Technical Report 2

The Residences
Anne Arundel County, Maryland

10/27/2010
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Executive Summary

This report contains an analysis of four different floor systems for the Residence. The four alternative systems that were included in this studied were:

- Hambro Floor system (Existing Floor System)
- Composite Steel Beams with Composite Deck
- Two-Way concrete Floor
- One-Way Slab

These systems were primary compared by their building weight, architectural impact, and serviceability. Several other factors were considered in comparison of the systems such as fire protection, constructability, and cost. This study revealed that all three alternative floor systems are to be considered for further research. The Two-Way concrete Floor does pose some problem with the lack of square bay; however the column layout may be altered. All floor systems are to be included for further research.

Introduction

Located in Anne Arundel County, Maryland the Residence is a new construction apartment and retail building part of the Arundel Preserve Town Center Phase I project (Figure 1). The Residence is a five to six story, 300,000 s.f., residential apartment building with 6,000 s.f. retail space surrounding a 5 story precast parking garage. This apartment building houses 242 upscale residential units consisting of studio, one, and two bedroom layouts and two level units. Along with the residential units the building also included a terrace level that contains a clubhouse, health center, and an outside pool. Construction of The Residence began in the fall of 2009 and should be completed in the beginning of 2011. It is owned and managed by the Somerset Construction Company and was designed by KTGY.

The structure of The Residence is comprises of the Hanbro floor system, this system uses a steel bar joist that supports a concrete slab (Figure 2). The floor systems are supported by 6" light gage metal studs bearing and shear walls located throughout the building. A more in-depth structural analysis and detail shall follow in this report.

Figure 1: Site plan, Light Brown-build, Gray-parking garage. Source: Cates Engineering.

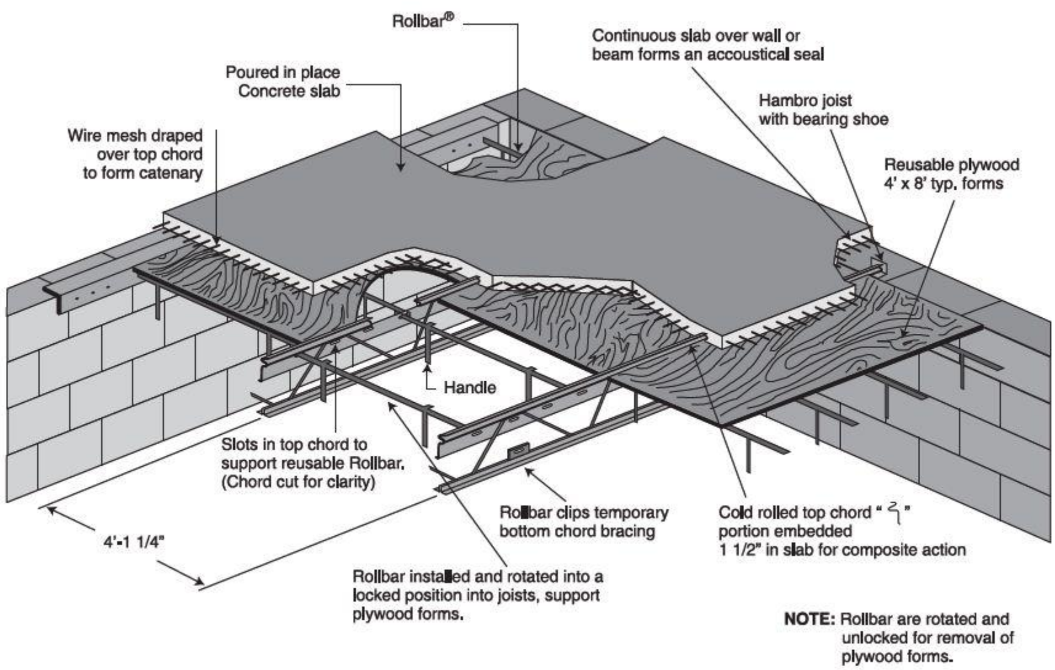


Figure 2: Hambro floor joist system. Source: Hambro.

Structural system

Foundation System

According to the geotechnical report the building rests on Silt-Clay Facies[1] which is identified as clay, silt, and subordinate fine to medium grained muddy sand. The groundwater table was located to be at a minimum 24 feet below existing grade, which is well below the foundation of the building. From the report it was determined that the structures can be supported on shallow spread footings with an allowable bearing pressure of 5,000 pounds per square foot.

The building foundation system uses a 3’-0” wide strip footing with 3’-0”x3’-0” to 15’-0”x15’-0” column footing pads located mainly around the retail space and clubhouse area (Figure 3). The concrete slab on grade was 4” thick reinforced with 6 x 6 W1.4 xW1.4 welded wire fabric. All foundation concrete was to be a 3,000 psi at 28 day strength.

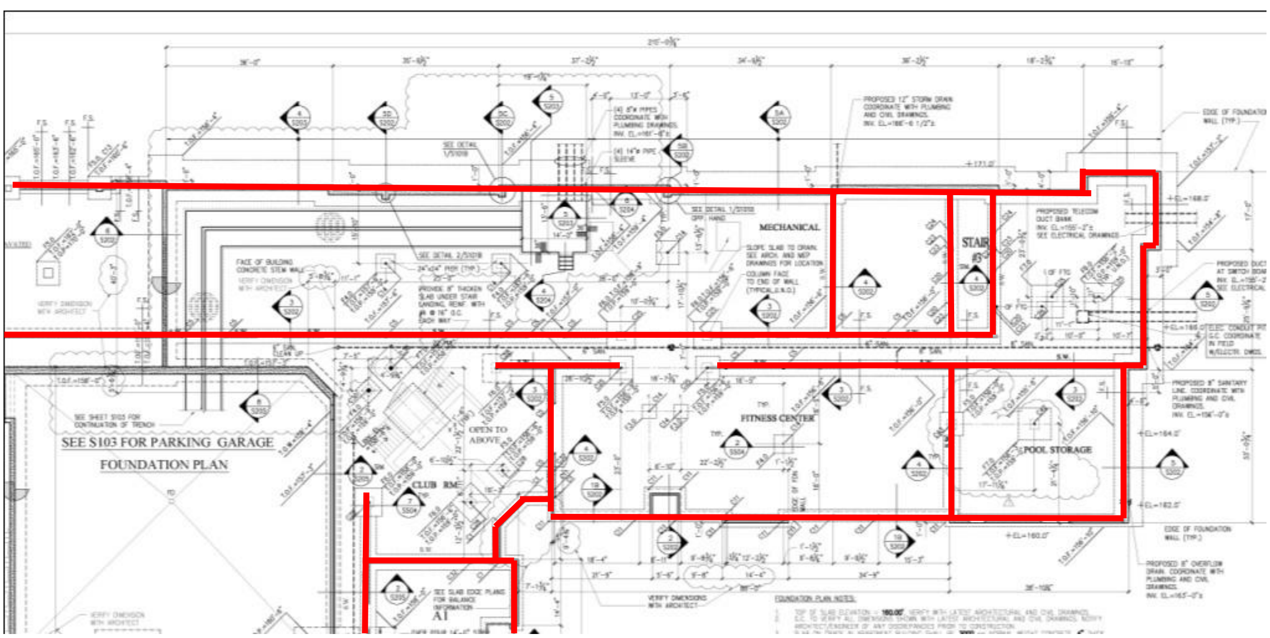


Figure 3: Foundation Plan, Part of the East wing. Source: Construction Documents.

[1] In geology, facies are a body of rock with specified characteristics.

Floor System

The floor system that was used for the Residence was the Hambro floor joist system (Figure 2). The Hambro floor system uses a specially design steel bar joist with a “S” shape top compression chord that serves three functions, a compression member in the non-composite joist during the construction stage, a chair for the welded wire fabric, and a continuous shear connection for the composite (cured concrete) stage. Detail information of the “s” shape top cord can be seen in Figure 4. The floor slab is a 3” thick 3,000 psi concrete with 6 x 6 W2.9 x W2.9 welded wire fabric, this particular floor thickness was chosen to give the system a 2 hour fire rated system. The slab is then supported by a 20” deep Hambro bar joist.

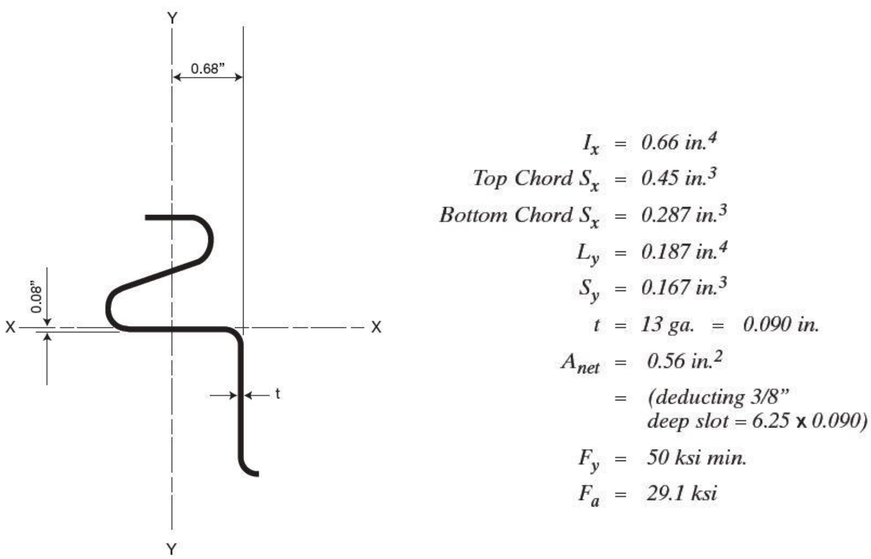
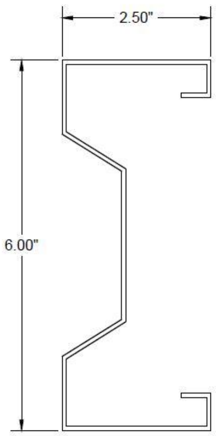


Figure 4: Top Chord of the Hambro joist, "s" chord, with section properties.

Framing System

The design framing system used in the Residence was light gage steel load bearing walls that are used to support the Hambro floor system and gravity loads in the building. The particular system used was the SigmaStud® load bearing light gage steel stud, a product of The Steel Network Company. The stud design is engineered to have a significant increase in load capacity when compared to the conventional “C” shaped studs. The Residence uses a 6" wide 18 gage stud with a flange length of 2.5", as detailed in Figure 5. The exterior wall and interior corridor walls of the Residence are the primary bearing walls in the building; Figure 6 shows the location of the bearing walls in the building.



$A=0.772\ in^2$
$I_x=4.183\ in^4$
$I_y=0.513\ im^4$
$F_y=50\ ksi$
$r_x=2.328\ in$
$r_y=0.815\ in$
$E=29{,}000\ ksi$

Figure 5: Section of light gage steel stud, with section properties.

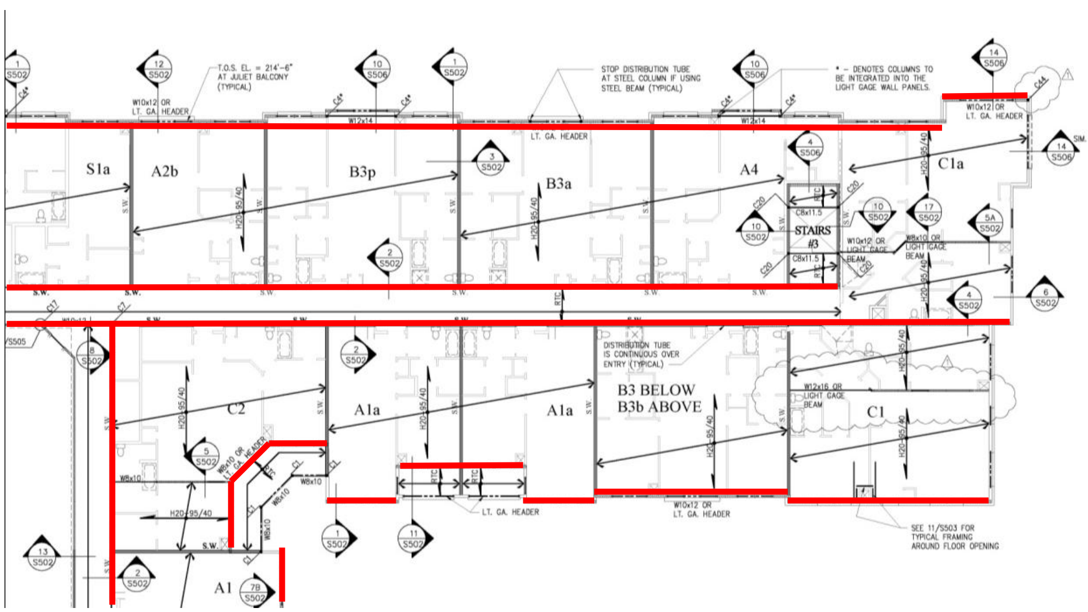


Figure 6: Location of bearing walls, See Appendix A for more plans. Source: Construction Documents.

