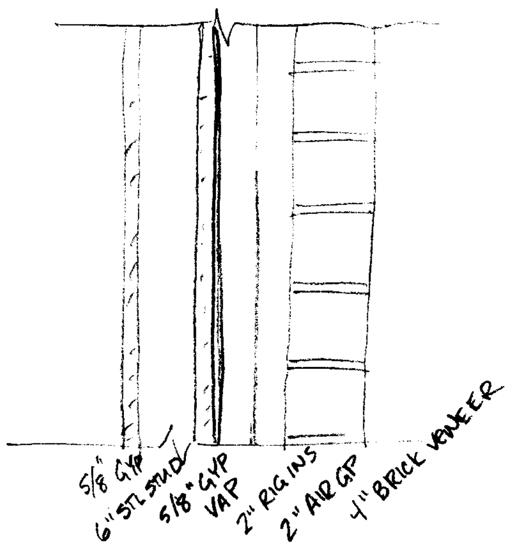
FINISHES:	2	psf
DECK:	48	psf
FRAMING:	7	psf
CEILING:	2	psf
MISC/EQUIP	5	psf
	64	psf

DECK 2:



FINISHES:	2	psf
DECK:	80	psf
FRAMING:	6	psf
CEILING:	2	psf
MISC/EQUIP:	5	psf
	95	psf

EXTERIOR ENCLOSURE



5/8" Gyp	:	2.75	psf
6" STL STUD	:	3	psf
5/8" GYP	:	2.75	psf
VAP	:	0.5	psf
2" RGD INS	:	3.0	psf
AIR	:	0	psf
BRICK	:	39	psf
		51	psf

Wall is supported at each floor by a steel angle. Vertical loads are transferred through the steel structure into the foundations.

2.2 Snow Loads

Snow Loads

- FLAT ROOF SNOW LOADS

$$p_f = 0.7\, C_e\, C_t\, I_s\, p_g$$

Terrain Category: B
C_e : 1.0
C_t : 1.0
I_s : 1.2 → OC IV
SNOW LOAD : 20 psf

$$p_f = 0.7(1.0)(1.0)(1.2)(20)$$
$$= 16.8 \text{ psf}$$

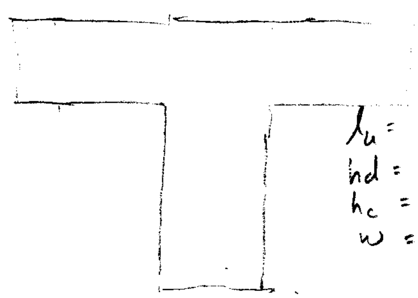
NOTE: RAIN ON SNOW SURCHARGE OF 5 PSF APPLIES.

$$p_m = I_s p_g \quad \text{for } p_g \le 20 \text{ psf}$$
$$= 24 \text{ psf}$$

- DRIFTS ON LOWER ROOFS

PENTHOUSE ROOF:

$\gamma = 0.13(20) + 14 = 16.6$ pcf
$h_b = 24/16.6 = 1.4457'$

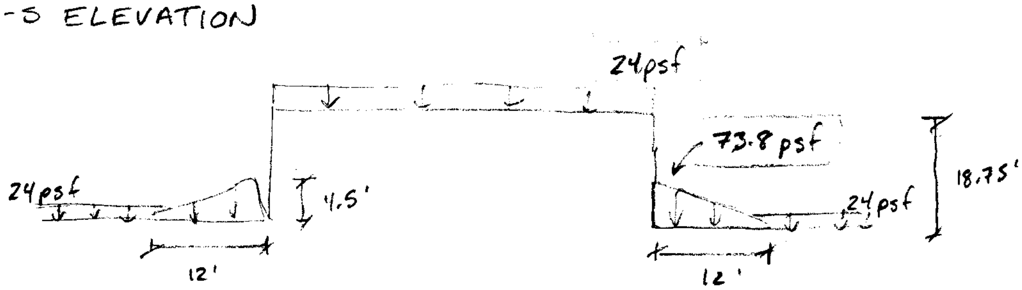


$l_u = 34.33' \sim 50'$
$h_d = 2'$
$h_c = 16.75'$
$w = 8$

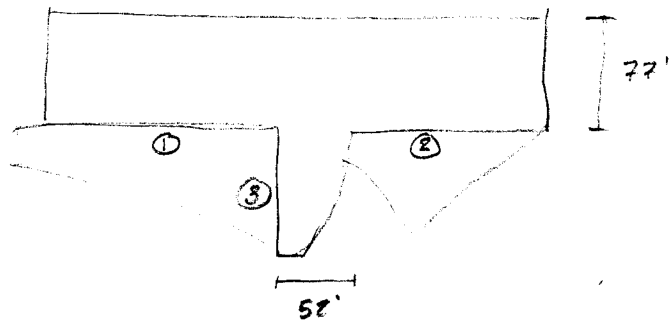
NOTE: N-S direction drifts truncated at 4'

$l_u = 97' \approx 100'$
$h_d = 3'$
$h_c = 16.75'$
$w = 12$

N-S ELEVATION



GARDEN LEVEL ROOFS:



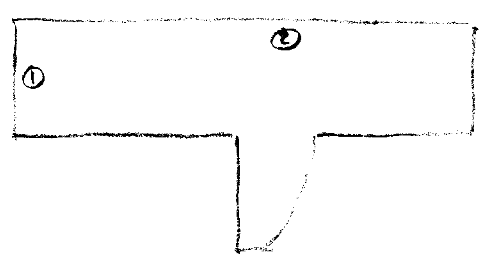
Area ① = Area ②

$l_u = 77' \sim 100'$
$h_d = 3'$
$h_c = 53'$
drift max $= \frac{24}{16.6}(16.6) + 3(16.6)$
$= 73.8$ psf

Area ③

$l_u = 52' \sim 50'$
$h_d = 2'$
$h_c = 54'$
drift max $= \frac{24}{16.6}(16.6) + 2(16.6)$
$= 57.2$ psf

PARAPET DRIFT



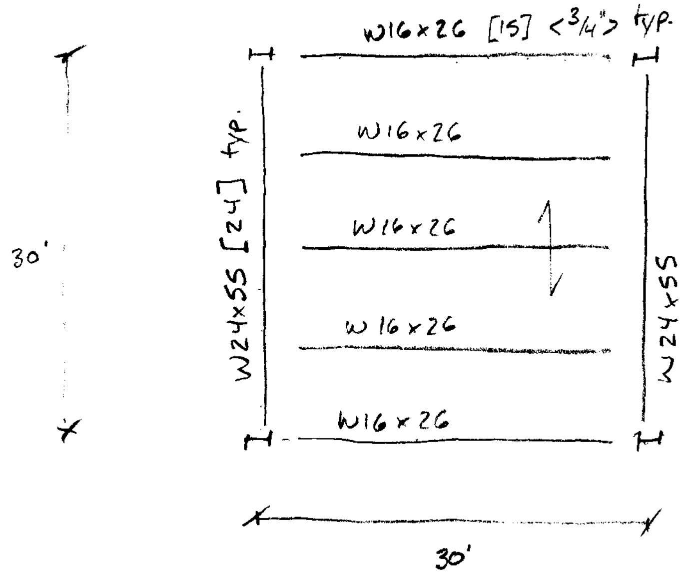
1) $l_u = 400$
$h_d = 6.5'$
$h_c = -4.3'$
drift max $= 24 + 6.5(0.75)16.6$
$= 104.93$ psf

2) $l_u = 100$
$h_d = 3'$
$h_c = -0.83'$
drift max $= 24 + 3(0.75)16.6$
$= 61.35$ psf

3 Typical Member Spot Checks for Gravity Loads

The following section contains spot checks for SSM St. Clare Health Center's original composite framing gravity system. The checks include deck unshored length and capacity, beam moment capacity and deflection limits, girder moment capacity and deflection limits, and column axial capacity (for an interior and exterior column).

GRAVITY LOAD "SPOT CHECKS"



TYPICAL BAY

DEAD LOADS: DECK 1

3" DECK w/ 3½" LW CONC	:	48	psf
BEAMS	:	5	psf
CEILING	:	2	psf
FINISHES	:	2	psf
MISC/EQUIP/MECH	:	5	psf
		62	

LIVE LOADS

PRIVATE ROOMS	:	40	psf
MOVABLE PARTITIONS	:	20	psf
CORRIDORS (ABOVE 1st)	:	80	psf

DECK CHECK

FROM VULCRAFT 2008 STEEL DECK CATALOGUE:

3VL118 w/ 3.5" LW CONC TOPPING

SPAN	SDI MAX UNSHORED
1	12'7"
2	14'9"
3	14'9"

7'6" < 12'7", any span condition could be used with this deck.

NOTE: 3VL122 COULD ALSO HAVE BEEN USED IN THIS SITUATION AS ITS MAX. UNSHORED SPAN FOR A SINGLE SPAN CONDITION IS 8'11"

7'6" < 8'11" ✓

LIVE LOADS (BEAMS):

- COMMON CASE: PATIENT ROOMS

40 psf	: patient room live
20 psf	: movable partitions
60 psf	

- COMMON CASE: UPPER FLOOR CORRIDORS

80 psf : upper corridor live (CONTROLS)

- LL REDUCTIONS

BEAMS

$$L_i = \max\begin{cases} 0.5(80) = 40 \text{ psf} \\ 80\left(0.25 + \frac{15}{\sqrt{15(30)}}\right) = 76 \text{ psf} \end{cases}$$

$15(30) = 450 > 400$ ✓

GIRDERS

$$L_i = \max\begin{cases} 0.5(80) = 40 \text{ psf} \\ 80\left(0.25 + \frac{15}{\sqrt{30 \times 60}}\right) = 48 \text{ psf} \end{cases}$$

$30(60) = 1800 > 400$ ✓

DEAD LOAD (BEAMS): PREVIOUS 64 psf - 2 psf (GIRDERS) = 60 psf

BEAM CHECK: COMPOSITE

- $1.2D + 1.6L = 1.2(62) + 1.6(76) = 196$ psf

  $196 \text{ psf}(7.5 \text{ ft}) = 1450$ plf

  $\frac{1.47(30)^2}{8} = 165.4$ k ft

- $1.4D = 1.4(62) = 86.8$ psf

  $(86.8 \text{ psf})(7.5 \text{ ft}) = 651$ plf

  $\frac{0.651(30)^2}{8} = 73.3$ k ft

- COMPOSITE ACTION ASSUMPTIONS

  ASSUME $a = 1''$

  $Y_2 = 6.5'' - 0.5'' = 6''$

- POSSIBLE SELECTIONS

(ORIGINAL) → W16×26	$\Sigma Q_n = 96$	$\frac{96}{17.2} = 5.58$	$2(6\times10) + 26(30) = 900$ lbs
✓ W14×22	$\Sigma Q_n = 81.1$	$\frac{81.1}{17.2} = 4.72$	$2(5\times10) + 22(30) = 760$ lbs
X W12×16	$\Sigma Q_n = 156$	$\frac{156}{17.2} = 9.06$	$2(10\times10) + 16(30) = 680$ lbs
X W12×19	$\Sigma Q_n = 104$	$\frac{104}{17.2} = 6.04$	$2(7\times10) + 19(30) = 710$ lbs

- UNSHORED STRENGTH

  $1.4(48)(7.5) + 1.4(26) = 540.4$ plf

  $1.2[(48)(7.5) + 26] + 1.6(20)(7.5) = 703.2$ plf

  $\frac{0.7032(30)^2}{8} = 79.1$ k ft

  CAPACITY = $166 > 79.1$ kft ✓

- WET CONCRETE DEFLECTION

$$W_{wc} = (48)(7.5) + 26 = 386 \text{ plf}$$

$$\Delta_{wc} = \frac{5(0.386)(30)^4(1728)}{384(29000)(301)}$$

$$= 0.806''$$

CAMBER 0.75" → 3/4"

- LIVE LOAD DEFLECTIONS

$$W_{LL} = (76)(7.5) = 750$$

$$I_{LB} = 596 \text{ in}^4 \quad \text{(TABLE 3-20)}$$

$$\Delta W_{LL} = \frac{5(0.75)(30)^4(1728)}{384(29000)(596)}$$

$$= 0.79'' < 1'' \quad ✓$$

- CHECK CONCLUSION:

DESIGNER SPECIFIED W16x26 [15] <3/4">. THE CALCULATIONS ABOVE CONFIRM THE NEED FOR A 0.75" CAMBER AND JUSTIFY THE USE OF A W16x26 SECTION TO MEET LL DEFLECTION CRITERIA.

15 STUDS WERE SPECIFIED.

CODE MAX SPACING = 32(3/4") = 24" , 8(6.5") = 52"

(3/4" = STUD SHANK DIA.)

$$\frac{30'}{15 \text{ studs}} = 2' \text{ per stud} \quad ✓$$

USE 15 STUDS SPACED EVENLY ACROSS BEAM EXCEEDS STRENGTH REQUIRED 2(6) = 12 STUDS.

CHECK a ASSUMPTION:

$$b_{eff} = \min \begin{Bmatrix} 3.75(12) \\ 30(12)/8 \end{Bmatrix} \times 2 = \begin{Bmatrix} 90'' \\ 90'' \end{Bmatrix} \checkmark$$

$$V_c = (3.5'')(90'')(3.5 \text{ ksi})(0.85)$$

$$= 937.2 \text{ kips}$$

$$V_s = (7.68 \text{ in}^2)(50 \text{ ksi})$$

$$= 384 \text{ kips}$$

15 studs (17.2 k/stud) = 258

$$a = \frac{258}{0.85(3.5)(90)}$$

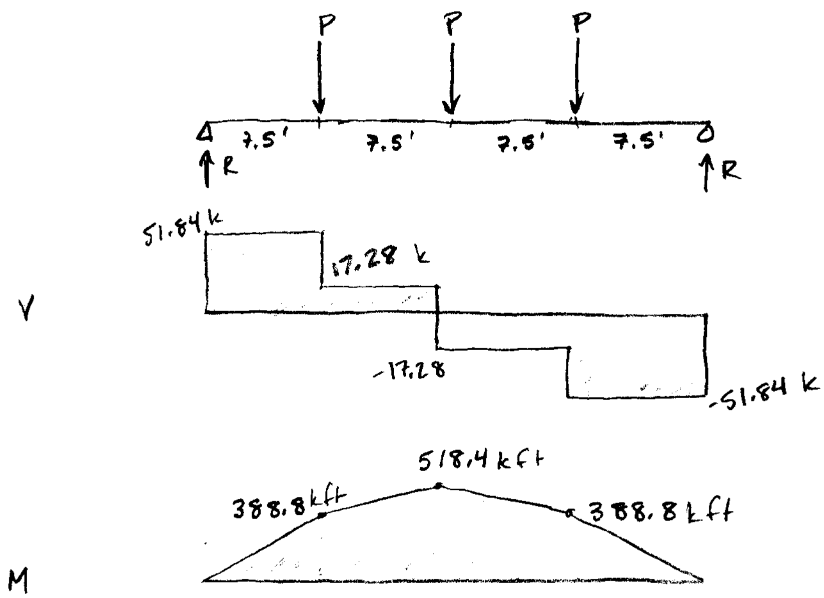
$$= 0.964 < 1.0 \checkmark$$

THIS BEAM IS <u>PARTIALLY COMPOSITE</u>.

$$\Sigma Q_n < V_s$$
$$258 < 384$$

GIRDER CHECK:

ASSUME BEAMS TAKE GRAVITY LOAD AND APPLY POINT LOADS TO GIRDERS.



DEAD LOAD

64 psf (from previous)

LIVE LOAD

48 psf (from previous)

POINT LOADS

$$P = [1.2(64) + 1.6(48)](7.5)(30)$$

$$= 34.56 \text{ k}$$

REACTIONS

$$R = \frac{3(P)}{2} = \frac{3(34.56)}{2} = 51.84 \text{ k}$$

CHECK ASSUMPTIONS

$a \approx 1''$

$y_2 = 6.5 - \frac{1.0}{2} = 6.0''$

3.5 ksi concrete

$\Sigma Q_n = 203 \qquad \frac{203}{17.2} = 11.8 = 12$

ECONOMY: $12(2)(10) + 55(30) = 1890$ lbs

GIRDER STRENGTH & SEVICEABILITY CHECKS

– CHECK UNSHORED STRENGTH

SUPERIMPOSE:

DISTRIBUTED MOMENT –

$1.4(55 \text{ lb}) = 77 \text{ lb}$

$1.2(55 \text{ lb}) + 1.6(0 \text{ lb}) = 66 \text{ lb}$

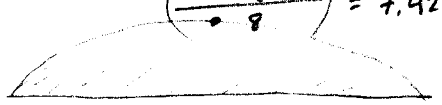
POINT LOAD MOMENT –

$1.2\left[(48(7.5)+26)(30)\right] + 1.6\left(20(7.5)(30)\right) = 21.096 \text{ k}$

$1.4\left[(48(7.5)+26)(30)\right] = 16.212 \text{ k}$

USE 1.2D + 1.6L CASE:

$\left(\frac{0.066(30)^2}{8}\right) = 7.425 \text{ kft}$



+

$\left(24.3(7.5) + 8.106(7.5)\right) = 243.18 \text{ kft}$

$7.425 + 243.18 = 250.6 \text{ kft}$

$574 \text{ kft} > 250.6 \text{ kft}$ ✓

- CHECK WET CONCRETE DEFLECTION

$P_{wc} = [48(7.5) + 26]\,30$

$= 11.58\ k$

$W_{wc} = 0.055\ k$

$\Delta_{wc} = 0.708'' < \frac{30(12)}{360} = 1''$ ✓

NO CAMBER USED.

- CHECK LIVE LOAD DEFLECTIONS

$P_{LL} = [48(7.5) + 26]\,30$

$= 11.58\ k$

$I_{LB} = 2380\ in^4$

$\Delta_{LL} = 0.387'' < \frac{30(12)}{360} = 1''$ ✓

- CHECK a ASSUMPTION

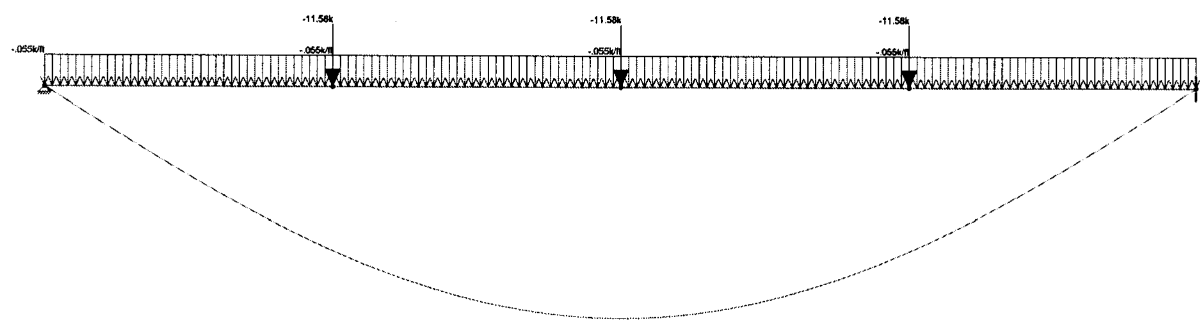
$b_{eff} = \begin{Bmatrix} \frac{1}{2}(30)(12) \\ 30(12)/8 \end{Bmatrix} \times 2 = \begin{Bmatrix} 360'' \\ 90'' \end{Bmatrix}$

$V_c = 0.85(3.5)(90)(3.5) = 937.2\ k$

$V_s = 16.2(50) = 810\ k$

$a = \frac{203}{0.85(90)(3.5)}$

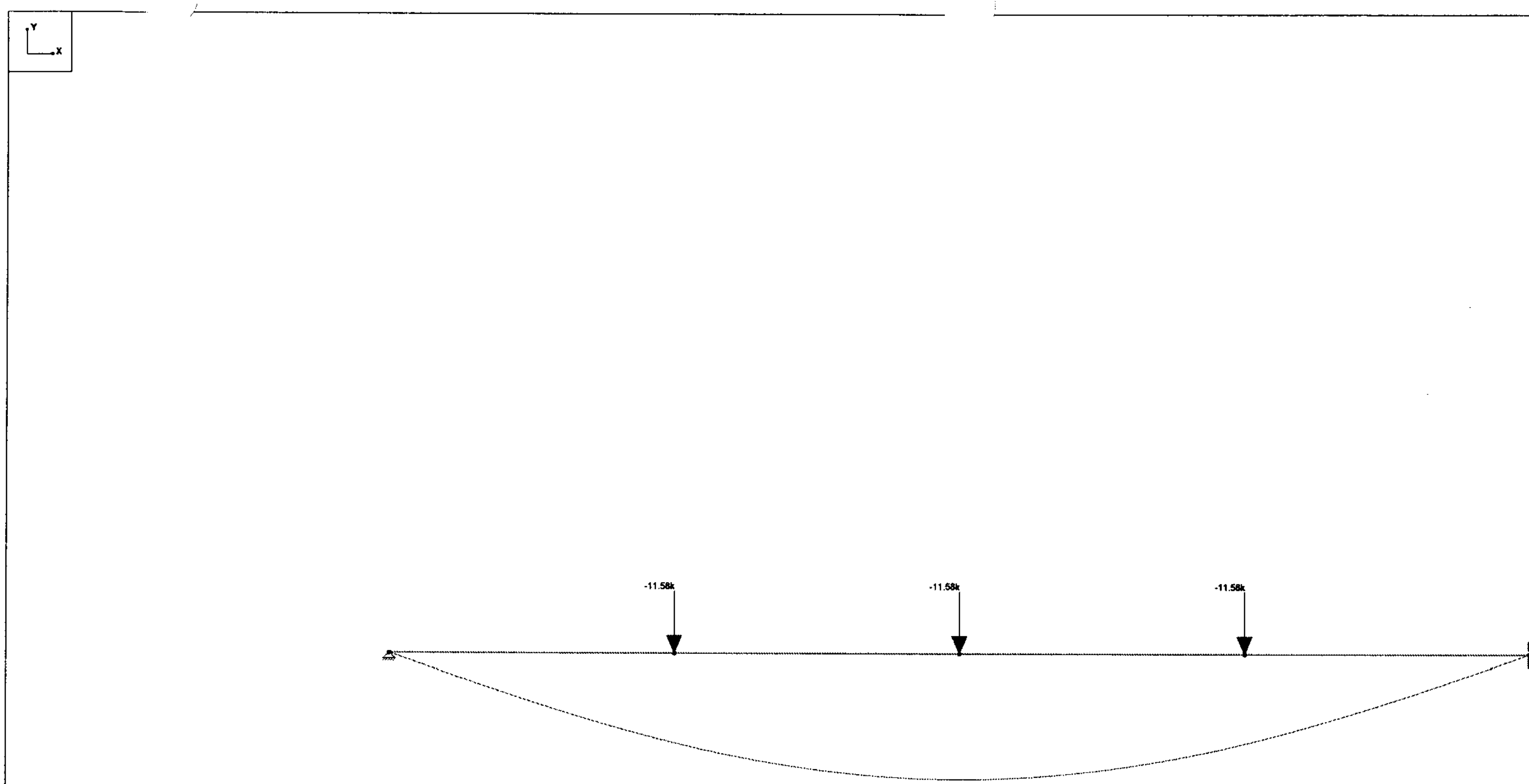
$= 0.758'' < 1.0''$ ✓



	Deflection Calculation	October 9, 2014
Brandmeier	WET CONCRETE DEFLECTION	11:10 PM
		Girder Deflection Check.r2e

Joint Displacements WET CONCRETE

Joint Label	X Translation (in)	Y Translation (in)	Rotation (radians)
N1	0	0	-6.217e-3
N2	0	-.503	-4.349e-3
N3	0	-.708	0
N4	0	-.503	4.349e-3
N5	0	0	6.217e-3



	Deflection Calculation	October 9, 2014
Brandmeier	LIVE LOAD	11:24 PM
		Girder Deflection Check.r2e

Joint Displacements

LIVE LOAD

Joint Label	X Translation (in)	Y Translation (in)	Rotation (radians)
N1	0	0	-3.397e-3
N2	0	-.275	-2.378e-3
N3	0	-.387	0
N4	0	-.275	2.378e-3
N5	0	0	3.397e-3

CHECK CONCLUSION:

DESIGNER SPECIFIED W24 × 55 MEETS ALL STRENGTH AND DEFLECTION CHECKS WITH SOME ADDITIONAL CAPACITY

24 STUDS IS ACCURATE AND MEETS MAX SPACING REQUIREMENTS.

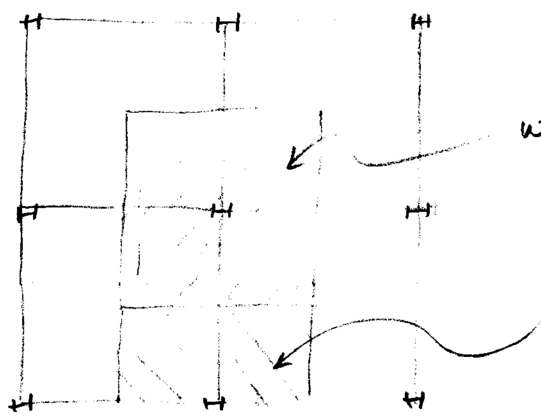
$$\begin{cases} 32(3/4'') = 24'' \\ 8(6.5'') = 52'' \end{cases}$$

$$\frac{30(12)}{24} = 15''$$

$$15'' < 24'' \quad \checkmark$$

COLUMN CAPACITY CHECK

Typical bay: INTERIOR COLUMN



	DEAD (psf)	LIVE (psf)	AREA (sf)
PENT.	70	24 (snow)	450
6	64	80	900
5	64	80	900
4	64	80	900
3	64	80	900
2	64	80	900
1	64	80	900