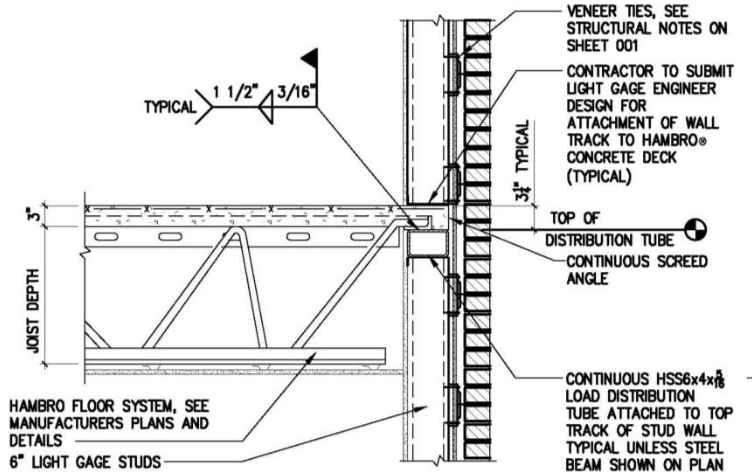


Figure 7: Exterior wall framing details. Source: Construction Documents.

Lateral System

The lateral system used in the Residence was a light gage shear wall system designed and engineered by The Steel Network Company. The system utilizes light gage 50 ksi steel hot dipped galvanized coated straps on both sides of the wall for shear resistance. A 6” wide flat strap was used in lateral system of the Residence. (See figure 8 for a simple framing detail). The shear walls are located all throughout the building (figure 9), with most of the shear wall located in the corridor walls and the walls separating adjacent apartments.

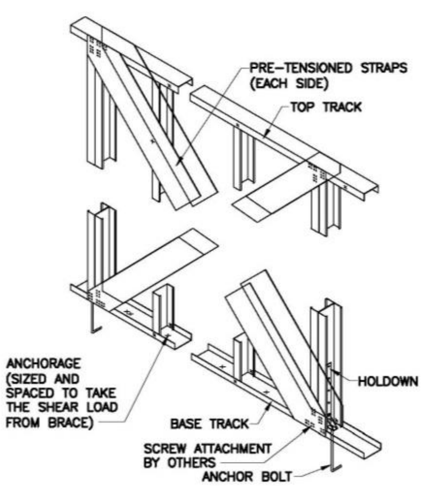


Figure 8: Lateral resistance system. Source: Construction Documents.

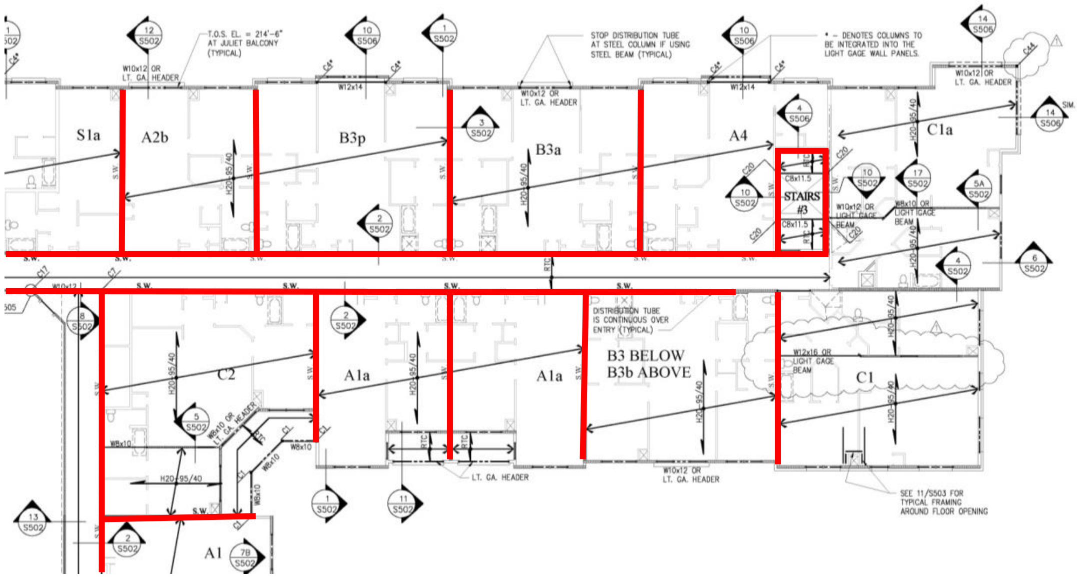


Figure 9: Location of the shear walls, Appendix A for more details. Source: Construction Documents.

Roof System

The roof system was the same system, Hambro flooring system, which was used for the floor throughout the building. The roof slab is 3" thick 3,000 psi concrete with 6 x 6 W2.9 x W2.9 welded wire fabric, which is supported by a 20" deep Hambro joist.

Materials Used

Concrete

Floor Slab	Normal Weight	f'c=3,000 psi
Roof Slab	Normal Weight	f'c=3,000 psi
Slab on grade	Normal Weight	f'c=3,000 psi
Footings	Normal Weight	f'c=3,000 psi

Steel

W shapes	ASTM A992	Grade 50
Square and Rectangular HSS	ASTM 500	Grade B
Channels	ASTM A36	
Angles shapes	ASTM A36	
Steel Plates	ASTM A36	

Reinforcement

Deformed bars	ASTM A-615	Grade 60
Welded wire Fabric	ASTM A-185	

Codes and References

Design Codes

National Model Code:
- 2006 International Building Code

Design Codes:
- Steel construction Manual 13th edition, AISC
- American Iron and Steel Institute (AISI) 2008 Design of Cold Formed Steel Structural members
- American Concrete Institute (ACI) ACI 530-05, Building Code Requirements for Masonry Structures
- American Concrete Institute (ACI) ACI 318-08, Building Code Requirements for Structural Concrete

Structural Standards:
- American Society of Civil Engineers (ASCE), ASCE 7-05, Minimum Design loads for Buildings and other Structures

Thesis Codes

National Model Code:
- 2006 International Building Code

Design Codes:
- Steel construction Manual 13th edition, AISC
- American Concrete Institute (ACI) ACI 318-08, Building Code Requirements for Structural Concrete

Structural Standards:
- American Society of Civil Engineers (ASCE), ASCE 7-05, Minimum Design loads for Buildings and other Structures

Load Analysis

Gravity Load

For this report and all further reports the use of the ASCE7-05 design loads will be used. When comparing the design live loads to the minimal ASCE7-05 loads it was found that all loads except the roof live load were identical to the ASCE7-05. Table 1.1 shows the design and ASCE7-05 live loads on the building. The roof live load was design to be 30 psf which is slightly higher than what is stated in ASCE7-05, 20 psf. It is likely that this value was higher to support some of the MEP system on the roof as well as experience of the designers.

Table 1.1: Live Loads

Location	Design (psf)	ASCE7-06 (psf)
Roof	30	20
Living	40	40
Private Decks/Balconies	60	60
Corridors Exit stairs	100	100
Light Storage	125	125

Dead loads values we found from a series of sources including, but not limited to ASCE7-05 and manufacturer specification. Design dead load on the building can be found in Table 1.2. A listing of assumed dead loads can also be found in Table 1.3.

Table 1.2: Design Dead Loads

Location	Design (psf)
Roof	40
Living	55
Private Decks/Balconies	45
Corridors Exit stairs	45
Light Storage	45

Table 1.3: Assumed Dead Load

	Assumed load (psf)
Slab	36*
Joist	5
Supper impose Dead load	15
wall	15

*** Slab dead load was calculated using a 3" think slab and 145 pcf for concrete**

Snow Load

Due to the location of this building being a snow region, snow loads were calculated in accordance to ASCE7-05 section 7. The results of the load calculation can be seen in table 2, with detail calculation and notes can be found in Appendix B.

Table 2: Snow loads

Ground snow load	Pg= 30 psf
Flat roof snow load	Pf= 21 psf
Slop roof snow load	Ps= 21 psf

Floor Systems

For this report, a typical interior bay lay out of The Residence will be analyzed for the existing floor system and three alternative floor systems; existing framing plans are provided in Appendix A. Figure 10 shows the layout of the typical interior floor plan that was uses in this report. This particular floor plan was chosen to minimize the need to place columns in the apartments. The design of each floor system along with their advantages and disadvantages shall fallow with detail calculation in Appendix C. The effects of lateral loads and sizing of column were not investigated in this report but would need to be done to complete a throw design.

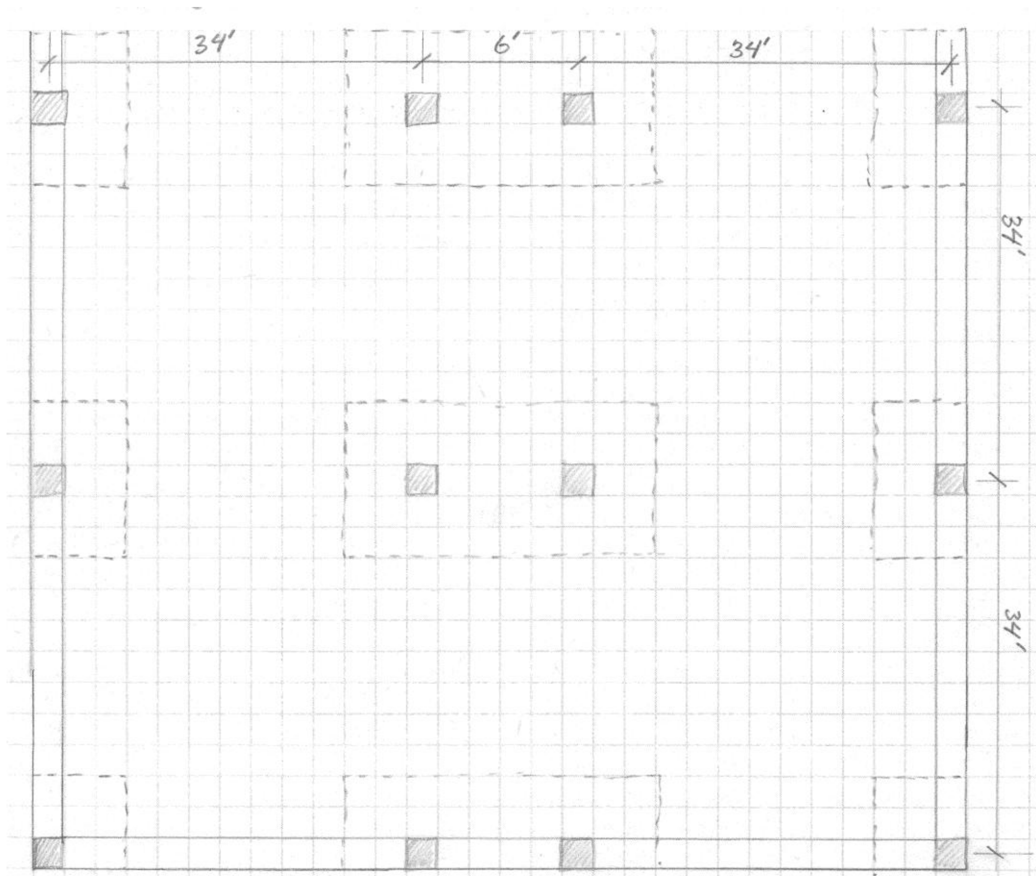


Figure 10: Typical Floor plan lay out.

Hambro Floor system (Existing Floor System)

Description

The Hambro floor system uses a 3" thick 3,000 psi concrete floor slab with 6 x 6 W2.9 x W2.9 welded wire fabric, this particular floor thickness was chosen to give the system a 2 hour fire rated system. The slab is then supported by a 20" deep Hambro bar joist. This is a specially design steel bar joist with a "S" shape top compression chord that serves three functions, a compression member in the non-composite joist during the construction stage, a chair for the welded wire fabric, and a continuous shear connection for the composite (cured concrete) stage.

A typical bay width used in The Residence is approximately 32'-0" with the length of the bay varying with the sizes of apartment units. For this report a 32'-0" x 36'-0" bay size was used to check member sizes, Figure 11 shows the bay layout.