LL REDUCTION

NOTE: NO LIVE LOAD REDUCTION PERMITTED ON ROOF AND ROOF NOT INCLUDED IN INFLUENCE AREA.

$$L = 80 \max \left\{ \begin{array}{l} 0.5 \\ 0.25 + \frac{15}{\sqrt{900(6)}} \end{array} \right\} = 40 \text{ psf}$$

AXIAL LOAD

$$P = \left[1.2(64) + 1.6(40)\right](900)(6) + \left[1.2(70) + 1.6(24)\right](450)$$

$$= 815.4 \text{ kips}$$

W14x90 column used: $\phi M_p = 979\ k > 815\ k$ ✓

Typical Bay: EXTERIOR COLUMN

	DEAD (psf)	LIVE (psf)	T AREA (ft²)	FACADE (psf)	T FAC AREA (ft²)
PENT	70	24	0	-	0
6	64	80	450	43.2	0
5	64	80	450	43.2	420
4	64	80	450	43.2	420
3	64	80	450	43.2	420
2	64	80	450	43.2	420
1	64	80	450	43.2	480

Glazing/Glass assembly dead load: 25 psf

Fenestrated area of walls ≈ 30%

$$51(.70) + 25(.30) = 43.2$$

Facade load rests on steel angle at story below

$$LL = 80 \max \begin{Bmatrix} 0.5 \\ 0.25 + \frac{15}{\sqrt{450(6)}} \end{Bmatrix} = 80 \begin{Bmatrix} 0.5 \\ 0.53 \end{Bmatrix} = 43.1 \text{ psf}$$

$$P = (1.2(64) + 1.6(43.1))(450)(6) + [(43.2)(420)(4) + 43.2(480)]1.2$$

$$= 393552 + 87091.2 + 24883.2$$

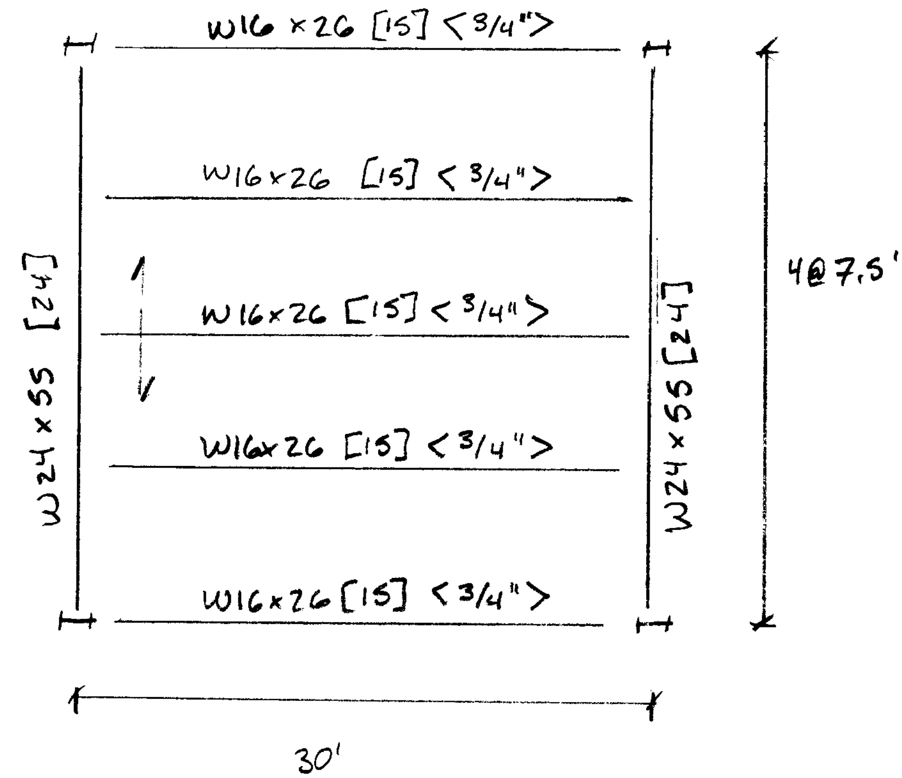
$$= 505526.4 \text{ lbs}$$

$$= 505.5\ k$$

W14x61, $\phi M_p = 514\ k > 505.5\ k$ ✓

SUMMARY DIAGRAM

COMPOSITE STEEL FRAMING



DECK: 3", 18G DECK W/ 3½" LW CONC TOPPING
2 HR FIRE RATING
TYP.

- 50 ksi steel
- 3500 psi concrete

4 Alternative Framing Systems for Gravity Loads

The following section examines four alternate framing systems to determine the best option for further study in a system redesign. The assumptions and evaluation criteria for the study will be presented first, followed by the calculations as supporting data.

4.1 Assumptions

The following design calculations are based on an analysis of purely vertical dead and live loads using ultimate strength design. Where possible, design aids such as Risa 2D and the concrete CRSI Tables have been used for simplified analysis and initial member sizing. All calculations are conducted on an interior bay that is 30'x30'. The analysis for concrete assumes that all criteria for Direct Design Method have been met. With the exception of member spot checks for the original composite steel gravity system, column design was not considered.

4.2 System Comparison and Decision Matrix

Tables 3 and 4 present the findings from evaluating the original composite steel framing, non-composite steel framing, 2 way flat plate concrete slab, 1 way concrete slab with intermediate beams, and 1 way concrete slab with a full span. Table 4 specifically uses weighted criteria to evaluate the overall effectiveness of each system. The values range from –2 to 2 depending on the performance of the system in the given category: 2 being the best performance and -2 the poorest performance. Section 5, the conclusion, contains a further discussion of the findings in this report and recommendations for a possible lateral system design.

The environmental impact category is based on the lifescycle net carbon emissions for an assembly in pounds of CO2 per pound of material. This category is important as more clients in the industry seek to achieve high LEED certifications.

Table 3: Values for Evaluation Criteria

Criteria	Composite Steel Framing	Non-Composite Steel Framing	2 Way Flat Plate Slab	1 Way Slab with Intermediate	1 Way Slab
Weight (psf)	53.5	49.5	124.4	127.4	165.3
Depth	24"	24"	10"	24"	24"
Cost	$14.25 / SF	$13.43 / SF	$11.25 / SF	$13.67 / SF	$11.72 / SF
Fire Protection	None	None	None	None	None
Fire Rating	2 Hr	2 Hr	4 Hr	4 Hr	4 Hr
Environmental Impact (lbCO2/lb)	9107.6	8744.7	6209.7	6349.6	8239.7

Table 4: Weighted Decision Matrix

Criteria	Importance Factor	Composite Steel Framing	Non-Composite Steel Framing	2 Way Flat Plate Slab	1 Way Slab with Intermediate	1 Way Slab
Cost	1.50	-1	0	1	0	1
Environmental Impact	1.50	-1	-1	1	1	1
Constructibility	1.00	2	2	2	0	0
Durability	1.00	1	1	1	1	1
Fire Resistivity	1.00	1	1	2	2	2
Weight	0.75	2	2	1	1	0
Vibration Susceptibility	0.75	-1	0	1	1	1
Detailing Intensity (Seismic)	0.50	1	1	-2	-2	-2
		2.25	4.5	8.5	5	5.75

4.3 Alternate Gravity System Calculations

NOTE ON FORMATTING! Each alternate system contains the same order of calculations that follow the natural load path of the structure:

1. Drawing or Layout of Typical Bay
2. Determination of Loads
3. Slab/Deck Design/Check
4. Beam Design/Check
5. Girder Design/Check
6. Software Output (where applicable)
7. Design Summary

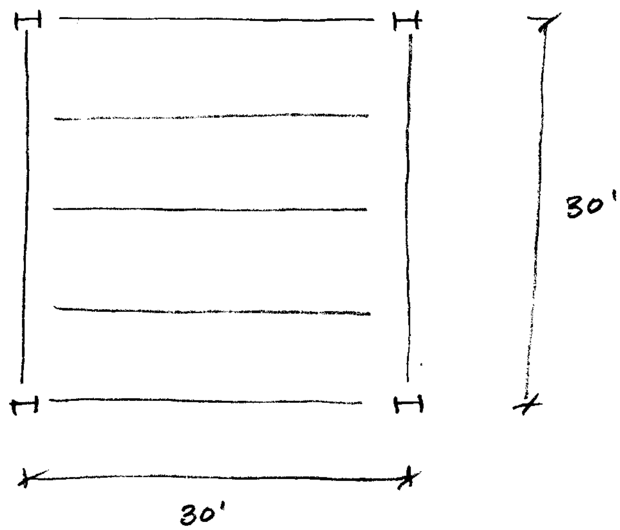
The final pages in this section are calculations comparing the five systems. The information presented on the final calculation pages has been summarized in Table 3 and Table 4.

ALTERNATE DESIGN:

NON-COMPOSITE STEEL

PURPOSE: COMPARISON OF ECONOMY BETWEEN COMPOSITE AND NON-COMPOSITE BAY.

KEEP BEAM CONFIGURATION THE SAME. ACHIEVE SAME FIRE RATING WITH STEEL DECK ASSEMBLY.



ORIGINAL ASSEMBLY: 3VLI w/ 3 1/2" LW CONC ACHIEVES A 2 HR. FIRE RATING.

USE VULCRAFT 2C22 CONFORM DECK W/ 6x6 - W2.9xW2.9 WWF AND 3.5" LW CONC TOPPING. 2 Hr FIRE RATING

WEIGHT OF ASSEMBLY: 43 lbs

DEAD LOAD

DECK ASSEMBLY	43
BEAMS	5
GIRDERS	2
FINISHES	2
MEP/EQUIP/MISC	5

57 psf girders
55 psf beams

LIVE LOAD

PATIENT ROOM : 40 psf
PARTITION : 20 psf

CORRIDOR : 80 psf ← CONTROLS

REDUCTIONS: SAME AS PREVIOUS SYSTEM

76 psf Beam
48 psf Girder

BEAM DESIGN

FIND M_u:

$1.2(55) + 1.6(76) = 187.6$ psf
$1.4(55) = 77$ psf

$187.6 \text{ psf } (7.5 \text{ ft}) = 577.5 \text{ plf}$

$$\frac{0.5775(30)^2}{8} = 64.968 \text{ kft}$$

$M_u = 0.9 M_n$

$$\frac{65}{0.9} = 72.2 \text{ k ft}$$

TRY W14×22 (TABLE 3-2)

$\phi M_{px} = 125$

- CHECK LL DEFLECTIONS

DISTRIBUTED LOAD:

$W_{LL} = 76(7.5) = 570$ plf

$$\Delta_{LL} = \frac{5(0.57)(30)^4(1728)}{384(29000)(199)}$$

$= 1.8" > 1.0$ FAILS $\frac{l}{360} = \frac{30(12)}{360} = 1.0"$

TRY W16×26

$$\Delta_{LL} = \frac{5(0.57)(30)^4(1728)}{384(29000)(301)}$$

$= 1.19" > 1.0$ FAILS

TRY W16×31

$$\Delta_{LL} = \frac{5(0.57)(30)^4(1728)}{384(29000)(375)}$$

$= 0.955" < 1.0$ PASSES

- CHECK TOTAL LOAD

$W_{TL} = (55 + 76)(7.5) = 982.5$

$$\Delta_{TL} = \frac{5(0.98)(30)^4(1728)}{384(29000)(375)}$$

$= 1.64" > 1.5"$ FAILS $= \frac{l}{240}$ $\frac{30(12)}{240} = 1.5"$

TRY W18×35

$$\Delta_{LL} = \frac{5(0.57)(30)^4(1728)}{384(29000)(510)}$$

$= 0.702" < 1.0"$ PASSES ✓

$$\Delta_{TL} = \frac{5(0.99)(30)^4(1728)}{384(29000)(510)}$$

$= 1.229" < 1.5"$ PASSES ✓

– WEIGHT CHECK

$$\frac{35}{7.5} = 4.667 \text{ psf} < 5 \text{ psf} \checkmark$$

SUMMARY

USE W18×35 BEAMS

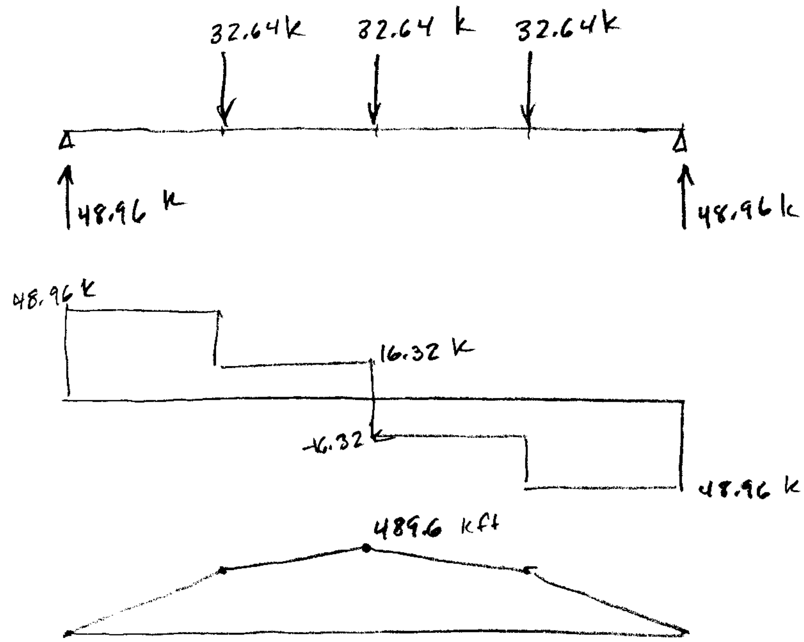
GIRDER DESIGN

POINT LOAD VALUES:

$P_{LL} = 48(7.5)(30) = 10.8\ k$

$P_{DL} = 57(7.5)(30) = 12.8\ k$
$55(7.5)(30) = 12.4\ k$

$M_u = 1.2(12.8) + 1.6(10.8) = 32.64\ k$



USE W24x55 : $M_{px} = 503$ kf > 489.6 kft

CHECK WET CONC. DEFLECTION:

$\Delta_{wc} = 0.76'' < 1.0''$

SEE RISA MODEL

CHECK LL DEFLECTION

$\Delta_{LL} = 0.64'' < 1.0''$

SEE RISA MODEL

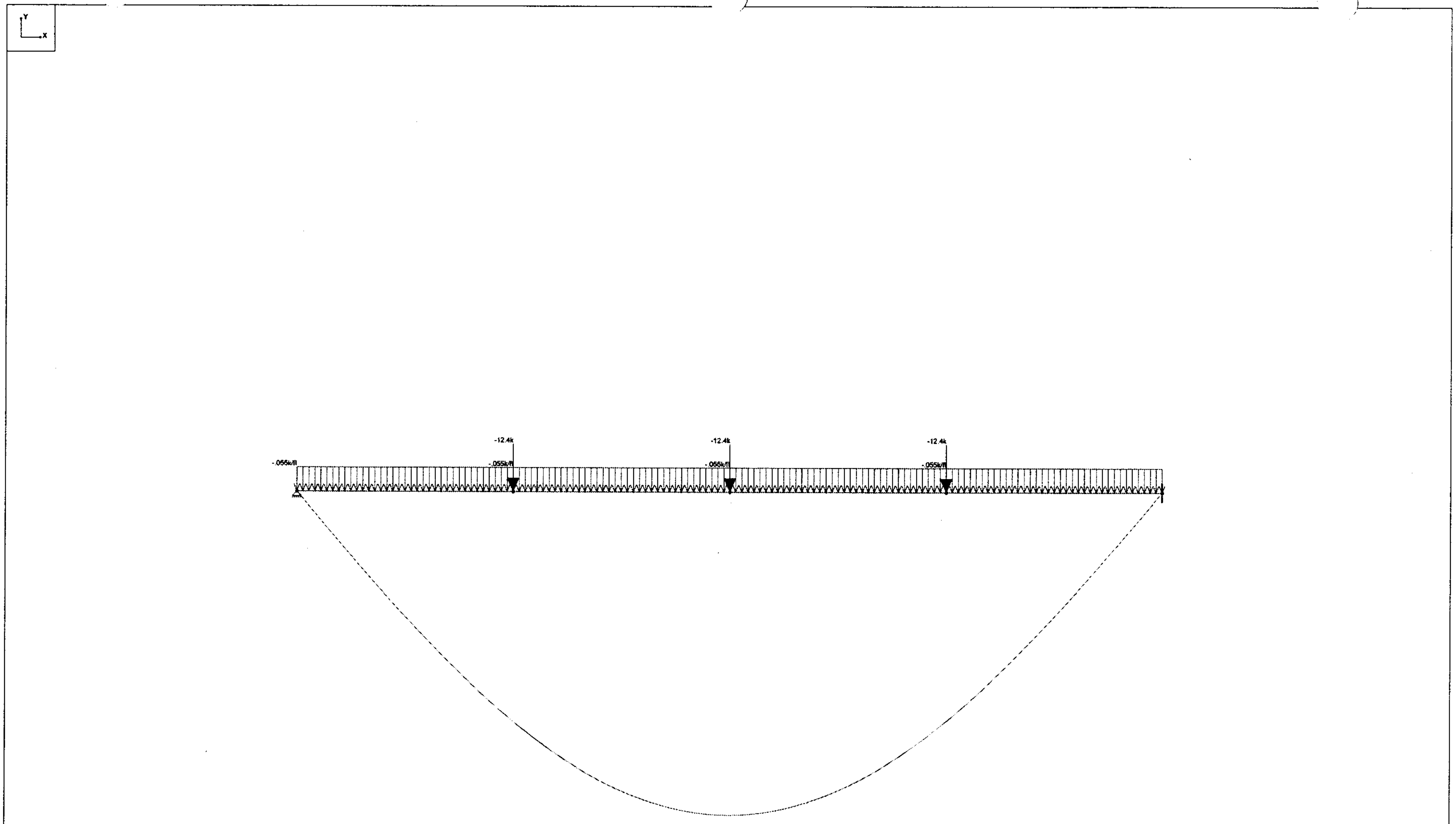
CHECK TOTAL LOAD DEFLECTIONS:

$P_{TL} = 10.8\text{ k} + 12.4\text{ k} = 23.2\text{ k}$

$\Delta_{TL} = 1.39'' < 1.5''$ PASSES!

Summary:

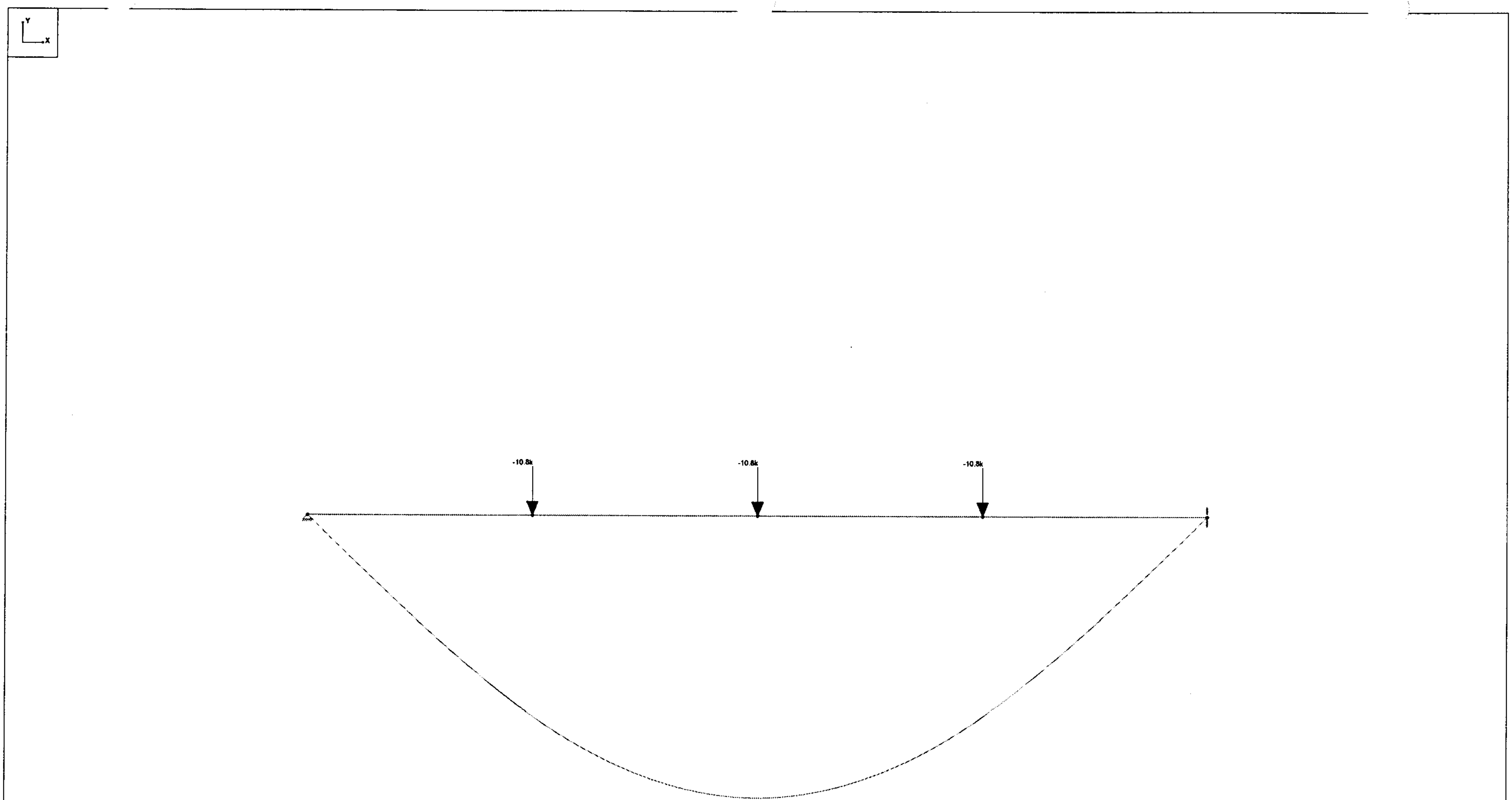
USE W24x55 GIRDERS w/ NO CAMBER



	Girder Design	October 12, 2014
Brandmeier	WET CONCRETE DEFLECTION	10:45 PM
		untitled.r2e

Joint Displacements WET CONCRETE

Joint Label	X Translation (in)	Y Translation (in)	Rotation (radians)
N1	0	-.538	-4.646e-3
N2	0	-.757	0
N3	0	-.538	4.646e-3
N4	0	0	6.641e-3
N5	0	0	-6.641e-3



	Girder Design	October 12, 2014
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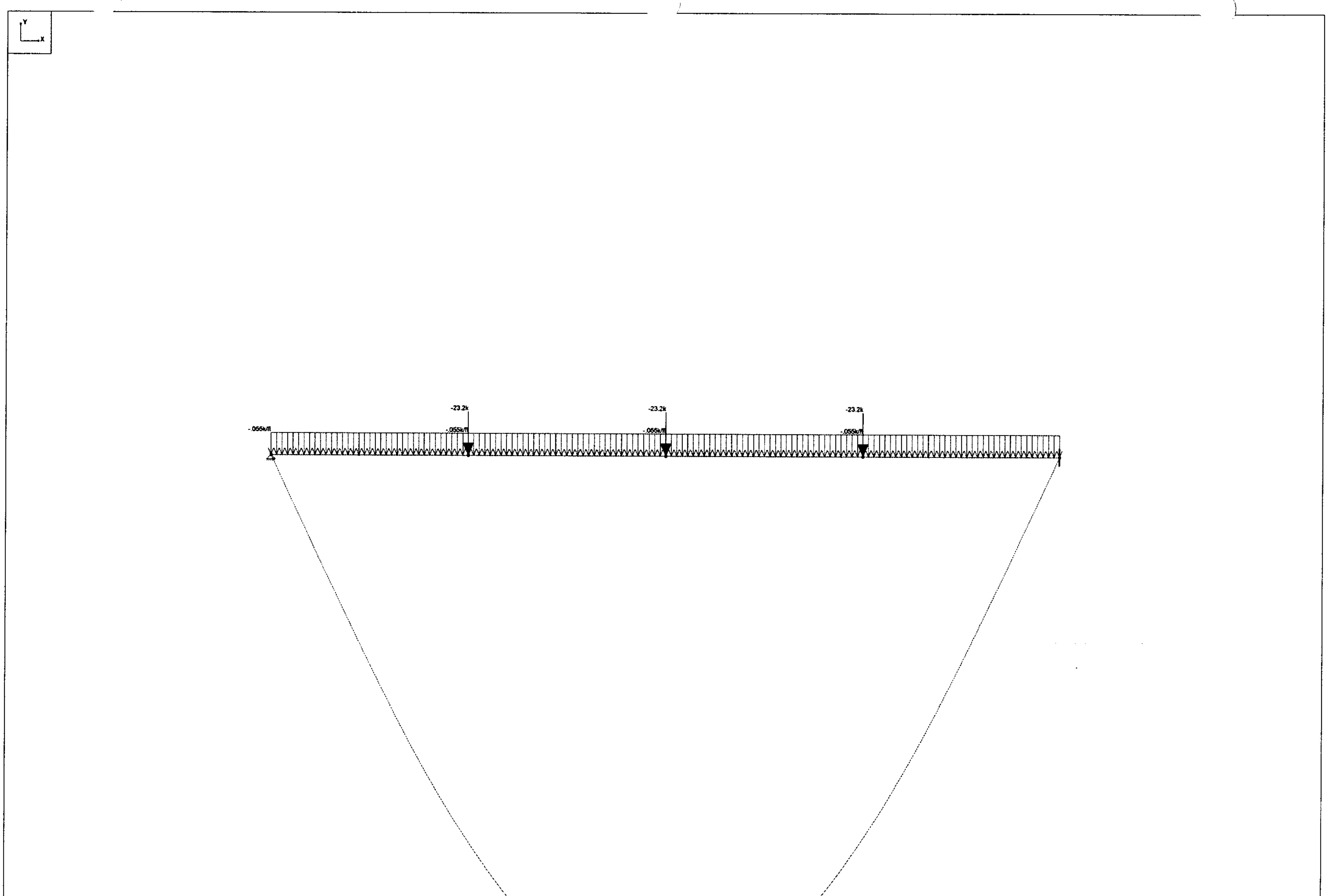
Joint Displacements LIVE LOAD