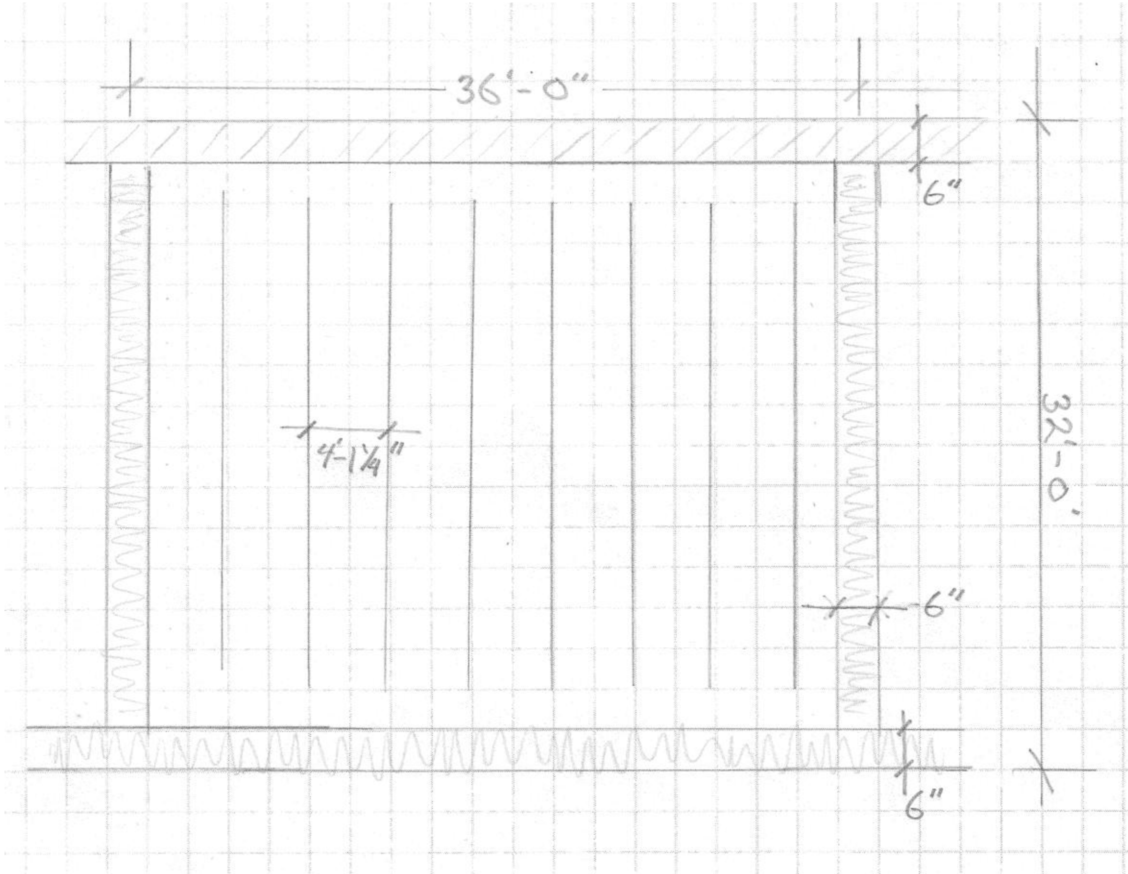


Figure 11: Hambro Floor play lay out.

Material Properties

Concrete:	3" Normal Weight concrete f'c = 3000 psi
Reinforcement	Fy = 60,000 psi Welded Wire Fabric 6 x 6 x W2.9 x W2.9

Loading

Dead Load (self weight):	41 psf
SDL:	15 psf
Live Load:	40 psf (Living units) 100 psf (Corridors)

Advantages

There are many advantages to using the Hambro floor system. The first advantage is the easiness of the construction which the Hambro uses a simple eight step approach to install the system allowing for a shorter construction time. The system also use stander 4' x 8' plywood sheets for the bottom formwork for the concrete. The use of bar joists allows significant space for the mechanical duct work, piping, and electrical wires. The overall weight of the system is much less than other system allowing the foundation to be much smaller.

Disadvantages

Only a few disadvantages could be found with the Hambro system. The first being that the contractor must have some understanding of the installation presses of the floor system, even with the simple eight step approach. The system must be installed properly to allow for adequate strength and safety of the system. To aquaria the specified fire rating a ceiling of at minimal ½" gypsum board must be used.

Composite Steel Beams with Composite Deck

Description

The composite metal deck on composite steel beam is a system that combines the strengths of steel in tension and compression of the concrete, to provide an effective system. A typical bay system was used to design the composite steel systems, (see figure 12 for the layout). W-shape girders span from column to column with an infill beam framing into the girder. The metal deck that sits on the beam spans perpendicular to the beam. When using metal decking, composite action is easily obtained. However, extra design steps are needed to obtain composite beam action. For a beam to obtain composite action with the slab, shear studs are required along the length of the beam. The shear studs transfer the load from the concrete slab into the beam. The supporting calculations for the design of the composite steel system may be found in Appendix C: Composite Steel Beams with Composite Deck.

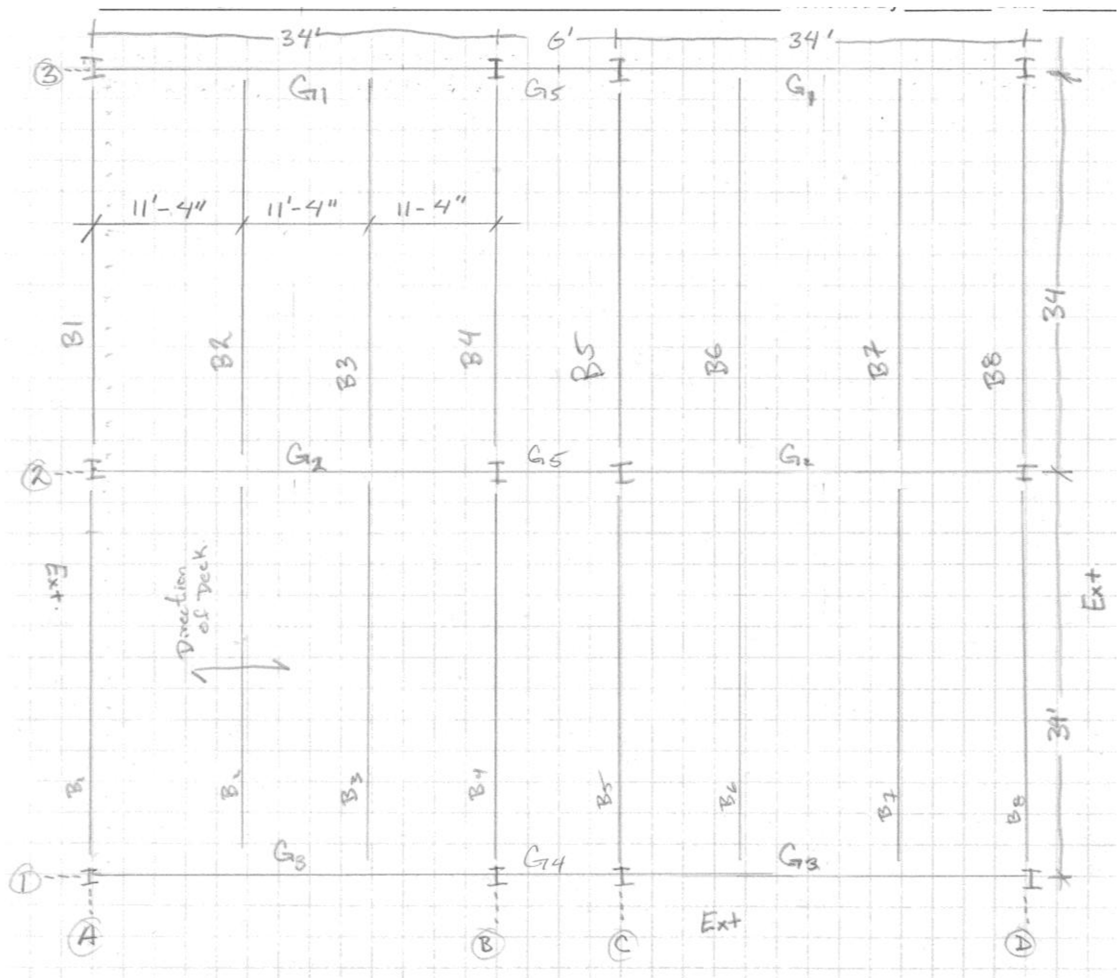


Figure 12: Composite Beam and Deck floor lay out.

Material Properties

Concrete	3" Normal Weight concrete slab on Metal Deck f'c = 3000 psi
Decking	17 Gage metal Deck, Valcraft 2VLI17
Steel	A922 W-Shapes Beams: W14 Girders: W16, W21

Loading

Dead Load (self weight):	51 psf (slab) 10 psf (Beam)
SDL:	15 psf
Live Load:	40 psf (Living units) 100 psf (Corridors)

Advantages

A composite metal deck on composite steel system has many advantages. The metal deck provides the necessary formwork to place the concrete, with proper beam spacing; no shoring is required during construction. The composite system allows the use of smaller steel members and a thinner concrete slab making it a light weight system. A shorter construction time is achieved with composite beam and deck system compared to other systems.

Disadvantages

A composite beam system does have smaller beams, but the beams are still around 16 inches deep. The space between the ceiling and the bottom of the slab may need to be increase to allow for the mechanical and electrical systems. There is a few more cost associative with the connections of a composite beam system. A faster construction time is achieved with the composite steel; however there is an increase in labor for the placement of the shear studs. To obtain the proper fire rating a spray on fireproofing is required for the structural steel.

Two-Way Flat Slab concrete Floor

Description

The design of the Two-Way reinforced flat slab system is comprised of 11” thick normal weight concrete slab with 2.75” drop panel, Figure 13 shows the layout of the floor system. The typical reinforcement used across the entire system is #8 bars at minimal 12 inches on center.

The slab was design to resisted flexural, shear, and deflection. The Equivalent Frame Method prescribed by ACI 318-08 was used to design the floor system. The slab thickness of 11” was minimum required in accordance with ACI 318-08 Table 9.5(c). Punching shear and wide beam shear was checked at the columns and drop panels, but was found not to exceed the limits. The preliminary sizes for the columns are 12” square; this however may have been an underestimation, further investigation would need to be conducted to confirm. The system was not design for progresses collapse but would need to be considered. The supporting calculations for the design of the Two-Way Flat Slab system may be found in Appendix C: Two-Way Flat Slab concrete Floor.

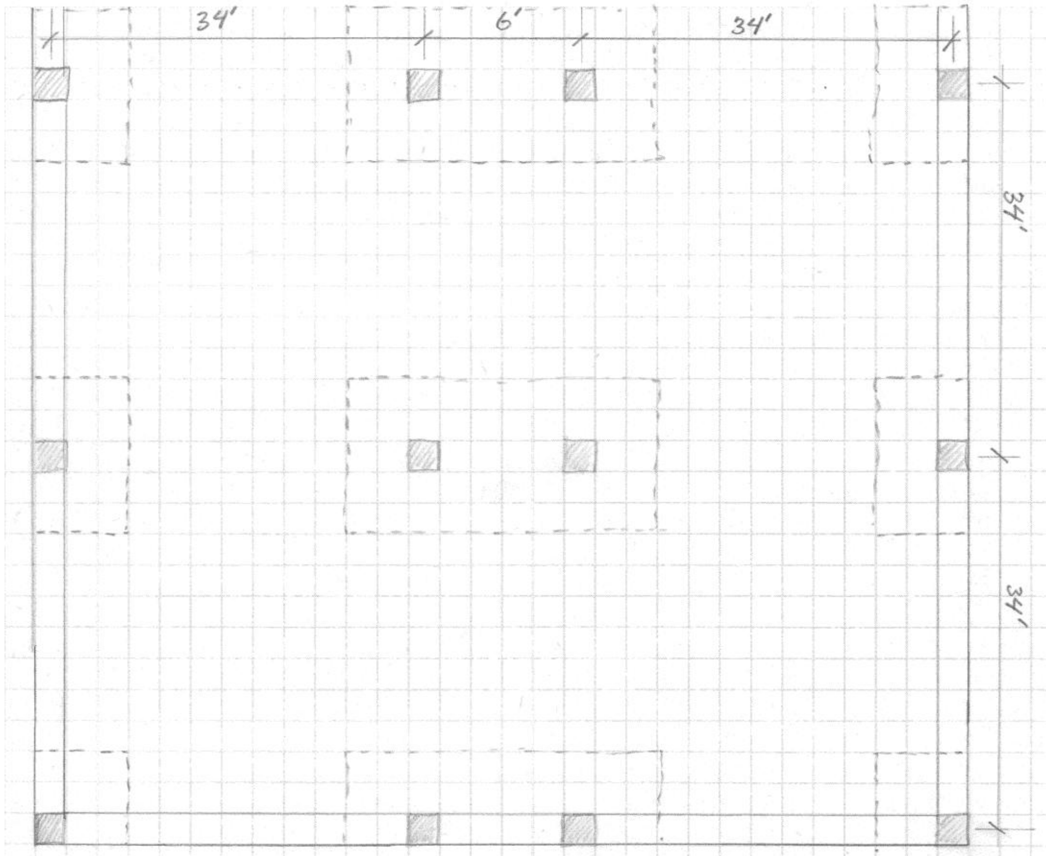


Figure 13: Two-Way Flat Slab floor lay out.