Joint Label	X Translation (in)	Y Translation (in)	Rotation (radians)
N1	0	-.452	-3.91e-3
N2	0	-.637	0
N3	0	-.452	3.91e-3
N4	0	0	5.586e-3
N5	0	0	-5.586e-3



	Girder Design	October 12, 2014
Brandmeier	TOTAL LOAD DEFLECTIONS	11:08 PM
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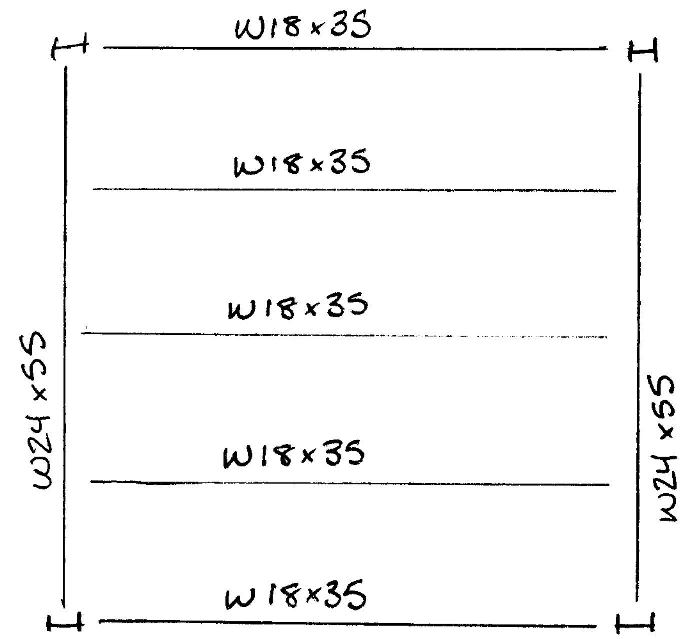
Joint Displacements TOTAL LOAD

Joint Label	X Translation (in)	Y Translation (in)	Rotation (radians)
N1	0	-.99	-8.556e-3
N2	0	-1.394	0
N3	0	-.99	8.556e-3
N4	0	0	1.223e-2
N5	0	0	-1.223e-2

SUMMARY DIAGRAM

NON-COMPOSITE STEEL



DECK: 3VLI 2C22 DECK w/ 3½" LW CONC
2 Hr FIRE RATING
TYP

50 ksi steel
3500 psi concrete (LW)

2 Way Slab System:

Limitations

1. ≥ 3 continuous spans ✓
2. rectangular w/ $\frac{l}{b}$ ratio < 2 ✓
3. span lengths vary < $\frac{1}{3}l$ ✓
4. columns offset < 10% ✓
5. service LL < 2x(service DL) ✓
6. stiffnesses meet ACI 318-11, 13.6.1.6 ✓
7. redistribution of moments by 8.4 N.P. ✓

CRSI RECOMMENDATIONS (p. 9-35)

Span col-col = 30'

Superimposed Factored load = 138.8 (from below)

Min. Col = 41"

$f_y = 60$ ksi
$f'_c = 4$ ksi

DEAD LOAD = $150\left(\frac{10}{12"}\right) = 125$ psf

LIVE LOAD = USE FULL LIVE LOAD B/C DEFLECTION IS NOT AN ISSUE
= 80 psf

Superimposed Dead =
Finishes: 2 psf
MEP: 5 psf
Ceil/Misc: 2 psf
9 psf

Factored Superimposed total = $1.2(9) + 1.6(80)$
= 138.8 psf

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARE
3-0137 — 200 SHEETS — FILLER
COMET

Min Panel thickness (ACI TABLE 9.5(c))

w/o drop panels, interior panels $= \frac{l_n}{33}$

w/o drop panels, exterior w/ edge beams $= \frac{l_n}{33}$

$l_n = 30(12) - 41$
$= 319$ in

$\frac{l_n}{33} = \frac{319}{33} = 9.666 \sim 10$ in

Find Shear Strength

TOTAL FACTORED LOAD $= 1.2(125) + 138.8$
$= 288.8$

ONE WAY

$V_u = 0.2888\left(\frac{30}{2} - \frac{36}{2(12)} - \frac{8}{2(12)}\right)30 = 114.1$ k

$d = \frac{(10-1.5)+(10-2.5)}{2} = 8''$

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

$= \frac{2(1)\sqrt{4000}\,(30\times12)(8)}{1000}$

$= 364.3$

$\phi V_c = 0.75(364.3) = 273.2$ k $> V_u = 114.1$ k

2 WAY

$V_u = 0.2888\left((30\times30) - \left(\frac{36+8}{12}\right)^2\right)$
$= 256.1$ k

$V_c = 4\lambda\sqrt{f'c}\, b_o d$
$= \frac{4(1)\sqrt{4000}\,(4(41+8))(8)}{1000}$
$= 396.7$ k

$\phi V_c = 0.75(396.75)$
$= 297.5$ k

297.5 k $> V_u = 256.1$ k

FIND TOTAL FACTORED MOMENT

$$M_o = \frac{q_u l_2 l_n^2}{8}$$

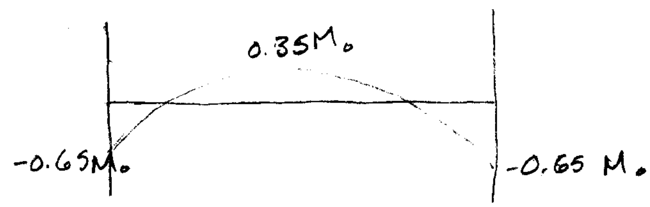
$$= \frac{0.2888(30)\left(30 - \frac{36}{12}\right)^2}{8}$$

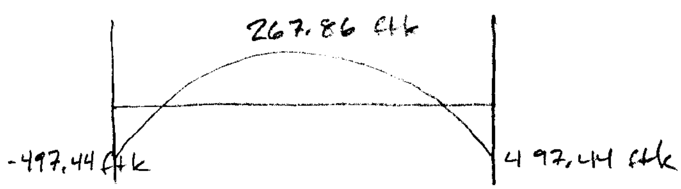
$$= 789.5 \text{ ftk}$$

DISTRIBUTE MOMENTS:

Interior bay, no beams

int. − fact. = 0.65
+ fact = 0.35
ext − fact = N/A





$\alpha_{f_1} = 0$

TO COLUMN + MIDDLE STRIPS:

COL (7.5')	MID (15')	COL (7.5')
−373.1 (0.75)	−124.4 (0.25)	−373.1 kft
160.7 (0.60)	107.1 (0.40)	160.7 kft
−373.1	−124.4	−373.1 kft

Design Flexural Reinforcement.

Calculate d

top : $d \approx h - 1.7 = 8.3$ ← use worst case.
bot : $d \approx h - 1.1 = 8.9$

$A_{s\,min} = 0.0018bh$
$= 0.0018(15 \times 12)(10)$
$= 3.24\ in^2$

$S_{max} = \min\begin{Bmatrix} 2h \\ 18" \end{Bmatrix} = \min\begin{Bmatrix} 20" \\ 18" \end{Bmatrix}$

$A_s = \dfrac{M_u}{\phi f_y jd}$ $jd = 0.95(8.3)$

	M_u	A_s req (in^2)	CRSI	CHECK A_s	CHECK S_{max}
− col	373.1	10.52	15 #8	11.85	~12"
+ col	160.7	4.53	11 #6	4.84	~16.4"
− mid	124.4	3.50	12 #5	3.72	~15"
+ mid	107.1	3.02	11 #5	3.41	~16.4"

Strain Check

$\varepsilon_s = \dfrac{d-c}{c}\varepsilon_{cu}$

$= \dfrac{8.3-1.367}{1.367}(0.003)$

$= 0.015 > 0.00207$
> 0.005

$a = \dfrac{11.85(60)}{0.85(4)(15\times12)}$

$= 1.162"$

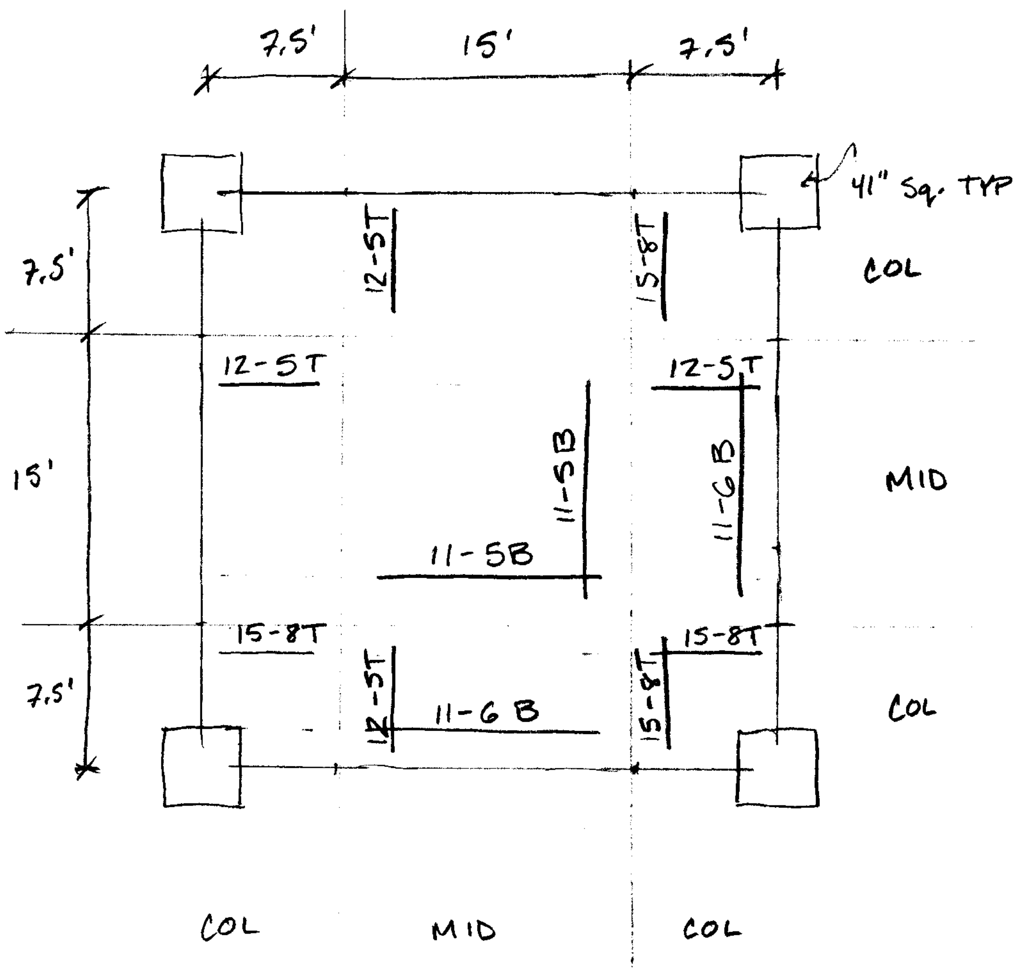
$c = \dfrac{1.162}{0.85} = 1.3667$

$\phi = 0.9$

	ε_s	ϕ
− col	0.015	0.9
+ col	0.041	0.9
− mid	0.055	0.9
+ mid	0.06	0.9

SUMMARY DIAGRAM

2-WAY FLAT PLATE CONCRETE SLAB



NOTE: All reinforcement sizing was evaluated based on CRSI TABLES

SPAN: 30' x 30'
SUPERIMPOSED LOAD: 150 psf

4000 psi NW concrete
60,000 psi steel

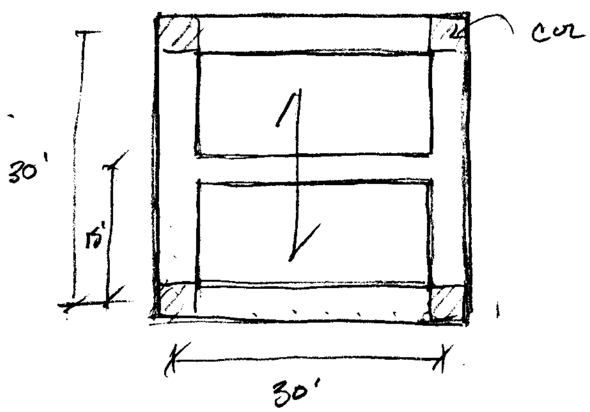
One-Way Slab w/ EDGE BEAMS

Minimum thickness Table 9.5 (a)

both ends continuous : $\frac{l}{28} = \frac{30(12)}{28} = 12.85 \rightarrow 13''$

one end continuous : $\frac{l}{24} = \frac{30(12)}{24} = 15''$

TOO LARGE. TRY DIFFERENT CONFIG.