Material Properties

Concrete:	11" Normal Weight concrete with Drop Panels 12" x 12" columns f'c = 4000 psi
Reinforcement	Fy = 60,000 psi

Loading

Dead Load (self weight):	150 psf
SDL:	15 psf
Live Load:	40 psf (Living units) 100 psf (Corridors)

Advantages

A Two-Way Flat Slab system provides a large floor to ceiling height; this allows more space between the ceiling and the bottom of the slab for mechanical and electrical system. No interior beams were used to support the slab; therefore more space could be coordinated with the mechanical and electrical disciplines. Additional fireproofing is not required for the concrete system because it is built into the clear cover of the steel.

Disadvantages

A Two-Way Flat Slab design requires an aspect ratio of less than 2; the corridor bays of the build do not meet this requirement. To achieve this ratio, the bay sizes would have to be change to be squarer; this would have an impact on the architectural design of the apartment units. Construction time for placing the concrete is long because of the increase of time for forming and shoring of the concrete. The weight of the system is much greater than the other systems there for the foundation may have to be redesign for the additional weight.

One-Way Slab

Description

The one-way slab system was designed for an 8" concrete slab that spans a maximum distance of 17'. A girder spans between the columns with a beam framing into the girder. Figure 14 shows the layout of the floor system. The 8" slab was designed to have the fallowing reinforcement; #6 at 12" o.c for flexure steel and #4 at 12" o.c. were provided for temperature steel. The preliminary sizes for the columns are 12" square this however may have been an underestimation; further investigation would need to be conducted to confirm.

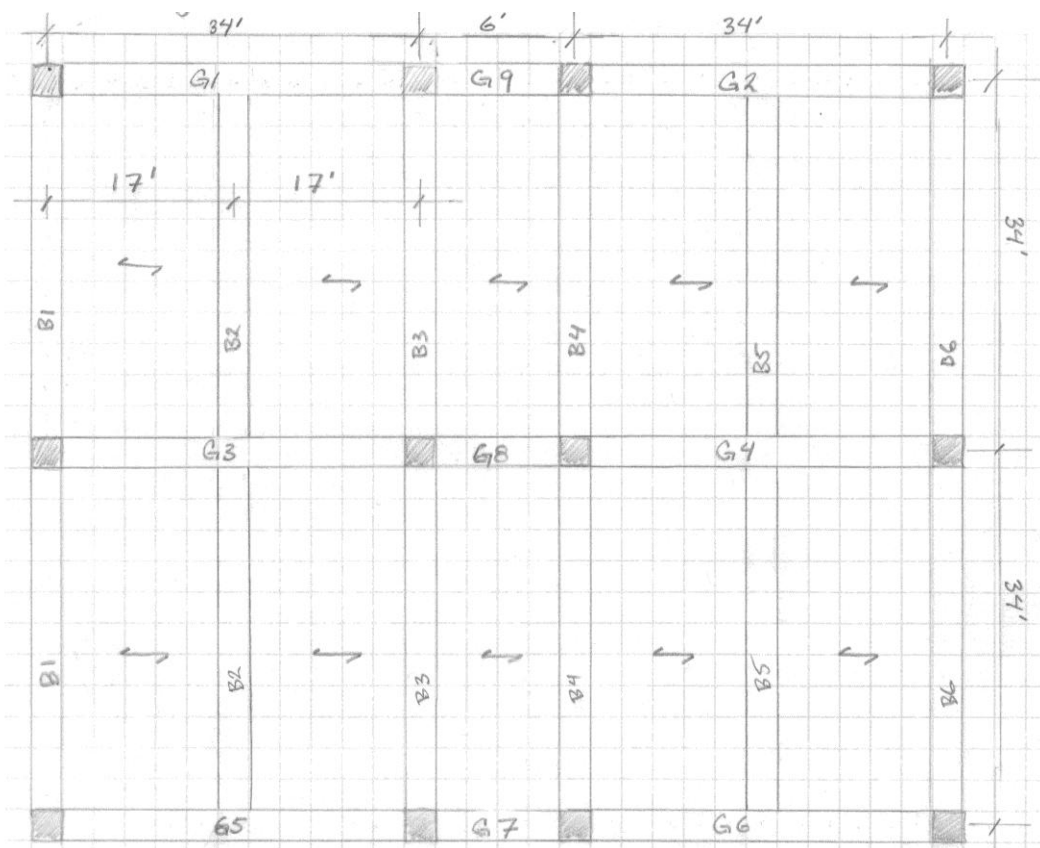


Figure 14: One-Way floor lay out.

Material Properties

Concrete	8" Normal Weight concrete 12" x 12" Columns f'c = 3000 psi
Reinforcement	Fy = 60,000 psi

Loading

Dead Lead (self weight):	110 psf
SDL:	15 psf
Live Load:	40 psf (Living units) 100 psf (Corridors)

Advantages

At this time the only advantage to a one-way floor system is that additional fireproofing is not required for the concrete system because it is built into the clear cover of the steel.

Disadvantages

A one-way slab has many disadvantages when compared to other floor systems. Construction time for placing the concrete is long because of the increase of time for forming and shoring of the concrete. The weight of the system is much greater than the other systems; the foundation may have to be redesign for the additional weigh.

Conclusion

The analysis of the three alternative floor systems and the existing floor system of the Residence revealed that there were many available systems that can be used for the design of the building. Each floor system presented their own set of advantages and disadvantages. The existing system, the Hambro floor system, provided a low weight, ease of construction, and a low cost system. The composite bean and deck has many of the same advantages and disadvantages that the Hambro floor system has. This system did come at an additional cost for the need to install fire protection and the installation of shear studs. The One-Way and Two-Way concrete system also shared similar advantages and disadvantages. One advantage that both of these systems has is the lack fore addition fire protection; the fire protection is built into the clear cover of the rebar. One drawback of these systems is the increase of weight; this would have an effect on the foundation and seismic load. The Two-Way system does have a problem with the layout of the column, this layout does not have continues square bays. Rearranging the column layout to achieve square bays maybe have an affect the architectural lay out of the apartment units. A comparison of the four systems can be found in the following table. All floor systems are to be included for further research.

System Comparison

	Floor System	Hambro Floor System	Composite Beam & Deck	Two-Way Flat Slab	One-Way Slab
Weight (Dead)		41 psf	61 psf	150 psf	110 psf
Architectural Impact	Column Layout	N/A	Good	Poor	Good
	Floor Depth	20" *	14"-21"	13.75"	19"
Deflection		Good	Good	Good	Good
Vibration		Good	Good	Good	Good
Constructability		Easy	Easy	Moderate	Moderate
Fire Protection		2 hr.	2 hr.	2 hr.	2 hr.
Foundation Impact		Little	Little	Major	Major
Approximate Cost +		$17.87 ++	$25.80 **	$19.20	$23.85
Additional Study		N/A	Yes	Yes	Yes

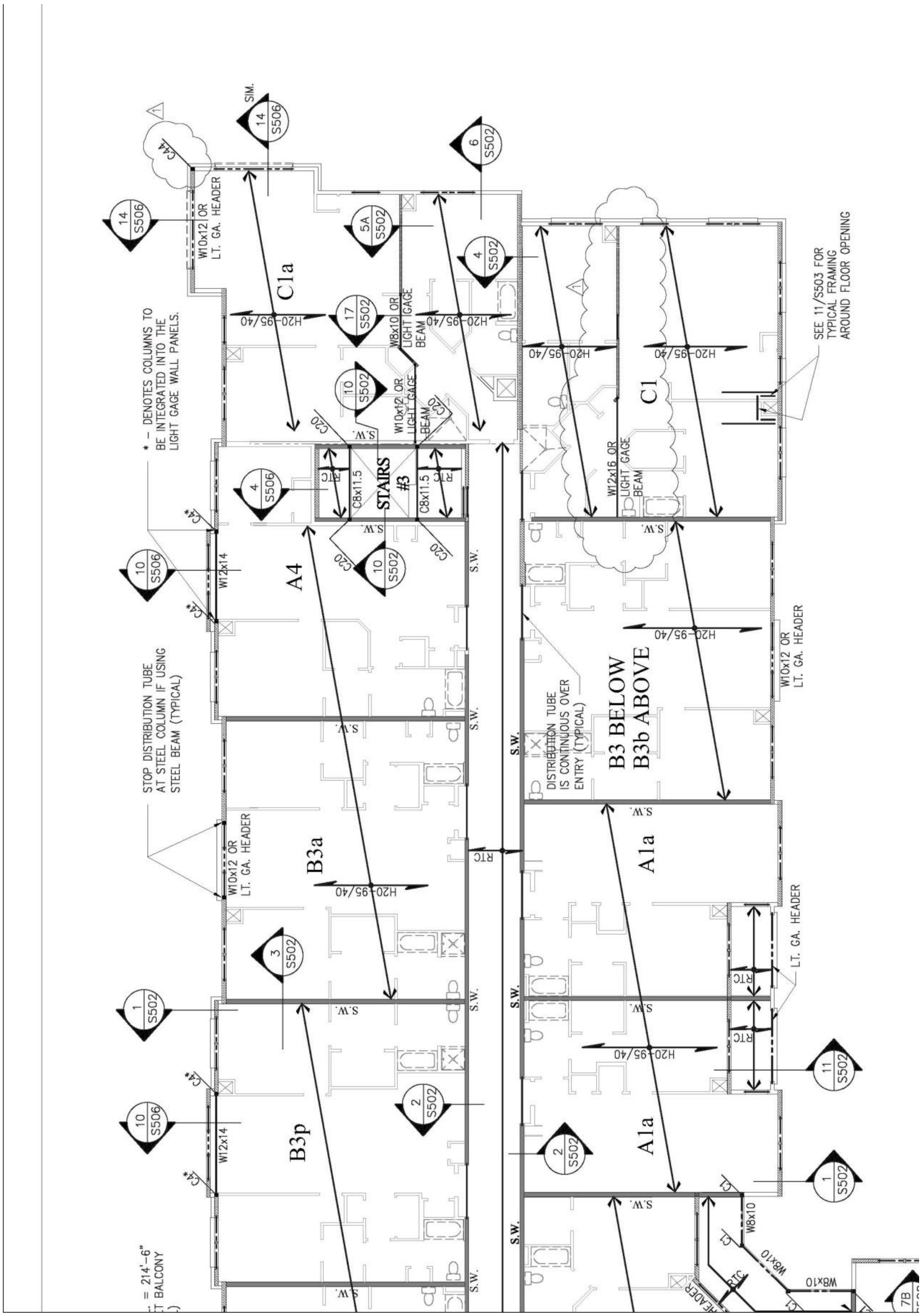
* The Hambro Floor system allow mechanical and electrical equipment to pass through the bar joist