Minimum thickness Table 9.5 (a)

one end continuous = $\frac{l}{24} = \frac{15(12)}{24} = \boxed{7.5''}$

Minimum clear cover = 3/4"

Dead load = $150\left(\frac{7.5}{12}\right) + 9\ (\text{misc}) = 102$ psf

LL = 80 psf

Total factored load = 1.2 (102 psf) + 1.6 (80 psf)

= <u>251.3 psf</u>

= 1.4 (102 psf)

= 143.85 psf

$$M_u = \frac{\frac{(1')(251\ psf)}{1000}(15)^2}{8}$$

$$= 7.067\ ft\,k$$

$$A_{s\,req} = \frac{M_u}{\phi f_y\, jd}$$

$$= \frac{7.1\ ftk\,(12)}{0.9(0.95)(60)(5)}$$

$$= 0.33$$

Use (2) #4 bar, $A_s = 0.40\ in^2$ @ 6" o.c.

- $S_{max} = \min\begin{Bmatrix} 2h \\ 18" \end{Bmatrix} = \begin{Bmatrix} 2(7.5) \\ 18" \end{Bmatrix} = \begin{Bmatrix} 15" \\ 18" \end{Bmatrix}$ ✓

- $A_{s\,min} = 0.002(b_w)d$

$$= 0.002(1\ ft)(7.5 - 1)$$

$$= 0.013$$ ✓

- Strain

$$\varepsilon_s = \frac{d-c}{c}\varepsilon_{cu}$$

$$= \frac{5 - 0.69}{0.69}(0.003)$$

$$= 0.214 > 0.005 > 0.00207$$

$$a = \frac{0.4(60)}{0.85(4)(1)(12)}$$

$$= 0.59"$$

$$c = \frac{0.59}{0.85}$$

$$= 0.69"$$

TENSION CONTROLLED, $\phi = 0.9$ ✓

- $\phi M_n = 0.9\left(0.4(60)(5 - 0.69)/12\right)$

$$= 7.758\ kft > 7.067$$ ✓

<u>Thermal / Cracking Reinforcement</u>

$0.013\ in^2$ → use (1) #4 @ 12" o.c.

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Beam Design

TABLE 9.5(a)

Beam both ends continuous : $\frac{l}{21}$ $\frac{30(12)}{21} = 17.14"$

$$W_u = \left[1.2\left(150\left(\tfrac{7.5}{12}\right) + 9\right) + 1.6(80)\right]15$$
$$= (123.3 + 128)\,15$$
$$= (251.3)(15)$$
$$= 3.77 \text{ k/ft}$$

$$M_u = \frac{w_u l_n^2}{8}$$
$$= \frac{3.77\left(30 - \frac{18"}{12}\right)^2}{8}$$
$$= 382.77$$

– Size estimate

$bd^2 = 20M_u$

$d = 20"$

$b = 19.14 \sim 20"$

$h = d + 2.5$

$h = 22.5$
$= 24"$

– Check self weight

$bd^2 = 8000$

$$W_{sw} = \frac{20(24)}{144}(150) = 500 \text{ plf}$$

$$W_n = 3.77 + \frac{500(1.2)}{1000} = 4.37$$

$$M_u = \frac{4.37(28.5)^2}{8} = 443.69$$

$20(443.69) = 8873.8 > 8000$

$M_u = 443.69$ kft

$$A_{s\,req} = \frac{M_u}{\phi f_y jd}$$

$$= \frac{444(12)}{0.9(60)(0.95)(20)}$$

$$= 5.193 \text{ in}^2$$

Try 6 # 9

$$A_s = 6(1.0) = 6.0 \text{ in}^2 > 5.193 \checkmark$$

$$\text{Spacing} = \frac{20 - 2(1.5) - 0.5(2) - 6(1.128)}{5}$$

$$= 1.846'' > 1.128'' > 0.75'' \checkmark$$

$$a = \frac{60(6)}{0.85(4)(20)}$$

$$= 5.294$$

$$c = \frac{5.294}{0.85}$$

$$= 6.23$$

$$d = 24 - 1.5 - 0.5 - \frac{1.128}{2}$$

$$= 21.436'' > 20'' \checkmark$$

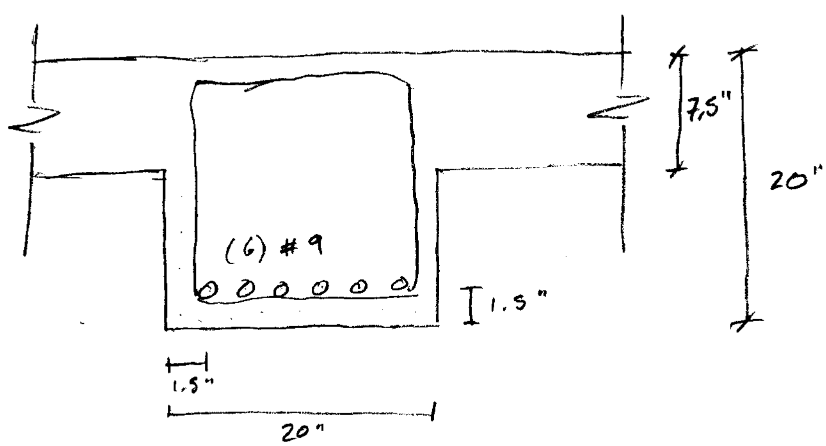
$$\phi M_n = 0.9\left(6(60)\left(21.436 - \frac{5.294}{2}\right)\left(\frac{1}{12}\right)\right)$$

$$= 507.303 > 444 \text{ k ft} \checkmark$$

$$\varepsilon = \frac{0.003}{6.23}(21.436 - 6.23)$$

$$= 0.0073 > 0.005 > 0.00207 \checkmark$$

TENSION CONTROLLED, $\phi = 0.9$ ✓



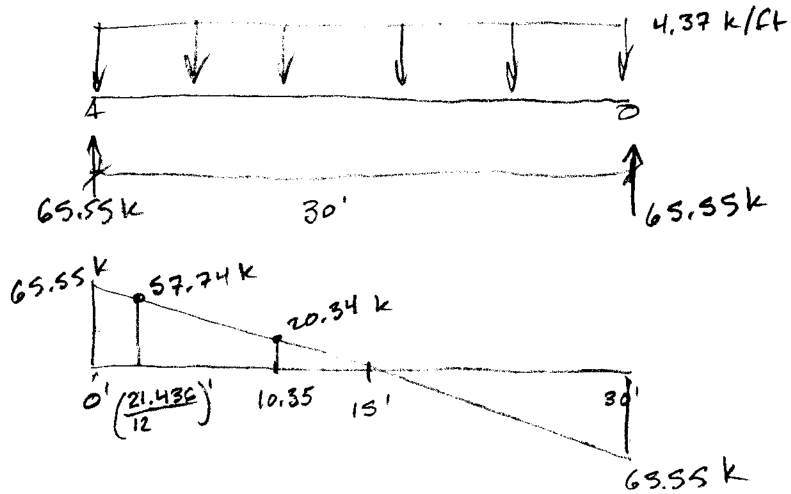
Beam shear

NOTE: THIS SHEAR CALCULATION IS FOR "PROOF-OF-CONCEPT" AND WILL NOT BE CALCULATED AND DESIGNED FOR EVERY MEMBER IN THIS REPORT. THE REASON FOR THIS CHOICE IS THAT SHEAR REINFORCEMENT IS MORE AN EXCERCISE IN CONTROLLING FAILURE MODE AND DOES NOT CONTRIBUTE SIGNIFICANTLY TO MEMBER SIZING, ECONOMY, OR CONSTRUCTIBILITY EXCEPT IN EXTREME CASES.

Calculate V_c

$$V_c = \frac{2\sqrt{4000}(20)(21.436)}{1000} = 54.23 \text{ k}$$

$$\phi V_n = 0.5\phi V_c$$
$$= 0.5(0.75)(54.23)$$
$$= 20.34 \text{ k}$$



Calculate V_s

$$V_s = \frac{V_u}{\phi} - V_c = \frac{57.74}{0.75} - 54.23$$

$$= 22.76 \text{ k}$$

$$V_s \leq 8\sqrt{f'_c}(b_w)(d)$$

$$22.76\text{k} \leq 8\sqrt{4000}(20)(21.436) = 216.9 \text{ k} \checkmark$$

Spacing Requirements

$$V_s \le 4\sqrt{f'_c}\, b_w d$$
$$\le 4\sqrt{4000}(20)(21.436)$$
$$22k \le 108.5\ k$$

$$\rightarrow s_{max} = \min\begin{Bmatrix} d/2 \\ 24'' \end{Bmatrix} = \begin{Bmatrix} \frac{21.436}{2} \\ 24'' \end{Bmatrix} = \begin{Bmatrix} 10.7'' \\ 24 \end{Bmatrix} \sim 10''$$

Minimum shear reinforcing

$$A_{v\,min} = \max\begin{Bmatrix} 0.75\sqrt{4000}(20)(10)/60000 \\ 50(20)(10)/60000 \end{Bmatrix}$$

$$= \begin{Bmatrix} 0.158\ in^2 \\ 0.166\ in^2 \end{Bmatrix}$$

Use #4 @ 10" min

$$2(0.2) = 0.4\ in^2 > 0.166\ in^2 \checkmark$$

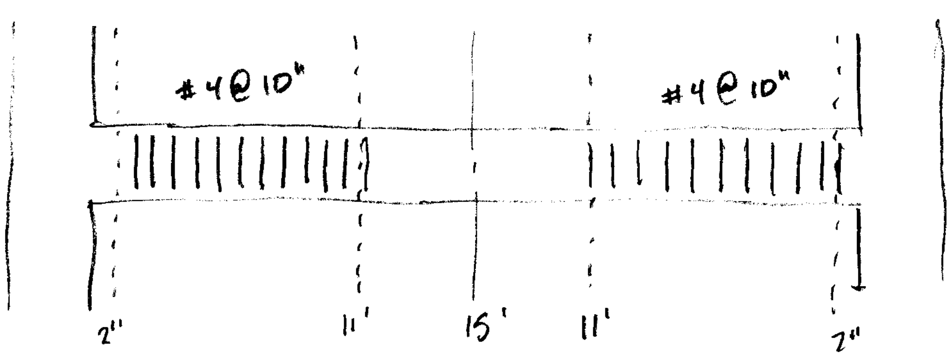
Shear Reinforcing

$$s = \frac{A_v f_y d}{V_s}$$

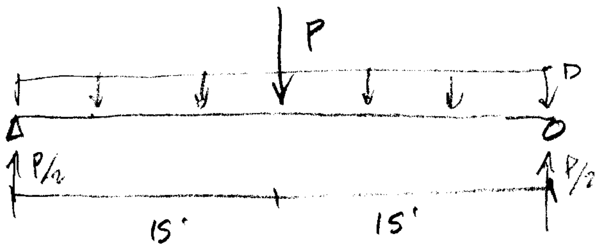
$$= \frac{0.4(60)(21.436)}{22.76}$$

$$= 22.6\ in$$

Use min throughout → to 11' then no reinf.
Start 2" from face of support



GIRDER DESIGN



$P = 4.37\ k/ft\ (15\ ft)$

$= 65.55\ k$

$D = \dfrac{\left(\frac{20\times20}{144}\right)(150)}{1000} = 0.416\ k/f$

$M_u = \dfrac{0.416(30)^2}{8} + \dfrac{65.55}{2}(15)$

$= 46.8\ kft + 491.625\ kft$

$= 538.5$

ESTIMATE SIZE: USE SAME SIZE, ADD REBAR

$h = 24$
$b = 20$
$A_s = 6\ \#10 = 6(1.27) = 7.62\ in^2$
$d = 24 - 1.5 - \frac{1.27}{2} - 0.5$

$= 21.365$

Checks:

Spacing: $\dfrac{20 - 1.5(2) - 0.5(2) - 1.27(6)}{5}$

$= 1.676'' > 1.27'' > 1.0''$ ✓

$a = \dfrac{60(6)(1.27)}{0.85(4)(20)} = 6.723''$

$c = \dfrac{6.723}{0.85} = 7.91''$

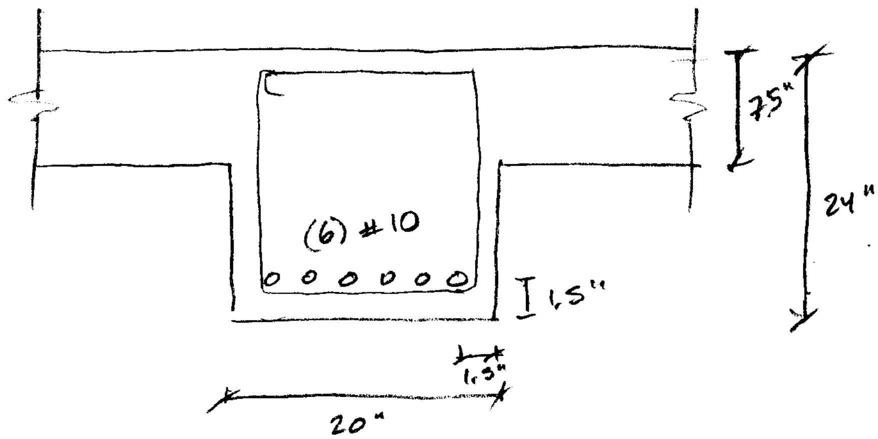
$\phi M_n = 0.9\left(7.62(60)\left(21.365 - \frac{6.723}{2}\right)\left(\frac{1}{12}\right)\right)$

$= 617.3 > 538.5\ kft$ ✓

$\varepsilon = \frac{0.003}{7.91}(21.365 - 7.91) = 0.0051 > 0.005 > 0.00207$ ✓

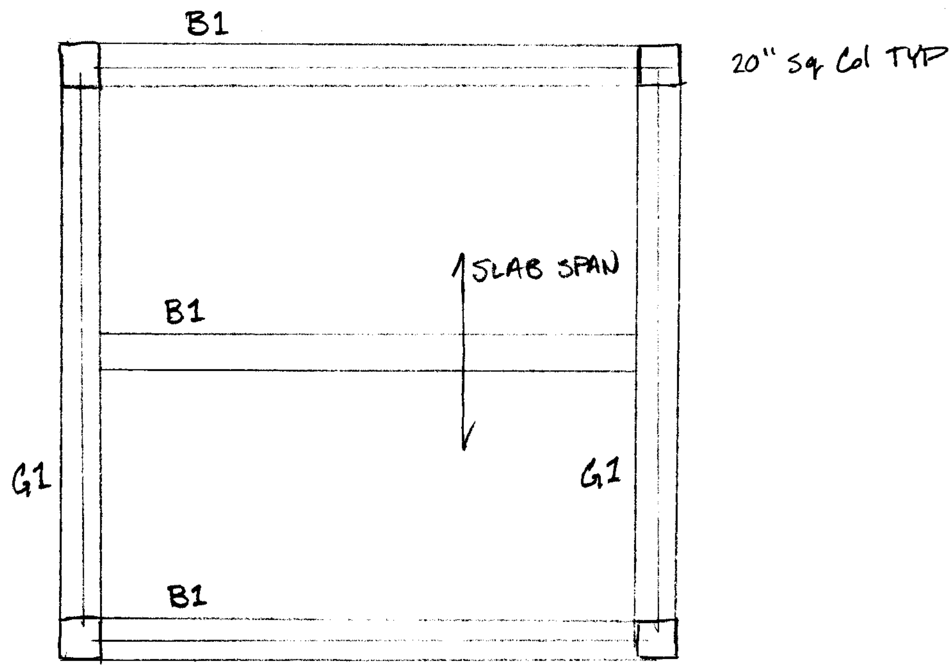
$\phi = 0.9$

GIRDER DIAGRAM



SUMMARY DIAGRAM

1 WAY SLAB WITH INTERMEDIATE BEAM



B1: 20" x 20"

(6) #9 bottom reinforcement, no T action assumed
#4 stirrups @ 10" o.c.

G1: 20" x 24"

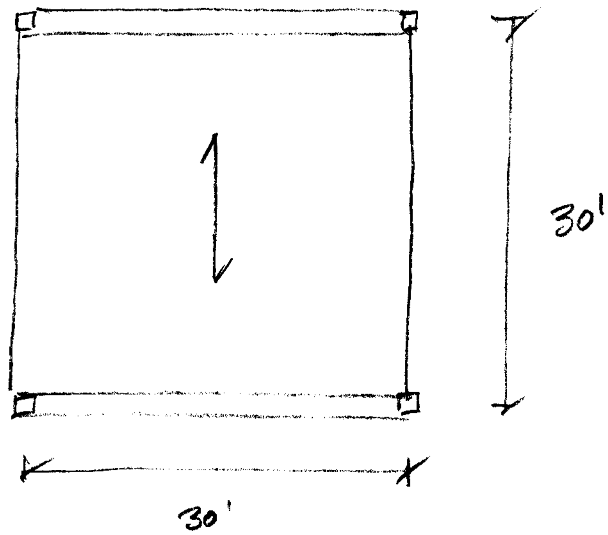
(6) #10 bottom reinforcement, no T action assumed

SLAB: 7.5" SLAB
#4 bars @ 6" O.C.

Thermal / cracking
#4 bars @ 12" o.c.

4000 psi NW concrete
60 ksi steel

1 WAY SLAB W/O EDGE BEAMS



Minimum slab thickness 9.5 (a)

both ends conti. $= \frac{l}{28} = \frac{30(12)}{28} = 12.85" \rightarrow$ 13"

one end cont. $= \frac{l}{24} = \frac{30(12)}{24} = 15"$

USE 13", run long building direction

DEAD LOAD : $150\left(\frac{13}{12}\right) = 162.5$

LL : 80 psf

TOTAL FACTORED LOAD : $1.2(162.5+9) + 1.6(80)$

$= 333.8$ psf

$M_u = \frac{(1)0.333(30)^2}{8}$

$= 37.462$ kft

$A_{sreq} = \frac{37.462(12)}{0.9(60)(0.95 \times (13-2.5))}$

$= 0.835$ in^2

USE # 9 @ 12" O.C.

$A_s = 1.0 > 0.8 \text{ in}^2$ ✓

CHECKS

$S_{max} = \min\left\{ \begin{matrix} 2h \\ 18" \end{matrix} \right\} = \left\{ \begin{matrix} 26" \\ 18" \end{matrix} \right\}$

$A_{smin} = 0.0018(12)(10.5)$
$= 0.2268 < 1.0$ ✓

$d = 13 - 1.5 - \frac{1.128}{2} = 10.936 > 10.5$ ✓

$\varepsilon_s = \frac{0.003}{1.73}(10.936 - 1.73)$
$= 0.015 > 0.005 > 0.00207$ ✓

$a = \frac{1.0(60)}{0.85(4)(1)(12)} = 1.47$

TENSION CONTROLLED, $\phi = 0.9$

$c = \frac{1.47}{0.85} = 1.73$

$\phi M_n = 0.9\left((1)(60)\left(10.936 - \frac{1.47}{2}\right)/12\right)$
$= 45.9 > 37.462 \text{ kft}$ ✓

Thermal/Cracking Reinforcement

$A_{smin} = 0.227 \rightarrow$ use (1) #5 @ 12" O.C.

BEAM DESIGN

TOTAL FACTORED LOAD = $333.8(30) + \frac{7 \times 20}{144}(150)(1.2)$

$= 10189$ plf

$= 10.2$ klf

$M_u = \frac{10.2(30)^2}{8}$

$= 1147.5$ kft

$bd^2 = 20 M_u$

$d = 24 - 2.5$
$= 21.5$

$b = 49.64"$
$= 50"$
$h = 24"$

$A_{sreq} = \frac{1147.5(12)}{0.9(60)(0.95)(21.5)}$

$= 12.4847$

Use (10) #10

$A_s = 10(1.27) = 12.7 \text{ in}^2$

Checks

$S_{max} = 15\left(\frac{40000}{\frac{2}{3}(60000)}\right) - 2.5(1.5)$
$= 11.25$

$S = \frac{50 - 2(1.5) - 0.5(2) - 10(1.27)}{9}$

$= 3.7" < 11.25$ ✓

$d = 24 - 1.5 - 0.5 - \frac{1.27}{2} = 21.365 < 21.5$ CHECK ϕM_n

$a = \frac{12.7(60)}{0.85(4)(50)} = 4.483$ in

$c = \frac{4.483}{0.85} = 5.27$ in

$$\varepsilon_s = \frac{0.003}{5.27}(21.365 - 5.27) = 0.00915 > 0.005 > 0.00207$$

TENSION CONTROLLED, $\phi = 0.9$

$$\phi M_n = 0.9\left(12.7(60)\left(21.365 - \frac{4.483}{2}\right)/12\right)$$

$$= 1092.9$$

FAILS!

USE (12) #10

$$s = \frac{50 - 1.5(2) - 0.5(2) - 12(1.27)}{11} = 2.796 \text{ in } ✓$$

$$a = \frac{15.24(60)}{0.85(4)(50)}$$

$$= 5.378$$

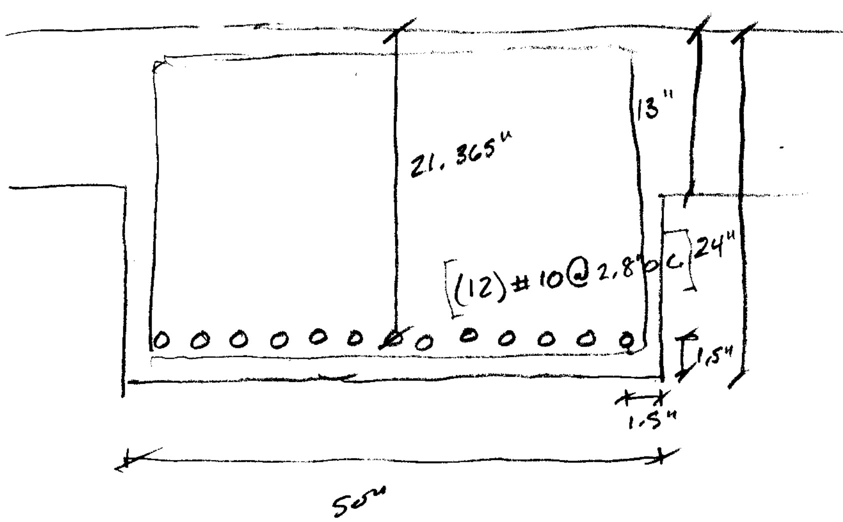
$$c = \frac{5.378}{0.85} = 6.328 \text{ in}$$

$$\varepsilon_s = \frac{0.003}{6.328}(21.365 - 6.328) = 0.007 > 0.005 > 0.00207$$

$$\phi M_n = 0.9\left(15.25(60)\left(21.365 - \frac{5.378}{2}\right)/12\right)$$

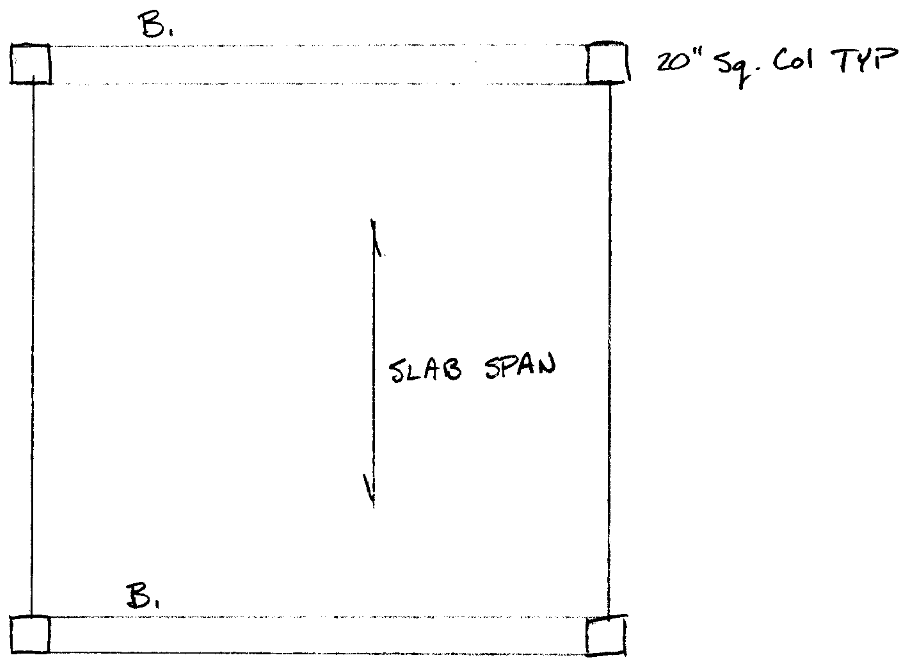
$$= 1281.64 > 1147.5 \text{ kft } ✓$$

PASSES



SUMMARY DIAGRAM

1 WAY CONCRETE SLAB



B_1: 50" x 24"

(12) #10 bottom reinforcement, no T action assumed

SLAB: 13" one way flat slab

#9 bars @ 12" O.C.