** Cost includes Fire Protection cost

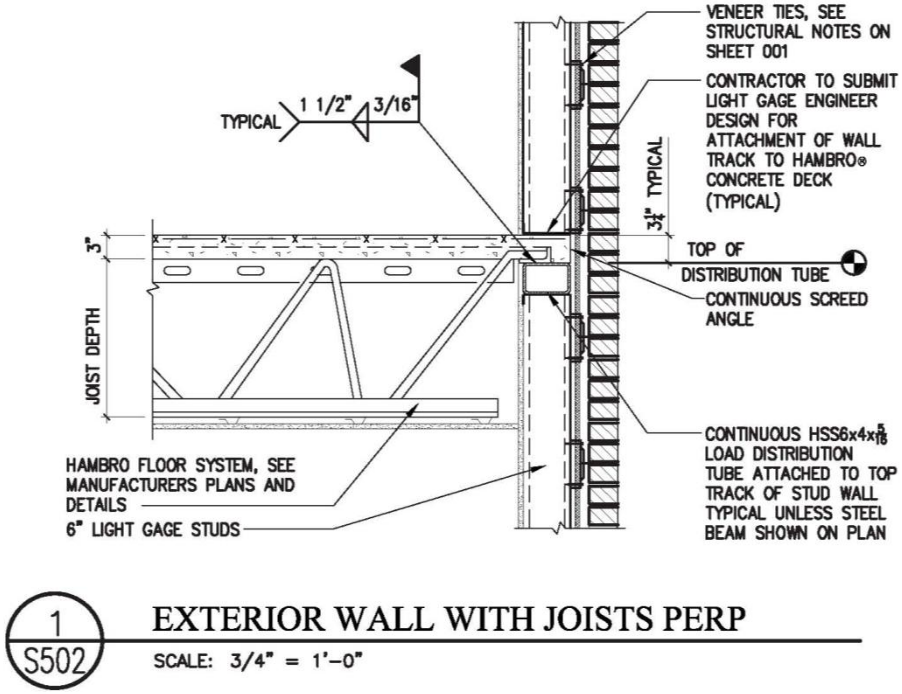
+ Cost data attained form RSMeans 2011

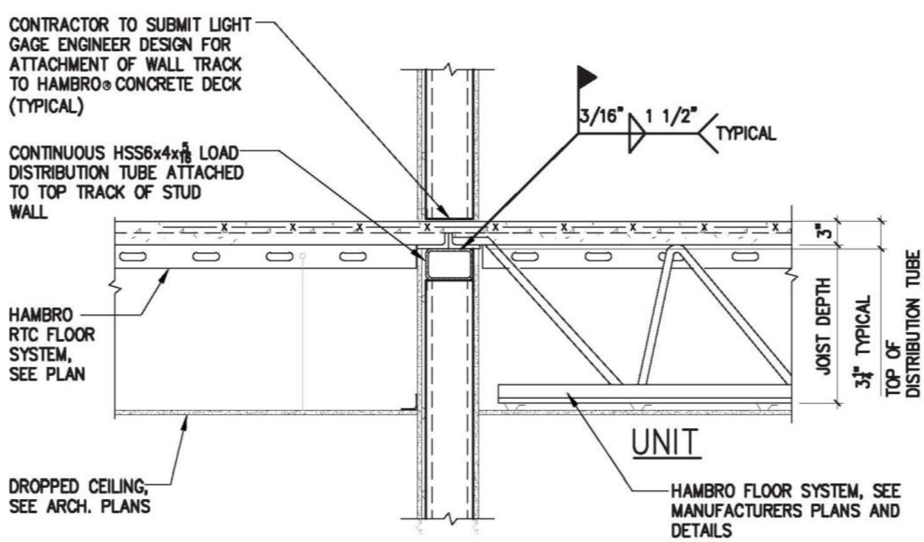
++ Cost date is for a typical steel bar joist system, cost data for the Hambro System was unavailable

Appendix A: Building Plans.



Fifth Floor lay out. Source: Construction Documents





Exterior and interior wall connection Source: Construction Documents

Appendix B: Snow Load Analysis

ASCE7-05 Section 7

(7.2) Ground snow load

Pg= 30 psf

(7.3) Flat Roof

Pf= 0.7 Ce Ct I Pg

(7.3.1) Exposure Factor

Table 7-2

Ce = 0.9

(7.3.2) Thermal Factor

Ct = 1.1

(7.3.3) Importance Factor

I = 1.0

Pf= 0.7(0.9)(1.1)(1.0)(30)= 20.79 -> 21 psf

(7.4) Slop Roof

Ps=Cs Pf

Cs =1.0

Ps = 21 psf

Snow Drifting

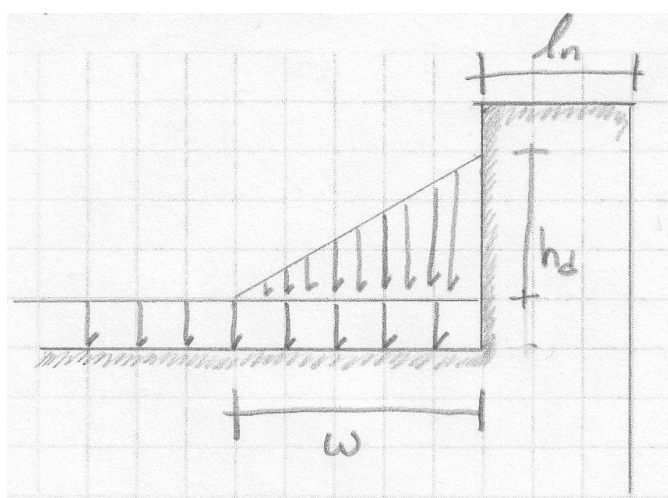
L_n = 11’-6”

h_d = $0.34\sqrt[3]{l_n}\sqrt[4]{P_g + 10} - 1.5$ = 2.64’

w = 4 h_d = 10.58’

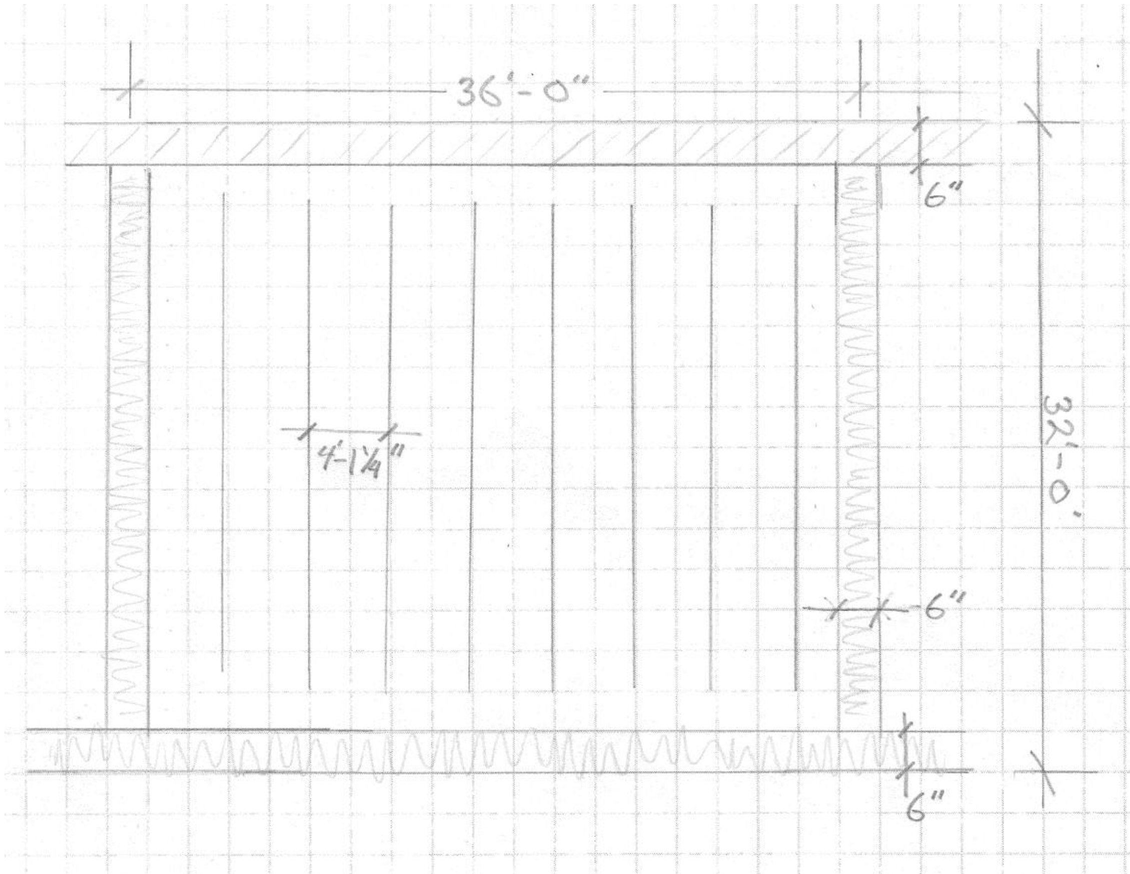
γ = 0.13 Pg +14 = 17.9

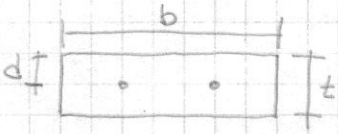
Pd = h_d γ = 47.25 psf



Appendix C: Floor System analysis

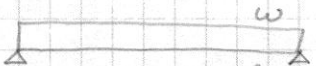
Hambro Floor system (Existing Floor System)





$b = 12''$
$t = 3''$
$f'_c = 3{,}000$ psi
$d = 2.1''$

$f_s = 60{,}000$ psi
6×6 W2.9×W2.9 WWF
$A_{1wire} = .029\ in^2$
$A = .058\ in^2/ft$ $D = .192$
Cover 3/4"



4'-1 1/4" $l_n = 4'$

M: $\omega = $ [SW = 36.2 lb/ft; SDL = 15 psf → 15 lb/ft; LL = 40 psf → 40 lb/ft] 125.4 plf

Ass Simply Supporting

$M_u = \frac{\omega L^2}{8} = \frac{(125.4\,plf)(4)^2}{8}$

$= 250.9$ lb/ft

S: $V_{max} = \frac{\omega L}{2}\ \ \frac{125.4(4.0)}{2} = 250.8$ lb

$\Delta_{max}\ \ \frac{5\omega L^4}{384 E_c I_c}$

$I_c = \frac{bh^3}{12} = \frac{12(3)^3}{12} = 27\ in^4$

$E_c = 57\sqrt{f'_c} = 3122$ Ksi

$\frac{5(125.4)(4)^4 \frac{1728}{1000}}{384 E_c I_c} = .0086$

Moment

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

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{.058(60{,}000)}{0.85(3{,}000)(12'')}$

$= .1137$

$d - a/2 = 2.1 - \frac{.1137}{2} = 2.043$

$\phi M_n = 0.9(.058)(60{,}000)(2.043)$

$= 6399$ lb-in

533 lb-ft/ft ≥ 250.9 ok

$\varepsilon_s = \frac{\varepsilon_u}{c}(d - c)$

$c = a/\beta_1 \quad \beta_1 = 0.85$

$c = .134$

$\varepsilon_u = .003 \quad \varepsilon_s = 0.0441 \geq .005$ ok.

$\rho = A_s/bd = .0023 \geq .0018$ ok.

Shear

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

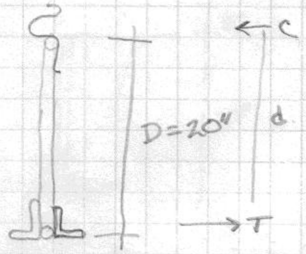
$\lambda = 1.0$ NW

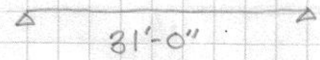
$2\sqrt{3{,}000}(12'')(2.1'') = 2760$ lb

$\phi V_n = .75 V_c = 2070$ lb ≥ 250.8 lb

$L/360 = \frac{(4)(12)}{360} = .1333 > .0086$ ok

ME-01





$w = \begin{cases} LL & 20\text{ psf} & 20 \\ DL & 36\text{ psf slab} & \\ & 5\text{ psf SW} & 41 \end{cases}$

$1.2D + 1.6L = 81.2$ psf

$w = 81.2(4.1458)$

$= 336.6$ plf

$M_u = \frac{wL^2}{8} = \frac{336.6(31)^2}{8}$

$= 40.4^{k\text{-}ft}$

$M_n = A_s f_y d$

$A_s = .56$ in^2

$f_y = 60$ ksi

$d = D + .08 - .6970$

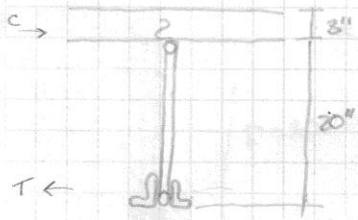
19.38

651.17 k-in

$54.27^{k\text{-}ft}$

$\phi M_n = 0.9(54.27) = 48.8 \geq 40.4^{k\text{-}ft}$ ok

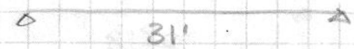
ME-01



$A_s = 1.33\ in^2$
$f_y = 50{,}000$ psi
$f'_c = 3{,}000$ psi

$b_{eff} = \begin{cases} 31'(12)/8 = 46.5'' \\ \tfrac{1}{2}(4.1958)(12) = \boxed{24.9''} \end{cases}$

$d = 20 - .697 = 19.3$



$w = \begin{cases} LL = 40\text{ psf} \\ DL = 36\text{ psf Slab},\ 4\text{ psf Joist},\ 15\text{ psf SDL} \end{cases}$

$1.2D + 1.6L = 130$ psf
$w = 130\text{ psf}(4.1958)$
$w = 539$ PLF

$M_u = \frac{wL^2}{8} = \frac{(539)(31)^2}{8} = 64.7$ K-ft

$L/360 = \frac{31(12)}{360} = 1.033$

$a = \frac{A_s f_y}{0.85 f'_c b_{eff}}$

$M_n = A_s f_y (d + t - a/2)$

$a = \frac{1.33(50)}{0.85(3)(24.9)} = 1.0473 \le 3$

$M_n = 1.33(50)\left(19.3 + 3 - \frac{1.047}{2}\right)$
$= 1448$ K-in $\rightarrow 120.6$ K-ft

$\phi M_n = 0.9(120.6) = 108.5 \ge 64.7$ K-ft

$I = \frac{bh^3}{12} + bh(a/2 - \bar{y})^2 + (n-1)A_s(d - \bar{y})^2$ $\quad n = \frac{29000}{57\sqrt{3000}} = \frac{29000}{3122} = 9.29$