Thermal/Cracking

#5 bars @ 12" O.C.

4,000 psi NW concrete
60 ksi steel

Comparisons

Weights:

- Comp.

STEEL: FRAME $26(30)(4) + 55(30)(1) = 4770$ lb

DECK $2.84(30\times30) = 2556$ lb

STUDS $2(15\times4 + 24\times1) = 160$ lb

7486 lb

CONC: FLOOR $(48-2.84)(30\times30) = 40644$ lb

TOTAL 48130 lb, 53.5 psf

- NON COMP

STEEL: FRAME $35(30)(4) + 55(30)(1) = 5850$ lb

DECK $1.68(30\times30) = 1512$ lb

Studs $0 = 0$

7362 lb

CONC: FLOOR $(43-1.68)(30\times30) = 37188$ lb

44550 lb, 49.5 psf

- 2-WAY

STEEL REBAR $\approx 0.005V = [490.0][0.005(30)(30)(\frac{10}{12})] = 1837.5$ lb

CONC: SLAB $0.995(30)(30)(\frac{10}{12})(150) - 1837.5\text{ lb} = 110{,}100$ lb

111937.5 lb, 124.4 psf

- 1 WAY W/ INT BEAM

STEEL REBAR $490(0.005)\left[(30)(30)(\frac{10}{12}) + \frac{12.5(20)(2)}{144} + \frac{16.5(20)}{144}\right] = 1851.6$ lb

CONC SLAB $150(0.995)[\quad "\quad] = 112797.8$ lb

114649.4 lb, 127.4 psf

- 1 way w/o INT BEAM

STEEL REBAR $490(0.005)\left[30(30)(\frac{13}{12}) + \frac{16.5(50)}{144}\right] = 2402.8$

CONC SLAB $150(0.995)[\quad "\quad] = 146373$

148776.6, 165.3 psf

ENVIRONMENTAL IMPACTS: BASED ON CRADLE TO

Steel 1770 $(lbCO2/tonne)\left(\frac{1 tonne}{2204.62}\right) = 0.8029$ $lbCO_2/lb$

Conc 110 $= 0.0498$ $lbCO_2/lb$

LW conc 168 $= 0.0762$ $lbCO_2/lb$

Reinf 872 $= 0.3955$ $lbCO_2/lb$

COMP : $7486(0.8029) + 40644(0.0762) = 9107.58$

NON COMP : $7362(0.8029) + 37188(0.0762) = 8744.7$

2 WAY : $1837.5(0.3955) + 110100(0.0498) = 6209.7$

1 WAY w/ : $1851.6(0.3955) + 112797.8(0.0498) = 6349.6$

1 WAY w/o : $2402.8(0.3955) + 146373(0.0498) = 8239.7$

5 CONCLUSION

An unbiased comparison of five gravity systems based on eight distinct criteria indicated that the two way flat plate concrete slab system was a clear stand-out. The system achieved overall a lower cost, shallower cross section, better fire rating, and better environmental impact than any other system.

The only system that was demonstrated to be unviable was the clear-span one way concrete slab system. The excessive depth of the slab at 13" created large bending moments, heavy beams, and wasteful use of material. Also, with square bay dimensions, a one way slab would tend to act in a two-way mode and cracking could develop in the transverse direction under high loads. Given the need for thermal and cracking reinforcement, it is reasonable to make the switch to a two way slab and drastically decrease the required slab thickness.

Of the concrete systems, the 1 way concrete slab with intermediate beams faired poorly primarily because of its complexity and relatively high cost of construction. Although it does make far more efficient use of materials than the one way clear span system, the additional complexity cannot overcome the difference, or save enough material to rival the two way flate plate concrete slab.

The original composite steel framing system performed surprisingly poorly compared to the other available systems. This is most likely due to the somewhat oversized members currently in place. The assumption of pure vertical gravity loading used in these comparative designs may be neglecting important loading conditions that contributed to the large composite member sizes and thus heavier weight, greater depth, and higher environmental impact. A redesign based on consistent assumptions would be necessary to properly evaluate the potential of the composite system, especially against the non-composite system.

The lateral system in the bed tower consists of special concentrically braced frames and special reinforced concrete shear walls in the East-West direction, and special moment frames in the North-South direction. The elements are efficiently placed at the far wings of the building and the center of rigidity appears to align well with the center of mass upon visual inspection. The system layout is adequate for the given layout, but the elements themselves may be optimized to best suit the building's stiffness requirements to prevent torsional irregularities.

6 Appendix A: Gravity Loads

<u>**Design Criteria (Live Loads)**</u>

Hospitals

Operating rooms, labs	60 PSF *
Private rooms	40 PSF *
Wards	40 PSF *
Corridors (above 1st floor)	80 PSF *

* Design for uniform load indicated or 1000# concentrated load over 2.5 feet square, whichever produces the greater load effect.

Offices

Offices	50 PSF **
Lobbies & 1st floor corridors	100 PSF **
Corridors (above 1st floor)	80 PSF **

** Design for uniform load indicated or 2000# concentrated load over 2.5 feet square whichever produces the greater load effect.

Misc. Live Loads

Corridors, except as otherwise indicated	100 PSF
Stairs and Exits	100 PSF ***
Dining Rooms and Restaurants	100 PSF
Retail Stores (first floor)	100 PSF
Mechanical rooms	125 PSF (Includes allowance for equipment pads)
Storage – Light	125 PSF

*** Design for uniform load indicated or 300# concentrated load over 4 inches square whichever produces the greater load effect

Partition loads	20 PSF

(Offices & locations where partitions are subject to change)

Design Floor Live Loads (Typical unless noted otherwise in calculations)

Typical floors: 80 PSF (60 PSF + 20 PSF Partitions) or (80 PSF Corridors)
First floor (typical): 100 PSF (60 PSF + 20 PSF Partitions) or (100 PSF Corridors)
First floor (equip): 120 PSF (60 PSF + 20 PSF Partitions + 40 PSF Equipment)
Mechanical Rooms: 125 PSF
Elevator Machine Rooms: 500 PSF
Interstitial Level: 25 PSF
Roof Top Mechanical Unit Support: 50 PSF (Live Load + Snow Load)

Other Live Loads

Handrails and guards	50 PLF or 200# concentrated load @ top rail
Components	50# over 1 foot square
Grab bars, shower seats, dressing rm. seats	250# load in any direction at any point

Impact Loads

Elevator loads shall be increased by 100 percent for impact
Machinery weight shall be increased to allow for impact
Elevator machinery: 100 percent
Light machinery, shaft or motor driven: 20 percent
Reciprocating machinery or power driven units: 50 percent
Hangers for floors or balconies: 33 percent

Live Load Reduction
Live loads to columns will be reduced in accordance with IBC Section 1607.9.1. Live loads that exceed 100 PSF and roof live loads will not be reduced.

Distribution of Floor Loads
Uniform floor live loads shall be patterned to produce the greatest effect on continuous framing.

Roof Loads
Uniform roof live loads shall be patterned to produce the greatest effect on continuous framing.
Minimum roof load will be less than snow load
See section 1607.11 for other roof loads (roof gardens, landscaped roofs, canopies)

Interior Walls and Partitions

Interior Partitions	5 PSF horizontal pressure
Medical Equipment	
MRI Equipment (four pt loads)	29000 lb/4 = 7250 lb
MRI Equip minus equip allowance	7250 lb – (40 PSF)*(25 ft2) = 6250 lb

Design Criteria (Dead Loads)

Hospital Floor (Composite slab, 2 Hour)	
3" Deck + 3 1/2" LW Conc	48 PSF
Beams/Girders/Columns	Self Wt (Assume = 9 PSF)
Ceiling/Mechanical/Misc	12 PSF
	60 PSF (Mass DL = 69 PSF + 10 PSF for Partition Mass)
Hospital Roof (Future Floor) (Composite slab, 2 Hour)	
3" Deck + 3 1/2" LW Conc	48 PSF
Beams/Girders/Columns	Self Wt (Assume = 9 PSF)
Ceiling/Mechanical/Misc	12 PSF
Roofing/Insulation/Ballast	18 PSF
	78 PSF (Mass DL = 87 PSF)
Hospital Roof (No future floors) (Composite slab, 2 Hour)	
3" Deck + 3 1/2" LW Conc	48 PSF
Beams/Girders/Columns	Self Wt (Assume 9 PSF)
Ceiling/Mechanical/Misc	12 PSF
Roofing/Insulation/Ballast	18 PSF
	78 PSF (Mass DL = 87 PSF)
Power Plant Roof (No future floors) (Composite slab, 2 Hour)	
3" Deck + 3 1/2" LW Conc	48 PSF
Beams/Girders/Columns	Self Wt (Assume 9 PSF)
Ceiling/Misc	7 PSF
Mechanical Piping	60 PSF
Roofing/Insulation/Ballast	18 PSF
	133 PSF (Mass DL = 142 PSF)
Penthouse Floor (Composite slab, 2 Hour)	
3" Deck + 3 ½" LW Conc	48 PSF
Beams/Girders/Columns	Self Wt (Assume = 9 PSF)
Mechanical/Misc	12 PSF
	60 PSF (Mass DL = 69 PSF + 10 PSF for Partition Mass)
Penthouse Roof (Steel Roof Deck)	
Steel Deck	3 PSF
Beams/Girders/Columns	Self Wt (Assume = 7 PSF)
Mechanical/Misc	7 PSF
Roofing/Insulation/Ballast	18 PSF
	28 PSF (Mass DL = 35 PSF)
Roof Top Mechanical Unit Support	
Beams/Girders/Columns	Self Wt (Assume = 7 PSF)
Mechanical Unit	60 PSF
Miscellaneous Pipes & Ducts 15 PSF	
	75 PSF (Mass DL = 82 PSF)
Hospital Floor – Piping Zone (Composite slab, 2 Hour)	
3" Deck + 3 1/2" LW Conc	48 PSF
Beams/Girders/Columns	Self Wt (Assume = 9 PSF)
Mechanical Piping	60 PSF
Ceiling/Misc	7 PSF
	115PSF (Mass DL = 94 PSF + 10 PSF for Partition Mass)
Hospital Floor/Power Plant (Composite slab, 2 Hour)	

3" Deck + 3 1/2" LW Conc	48 PSF
Beams/Girders/Columns	Self Wt (Assume = 9 PSF)
Mechanical Piping	60 PSF
Ceiling/Misc	7 PSF
	115PSF (Mass DL = 94 PSF + 10 PSF for Partition Mass)
Hospital Floor – MRI Zone (Composite slab, 2 Hour)	
3" Deck + 3 1/2" LW Conc	48 PSF
Beams/Girders/Columns	Self Wt (Assume = 9 PSF)
2" Concrete Topping	18 PSF
Mass for Permanent Equip	(15 PSF Mass DL)
Ceiling/Mechanical/Misc	12 PSF
	78 PSF (Mass DL = 102 PSF + 10 PSF for Partition Mass)
Hospital Floor – Piping Zone plus MRI Zone (Composite slab, 2 Hour)	
3" Deck + 3 1/2" LW Conc	48 PSF
Beams/Girders/Columns	Self Wt (Assume = 9 PSF)
2" Concrete Topping	18 PSF
Mass for Permanent Equip	(15 PSF Mass DL)
Mechanical	30 PSF
Ceiling/Misc	7 PSF
	103 PSF (Mass DL = 127 PSF + 10 PSF for Partition Mass)
MOB Floor (Non-Composite slab, 0 Hour)	
1 ½" Deck + 2" LW Conc	29 PSF
Beams/Girders/Columns	Self Wt (Assume 9 PSF)
Ceiling/Mechanical/Misc	7 PSF
	36 PSF (Mass DL = 45 PSF + 10 PSF for Partition M ass)