$\frac{(24.9)(3)^3}{12} + 24.9(3)\left(\frac{1.047}{2} - 4.06\right)^2 + (9.29-1)1.33(3.37) = 1027$

$\frac{5}{384}\frac{(539)(31)^4(1728)}{(29000)(1027)(1000)} = .3764$

ME-01

Roof

w = { DL 36 Slap, 5 Joist; LL 20; Snow 21 }

$1.2(41) + 1.6(21) + 20 = 101.6$ psf

$16'(101.6) = 1625.6$ plf

floor

w = { DL 36 Slab, 5 Joist, 15 SDL; LL 40 }

$1.2(56) + 1.6(40) = 131.2$ psf

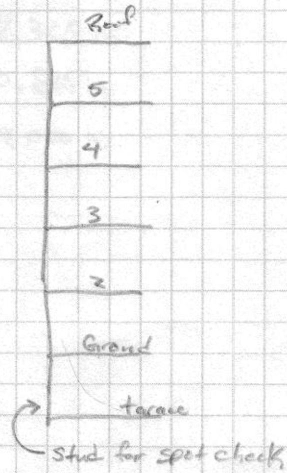
$16(131.6) = 2105.6$ plf

Wall

Ass 15 psf $1.2(15) = 18.0$ psf

$11'(18) = 198$ plf

floor + wall 2303.6 plf



1625.6 plf	Roof
2303.6 plf	5
2303.6	4
2303.6	3
2303.6	2
2303.6	Grand
198	trace

13341.6 plf → 13.34 klf

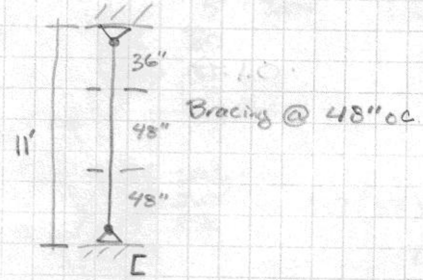
Ass stud are place 16" oc

$13.34\left(\frac{16}{12}\right) = 17.79^{k}/16''$

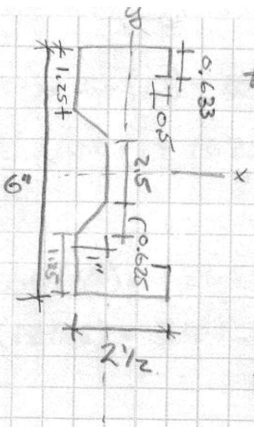
$P_u = 17.79^{k}$

ME-01

6" × 2½" stud



$L = 11' \rightarrow 132$ in



t = 0.0713 in (54 mils)

600 SG250 -54

$A = 0.772\ in^2$

$I_x = 4.183\ in^4$

$I_y = 0.513\ in^4$

$F_y = 50$ ksi

$r_x = 2.328$ in

$r_y = 0.815$ in

$E = 29{,}000$ ksi

$\phi P_n = F_{cr} A_g (0.9)$

$K_y = \frac{48}{132} = .3636 \quad K_x = 1.0$

$\frac{KL}{r} = \frac{.3636(132)}{0.815} = 58.9 \quad \frac{1(132)}{2.328} = 56.7 \leftarrow$ Controlling

$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \quad \frac{\pi^2(29000)}{56.7^2} = 89.0 > 0.44 F_y = 22$

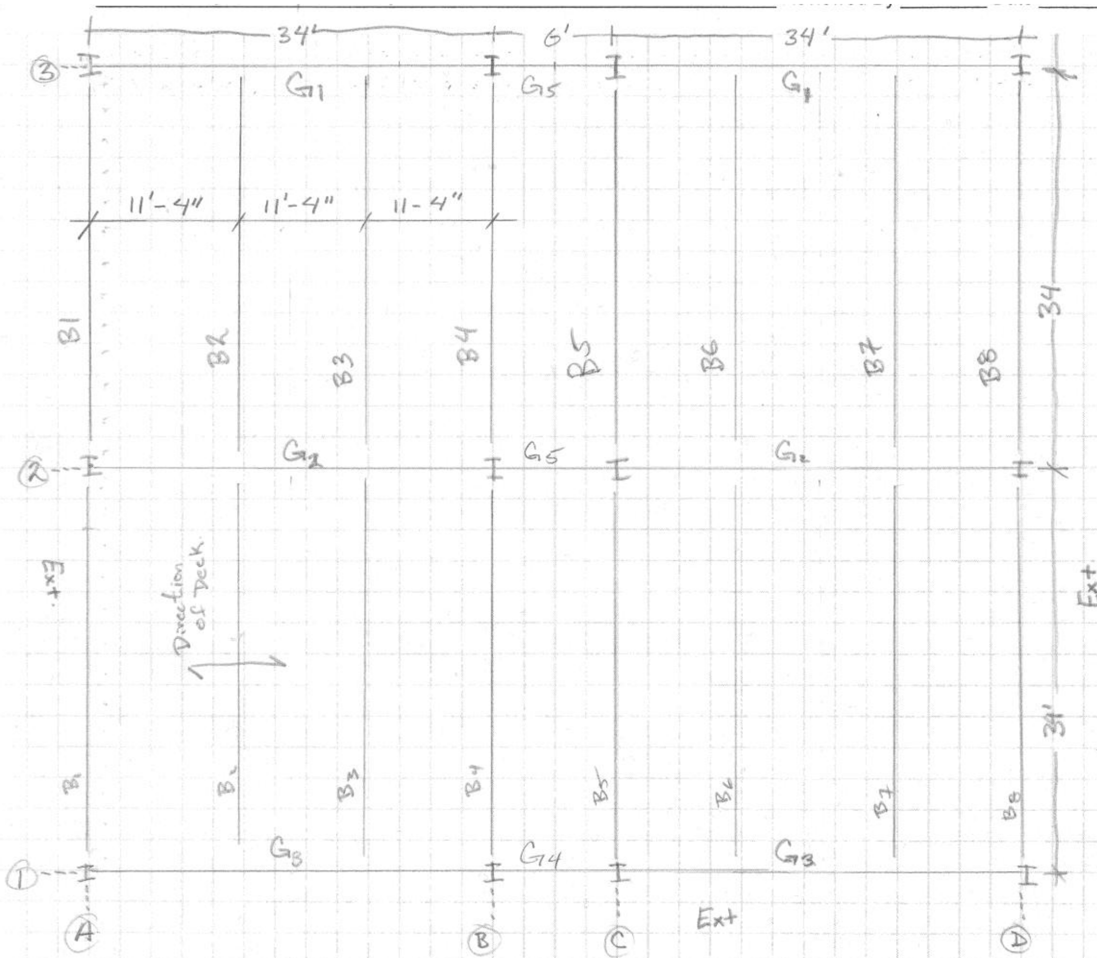
$F_{cr} = \left[0.658^{F_y/F_e}\right] F_y$

$\left[0.658^{50/89}\right] 50 = 39.5$

$\phi P_n = 39.5(0.772)(0.9) = 27.4^k \geq 17.79^k$

ME-01

Composite Steel Beams with Composite Deck



2 Hr Fire Rating t = 3" NW concrete
Valcraft 2VLI17 unshor Clear Span 11'-10" ≥ 11'-4"

Live load : 100 psf
SDL 15 psf
115 psf

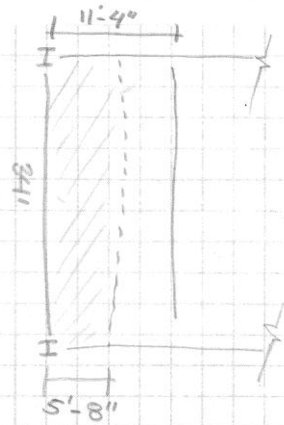
Valcraft catalog p46
@ 11'-6" Superimposed LL = 135 > 115 psf ok

ME-01

Design B1, B8

LL: 50 psf

DL SDL 15 psf
Slab 51 psf
Ass SW 10 psf
Ext wall 15 psf

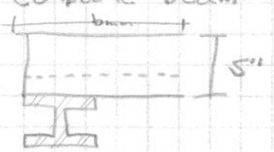


$w_u = [1.2(15+51+10) + 1.6(50)]5'\text{-}8'' +$
$1.2(15)(11') = 1168 \text{ plf} \rightarrow 1.17^{K}/ft$

$M_u = \frac{wl^2}{8} = \frac{1.17(34')^2}{8} = 169 \text{ K-ft}$

$\Delta_{max} = l/360 = 34(12)/360 = 1.13''$

Composite Beam



$b_{eff,min} = \frac{34(12)}{8} = 51''$

Ass' $Y_2 \approx 5-1 = 4$

Deck is perpendicular
Weak studs
1 stud 3/4" stud
$f'_c = 3$ ksi
$Q_n = 17.2^{kip}$

Table 3-19

∴ W12×22
$\phi M_n = 179 > 169$
* $Y_1 = 1.66$
$Y_2 = 4$
$\Sigma Q_n = 117$
$n = 117/17.2 = 6.8 \rightarrow 7 \ (14)$
$22(34) + 14(10) = 888$

∴ W12×19 $\phi M_n = 178 > 169$
$Y_1 = 0.263$
$Y_2 = 4$
$\Sigma Q_n = 174$
$n = 10.1 \rightarrow 11 \ (22)$
$19(34) + 22(10) = 866$

∴ W12×16
$\phi M_n = 177 > 169$
$Y_1 = 0$
$\Sigma Q_n = 236$
$n \ 13.7 \rightarrow 14 \ (28)$
$16(34) + 28(10) = 824$

∴ W10×26
$\phi M_n = 170 > 169$
* $Y_1 = 1.51$
$\Sigma Q_n = 95.1$
$n = 5.5 \rightarrow 6 \ (12)$
$26(34) + 12(10) = 1004$

∴ W10×22
$\phi M_n \ 173 > 169$
$Y_1 = 0.27$
$\Sigma Q_n = 169$
$n = 9.8 \rightarrow 10 \ (20)$
$22(34) + 20(10) = 948$

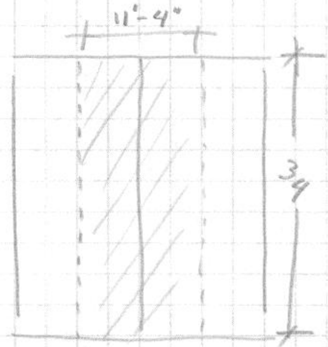
∴ W10×19
$\phi M_n \ 180 > 169$
$Y_1 = 0.0988$
$\Sigma Q_n = 241$
$n = 14.01 \rightarrow 14 (28)$
$19(34) + 28(10) = 926$

∴ W14×22
$\phi M_n = 180 > 169$
* $Y_1 = 3.34$
$\Sigma Q_n = 81.2$
$n = 4.7 \rightarrow 5 \ (10)$
$22(34) + 10(10) = 848$

ME-01

Desig B2, B3, B6, B7

LL: 50psf DL SDL 15
Slab 51psf
Ass SW 10psf



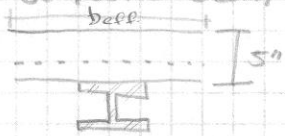
$w_u = [1.2(15+51+10) + 1.6(50)]\,11'\text{-}4"$

$= 1940\,plf \rightarrow 1.94\,klf$

$M_u = \frac{wl^2}{8} = \frac{1.94(34)^2}{8} = 280\,k\text{-}ft$

$\Delta_{max} = L/360 = \frac{34(12)}{360} = 1.13"$

Composite Beam



$b_{eff} = min \left| \frac{34(12)}{8} = \boxed{102"} \right| \frac{(11'-4)(12)(2)}{2} = 136"$

Deck is Perpendicular: weak studs

1 stud 3/4" stud $f'_c = 3\,ksi$ $Q_n = 17.2\,kip$

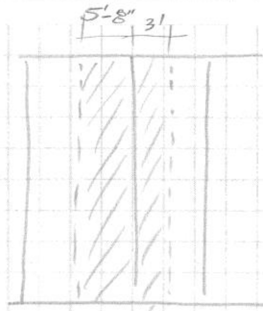
$Y_2 \approx 5 - 1 = 4$

Table 3-19

∴ W12x30	∴ W12x26	∴ W14x26
$\phi M_n = 291 > 280$	$\phi M_n = 290 > 280$	$\phi M_n = 283 > 280$
$Y_1 = 0.22$	$Y_1 = 0$	$Y_1 = 0.21$
$\Sigma Q_n = 296$	$\Sigma Q_n = 382$	$\Sigma Q_n = 279$
$296/17.2 = 17.2 = 18\,(36)$	$n = 22.2 \rightarrow 23\,(46)$	$n = 16.2 \rightarrow 17\,(34)$
$30(34) + 36(10)$	$26(34) + 46(10)$	$26(34) + 34(10)$
1380	1344	1224

ME-01

Design B4, B5



LL 50 Living units
100 Corridors

DL SDL 15
Slab 51
SW 10

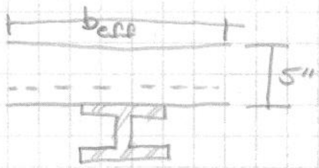
$w_u = 1.2(15+51+10)(8'\text{-}8'') + 1.6(50)5'\text{-}8 + 1.6(100)(3')$

$= 1724 \text{ plf} \rightarrow 1.72 \text{ klf}$

$M_u = \frac{wl^2}{8} = \frac{1.72(34)^2}{8} = 249 \text{ k-ft}$

$\Delta_{max} = 1.13''$

Composite Beam



$b_{eff} = \min \left| \frac{34(12)(2)}{8} = 102 \right| \frac{12(12)+6(12)}{2} = 108$

Deck perpendicular

1 stud 3/4" ϕ $f'_c = 3\text{ksi}$ $Q_n = 17.2$

Ass a ~ 1" $y_2 = 4''$

Table 3-19

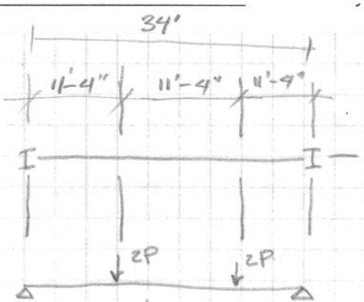
∴ W12x30
$\phi M_n = 268 > 249$
$y_1 = 0.33$
$\Sigma Q_n = 225$
$n = 13.0 = 13 (26)$
$30(34) + 26(10)$
1280

W12x26
ϕM_n 252 > 249
$y_1 = 0.19$
$\Sigma Q_n = 259$
$n = 15 (30)$
$26(34) + 30(10)$
1184

W14x22
$\phi M_n = 252 > 249$
$y_1 = 0.0838$
$\Sigma Q_n = 283$
$n = 16.4 \rightarrow 17 (34)$
$22(34) + 34(10)$
1088

ME-01

G_1, G_2



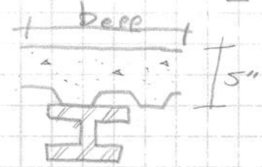
$P = \frac{wl}{2}$ $w = 1.94$ Klf. $L = 34'$

$= \frac{1.94(34)}{2} = 32.98\,K \rightarrow 33^K$

$2P = 66^K$

$M_u = Pa = 66(11'\text{-}4") = 748$ ft-K

$\Delta_{max} = l/360 = 1.13$



Composite Beam

$b_{eff} = \min \left| \frac{34(12)^2}{8} = \boxed{108} \quad \frac{34(12)^2}{2} = 408 \right.$

Ass $a \sim 1"$ $Y_2 = 4"$

Table 3-19

∴ W18x60
$\phi M_n = 785 > 748$
$Y_1 = 0.521$
$\Sigma Q_n = 488$
$n = 28.3 \rightarrow 29(58)$
$60(34) + 58(10) = 2620$

W18x55
$\phi M_n = 757 > 748$
Y_1 0.158
$\Sigma Q_n = 691$
$n = 40.2 \rightarrow 41(82)$
2690

W21x50
$\phi M_n = 769 > 748$
$Y_1 = 0.134$
$\Sigma Q_n = 648$
$n = 37.6 \rightarrow 38(76)$
2460

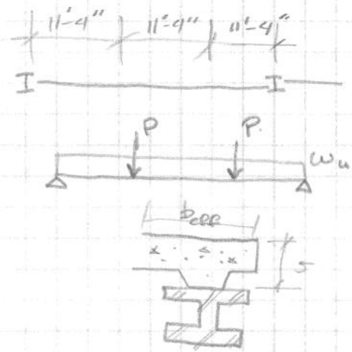
W21x55
$\phi M_n = 774 > 748$
$Y_1 = 0.392$
$\Sigma Q_n = 488$
$n = 28.3 \rightarrow 29(58)$
2450

ME-01

G_3

$P = \frac{wL}{2}$ $w = 1.44$ $L = 34$

$= 33^k$



$w_u = 1.2(15 \cdot 11') = 198$ plf
.198 klf

$M_u = 403$
$\Delta_{max} = 1.13$

Composite Beam

$b_{eff} = \min \left| \frac{34(12)}{8} = 51 \text{ in.}\right.$ Ass $a \sim 1"$ $Y_2 = 4^{1/2}$

Table 3-19

W16×36
$\phi M_n = 427 > 403$
$\Sigma Q_n = 378$
$Y_1 = 0.215$
$n = 21.4 \rightarrow 22(44)$
1664

W14×38
$\phi M_n = 436 > 403$
$Y_1 = 0.129$
$\Sigma Q_n = 471$
$n\ 27.3 \rightarrow 28(56)$
1852

W14×34
$\phi M_n = 412 > 403$
$Y_1 = 0$
$\Sigma Q_n = 500$
$n = 29.0 \rightarrow 29(58)$
1736

ME-01

B1, B8 W14x22

Check a

$a = \frac{81.2}{.85(3)(51)} = 0.62 \quad Y_2 = 5 - \frac{0.62}{2} = 4.69 > 4"$ ok.

ϕM_n Beam Unshored strength

$\phi_b M_p = 125^{k\text{-}ft}$

$w_u = 1.4(51(5'\text{-}8) + 22) = 435$ plf

$w_u = 1.2(51(5.67) + 22) + 1.6(20(5.67)) = 554$ plf

$M_u = \frac{wl^2}{8} \quad \frac{.554(34)^2}{8} = 80.2^{k\text{-}ft} < 125$ ok

Δ_{LL}

$w_{ll} = 50(5.67) = 284$ plf

$\Delta_{ll} = \frac{5}{384}\frac{wl^4}{EI} = \frac{5}{384}\frac{0.284}{29{,}000}\frac{(34)^4}{367}(1728) = 0.8 < 1.13"$ ok

$\Delta_c \quad w = .51(5.67) + 22 = 311.2$

$\Delta_c = \frac{5}{384}\frac{0.311}{29{,}000}\frac{(34)^4(1728)}{199} = 1.62 \nless 1.13$ No good

add 1/2" camber

B2, B3, B6, B7 W14x26

Check a

$a = \frac{279}{.85(3)(102)} = 1.07 \quad Y_2 = 5 - \frac{1.07}{2} = 4.46 > 4"$ ok

ϕM_n Beam Unshored Strength

$\phi_b M_p = 151$

$w_u = 1.2(51(11.33) + 26) + 1.6(20(11.33)) = 1.09$ k/ft

$M_u = \frac{wl^2}{8} = \frac{1.09(34)^2}{8} = 157.5 < 151$ NO

∴ W14x30

$Y_1 = 2.82 \quad \Sigma Q_n = 111 \quad \phi M_n = 251 > 280$

$a = \frac{111}{.85(3)(102)} = .426 \quad Y_2 = 5 - \frac{.426}{2} = 4.79 > 4$ ok

$\phi_b M_p = 177^{'k} > 158$ ok

ME-01

Δ_{LL} $\omega_{ll} = 50(11.33) = .566$ klf

$\Delta_{ll} = \frac{5}{384} \frac{0.566}{29000} \frac{(34)^4}{522}(1728) = 1.12 < 1.13$ ok.

Δ_{Con}

$\omega = 51(11.33)+26) = .604$ klf

$\Delta_{cw} = \frac{5}{384} \frac{0.604}{29000} \frac{(34)^4}{291}(1728) = 2.15 \not< 1.13$

add 1" camber.

B4, B5 W14x22.

Check a

$a = \frac{283}{.85(3)(102)} = 1.09$ $Y_2 = 5 - \frac{1.09}{2} = 4.45 > 4$ ok.

ϕM_n Beam unshored strength.

$\phi_b M_p = 125$

$W_u = 1.2(51(8.67)+22) + 1.6(20(8.67)) = .834$ klf

$M_u = \frac{wl^2}{8} = \frac{0.834(34)^2}{8} = 120^{k\text{-}ft} < 125^{1k}$ ok.

Δ_{LL} $\omega_{LL} = 50(5.67) + 100(3) = .583$

$\Delta_{LL} = \frac{5}{384} \frac{0.583}{29000} \frac{(34)^4}{556}(1728) = 1.09 < 1.13$ ok.

Δ_c $w_c = 51(8.67)+22 = 0.464$ klf

$\Delta_c = \frac{5}{384} \frac{0.464}{29000} \frac{34^4}{199} 1728 = 2.42 \not< 1.13$

add 1½" camber

ME-01

G_1, G_2 **W21×55**

check

$a = \frac{488}{.85(3)(108)} = 1.77 \quad Y_2 = 5 - \frac{1.77}{2} = 4.11 > 4$ ok.

ϕM_n Beam unshored strength

$\phi_b M_p = 473$

$P = \frac{wl}{2} \quad w = 1.09, \; L = 34$

$= 18.5^k \quad 2P = 37^k$

$M_u = 37(11.33) = 419 < 473$ ok

Δ_{LL}

$P_{LL} = \frac{wl}{2} = \frac{0.566(34)}{2} = 9.6 \quad 2P = 19.2^K$

$\Delta_{LL} = \frac{Pa}{24EI}(3L^2 - 4a^2) = \frac{19.2(11.33)}{24(29000)(2400)}(3(34)^2 - 4(11.33)^2)(144) = .022 < 1.13$ ok.

Δ_c

$P_c = \frac{wl}{2} = \frac{(0.604)(34)}{2} = 10.2^K \quad 2P = 20.4^K$

$\Delta_c = \frac{20.4(11.33)}{24(29000)(1140)}(3(34)^2 - 4(11.33)^2)(144) = 0.049 < 1.13$ ok.

G_3 **W16×36**

check a

$a = \frac{378}{0.85(3)(51)} = 2.9 \quad Y_2 = 5 - \frac{2.9}{2} = 3.55 < 4$

Recheck of $\phi M_n = 412 > 403$ ok

ϕM_n Beam Unshored Strength

$\phi_b M_p = 240$

$P = \frac{wl}{2} = \frac{1.09(34)}{2} = 18.5$

$M_u = 18.5(11.33) = 209.6 < 240$ ok

ME-01

Δ_{LL}

$$P_{LL} = \frac{wl}{2} = \frac{0.566(34)}{2} = 9.6^{k}$$

$$\Delta_{LL} = \frac{9.6(11.33)}{24(29000)(1020)}\left(3(34)^2 - 4(11.33)^2\right)(144) = 0.026 > 1.13 \text{ ok}$$

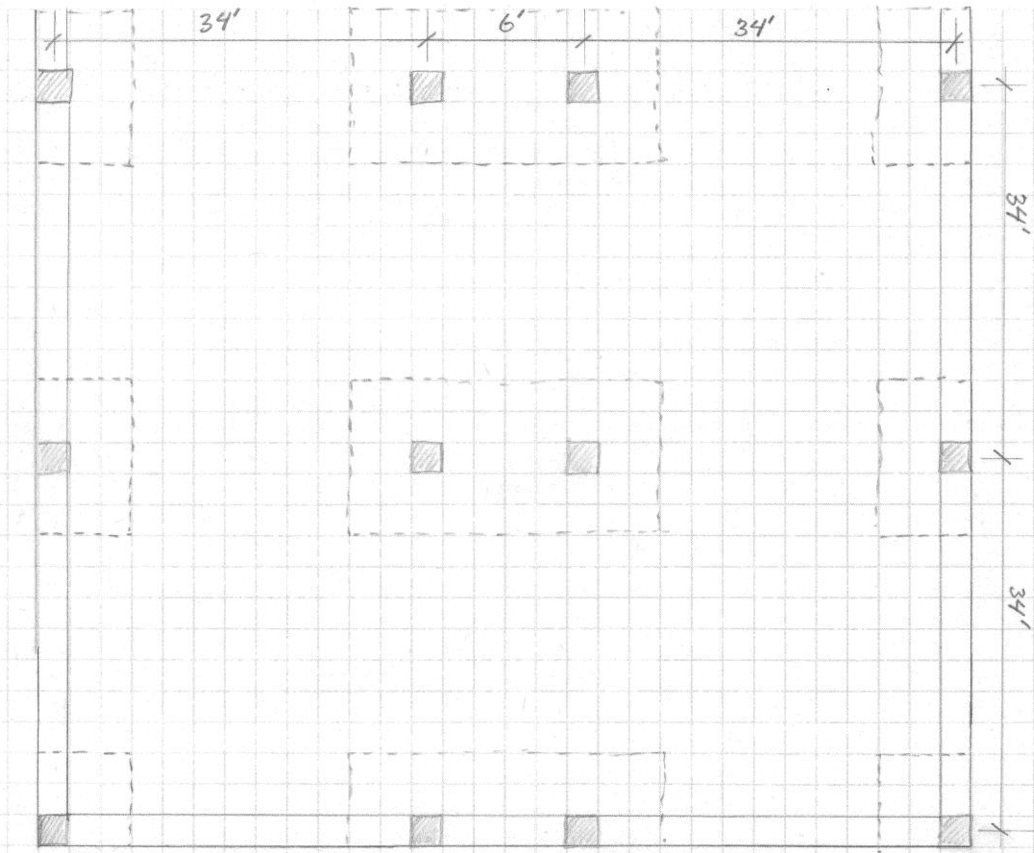
Δ_C

$$P_C = \frac{wl}{2} = \frac{0.604(34)}{2} = 10.2$$

$$\Delta_C = \frac{10.2(11.33)}{24(29000)(448)}\left(3(34)^2 - 4(11.33)^2\right)(144) = 0.062 > 1.13 \text{ ok}$$

ME-01

Two-Way Flat Slab concrete Floor



Flat slab with drop panels with edge beams.

Slab thickness

ACI 318-05 Table 9,5c

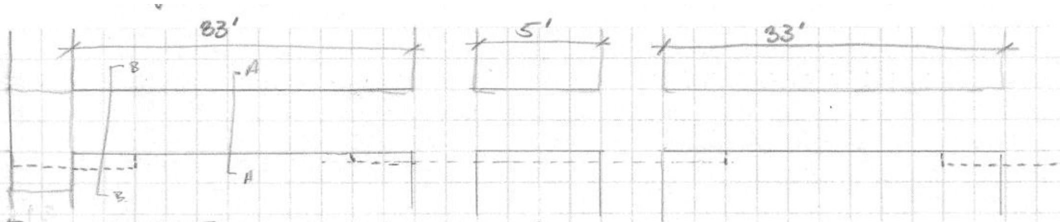
$$h = \frac{l_n}{36} = \frac{33'(12)}{36} = 11''$$

Drop panel

$$t = \tfrac{1}{4}\, 11'' = 2.75''$$

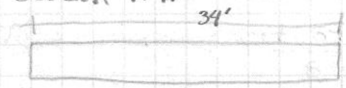
$$\tfrac{1}{6}\, 33' = 5.5'$$

ME-01



Equivalent Frame method must be used

Section A-A slab.



h = 11" $A = 4488\ in^2$

$$I = \frac{34(12)\,11^3}{12} = 45254\ in^4$$

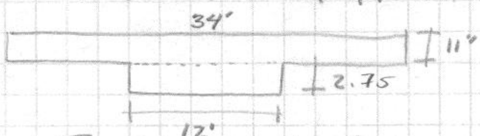
DL 15 psf SDL
137.5 psf Slab
5.09 plf Slab-Drop-panel.

LL 40 psf Living
100 psf Corridor.

$f'_c = 4{,}000$ psi

$E_c = 3605$ Ksi

Section B-B Slab with drop panel



$I = 62{,}544\ in^4$ $A = 4884$

Column section

$$I = \frac{62544}{(1 - 1/34)^2} = 66392\ in^4$$

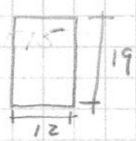
From Computer analysis M_u (K-ft)

1st Span			Middle			3rd Span		
1st Sup	Mid	Sup	Sup	Mid	Sup	Sup	Mid	Sup
-121.2	653.5	-948.1		-954		-948.1	653.5	-121.2

$$l_2/l_1 = \frac{34}{33} = 1.03$$

$\alpha = 0$

$C = 6589$



$$\beta_T = \frac{6589}{2 \cdot 45254} = .072$$

Exterior Span
Column strip interior negative 75%
Exterior 99%
Positive 60%

Int. Span
negative 75%
positive —

ME-01

4488 $\bar{y} = 8.25$ $\quad \bar{y} = 7.69''$ ↑

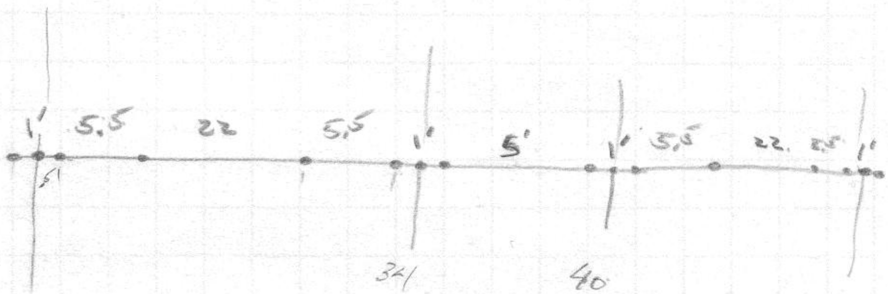
396 $y = 1.375$

$I_1 = I + Ad^2$

$45254 + 4488(8.25-7.69)^2 = 46661$

$90.75 + 396(7.69-1.375)^2 = 15883$

62544



$1.2(15)(34') = .612$ klf

$1.2(137.5 \cdot 34) = 5.610$ klf

$1.2(5.09) = 6.108$ klf

$1.6(40)(~~34~~) = 2.176$ klf

100

$= 5.44$ klf

	Ext Span			Int Span
	Ext Supp	Middle	Int Sup.	
Colum	-120[1K]	392.1[1K]	-711[1K]	-715.5[1K]
Middle	-1.2[1K]	261.4[1K]	-237[1K]	-238.5[1K]

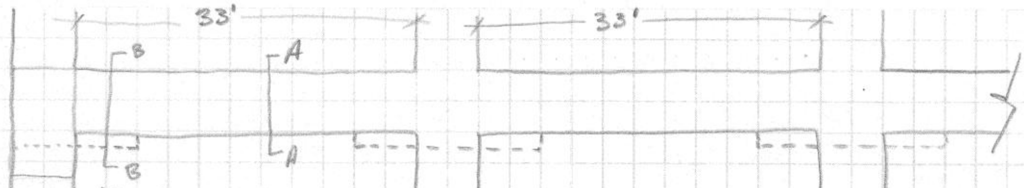
∴ Int span is to short to Develope positive moments.

Required Reinforcement.

		Mu K-ft	b in	d (in)	As. in²	Reinforcement	
Ext span							
Colum	Ext	-120	204	9	3.33	5 #8	$A_s = 3.95 in^2$
	Pos	392.1	204	9	10.89	14 #8	$A_s = 11.06 in^2$
	Int	-711	204	9	19.75	25 #8	$A_s = 19.75 in^2$
Middle	Ext	-1.2	204	9	.0333	1 #8	$A_s = 0.79 in^2$
	Pos	261.4	204	9	7.26	10 #8	$A_s = 7.9 in^2$
	Int	-237	204	9	6.58	9 #10	$A_s = 7.11$
Int Span							
Colum		-715.5	204	9	19.87	26 #8	$A_s = 20.54 in^2$
Middle		-238.5	204	9	6.62	9 #10	$A_s = 7.11 in^2$

$$A_s = \frac{M_u}{4d}$$

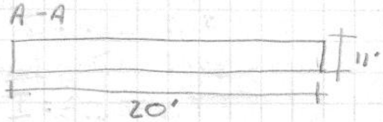
ME-01



Equivalent Frame method

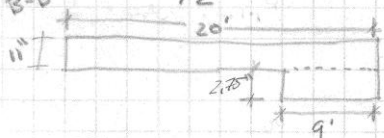
Slab-beam

A-A



$I = \frac{20(12)(11^3)}{12} = 26620\ in^4$ $A = 2640\ in^2$

B-B



$I = 39425\ in^4$ $A = 2937\ in^2$

DL 15 psf SDL
137.5 psf slab
3.06 K/ft slab w/Drop

LL 40 psf living units
100 psf Corridors

$f'_c = 4{,}000\ psi$
$E_c = 3605\ ksi$

Column

$I = \frac{39425}{1 - \frac{1}{20}} = 30973\ in^4$

From Computer Analysis M_u (K-ft)

Ext Span			Int Span	
Ext sup	Midd	Int sup	Int Sup	Middle
-102.7	366.7	-625	-595	200

$\ell_2/\ell_1 = \frac{20}{33} = .61$

$\alpha = 0$

$C = 6589$

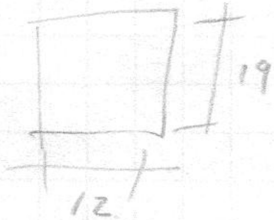
$\beta_T = \frac{6589}{2 \cdot 39425} = .084$

Exteiror Span
Columnstrip interior negative 75%
positive 60%
Exterior 99%

Int. Span
negative 75%
positive 60%

ME-01

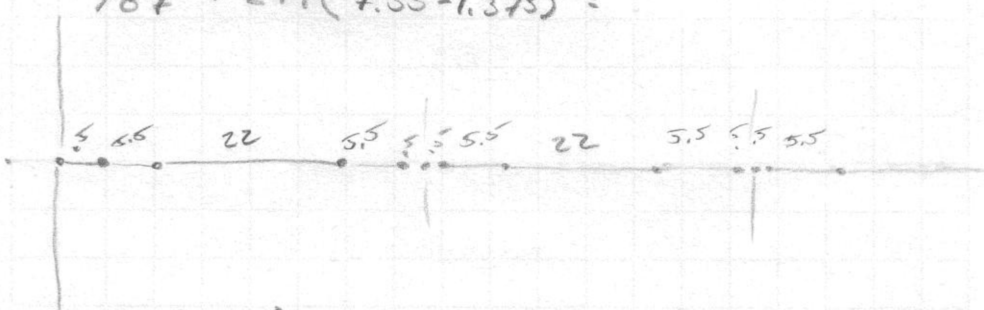
75
74.1



6589

26620 A = 2640 y = 8.25 $\bar{y}$ = 7.55
187 A = 297 y = 1.375

$26620 + 2640(8.25 - 7.55)^2$
$187 + 297(7.55 - 1.375)^2 =$



1.2 (15 · 20) = .36 Klf
1.2 (137.5 · 20) = 3.3 Klf
1.2 (3.06) = 3.67 Klf
1.6 (40)(17) + 1.6(100)(3) = 1.57 Klf

	Ext span			Int span	
	Ext sup	Middle	Int	Int	middle
Column	-101.7^1K	220^1K	-468.7^1K	-446.2^1K	120^1K
Middle	-1.0^1K	146.7^1K	-156.2^1K	-148.8^1K	80^1K

Required Reinforcement

		Mu(ft-K)	b(in)	d(in)	As (in²)	Reinforcement
Ext span						
Column	Ext	-101.7	120	9	2.83	4#8 As=3.16
	Pos	220	120	9	6.11	8#8 As=6.32
	Int	-468.7	120	9	13.0	17#8 As=13.43
Middle	Ext	-1.0	120	9	.028	1#8 As=0.79
	Pos	146.7	120	9	4.07	6#8 As=4.74
	Int	-156.2	120	9	4.33	6#8 As=4.74
Int span						
Column	Int	-446.2	120	9	12.39	16#8 As=12.64
	Pos	120	120	9	3.33	5#8 As=3.95
Middle	Int	-148.8	120	9	4.13	6#8 As=4.74
	Pos	80	120	9	2.22	3#8 As=2.37

$A_s = \frac{M_u}{4d}$

ME-01

Shear.

Beam shear

$V_u = 166^K$

$V_c = 0.75 \cdot \sqrt{4000}\,(34)(12)(9) = 174.2$

$166 < 174.2$ ok

$V_u = 103.6^K$

$V_c = 0.75\sqrt{4000}\,(20)(12)(9.5) = 108.2^K$

$108^K > 103.6^K$ ok

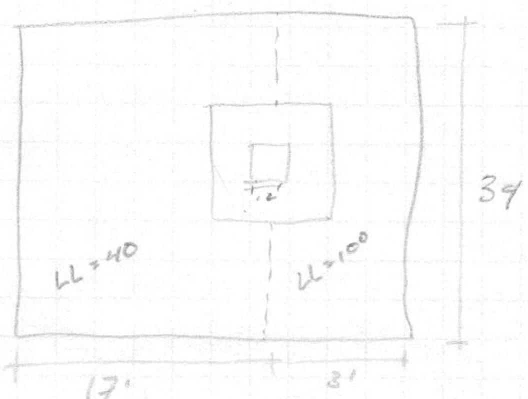
Punching Shear. $d = 11.75$ in

$V_u = 202.7^K$

$V_c = 0.75\sqrt{4000}\,(4.0)(11.75)(95'') = 211.8^K > 202.7^K$ ok

∴ Torsion was not taking into acount in this Report but would need to be looked into.

$d = 11.75$



1.2D + 1.6L

DL	15 psf	18 psf	} 133.86^K
	150 psf	180 psf	
LL	40 psf	64 psf	→ 36.87^K
	100 psf	160 psf	→ 31.96

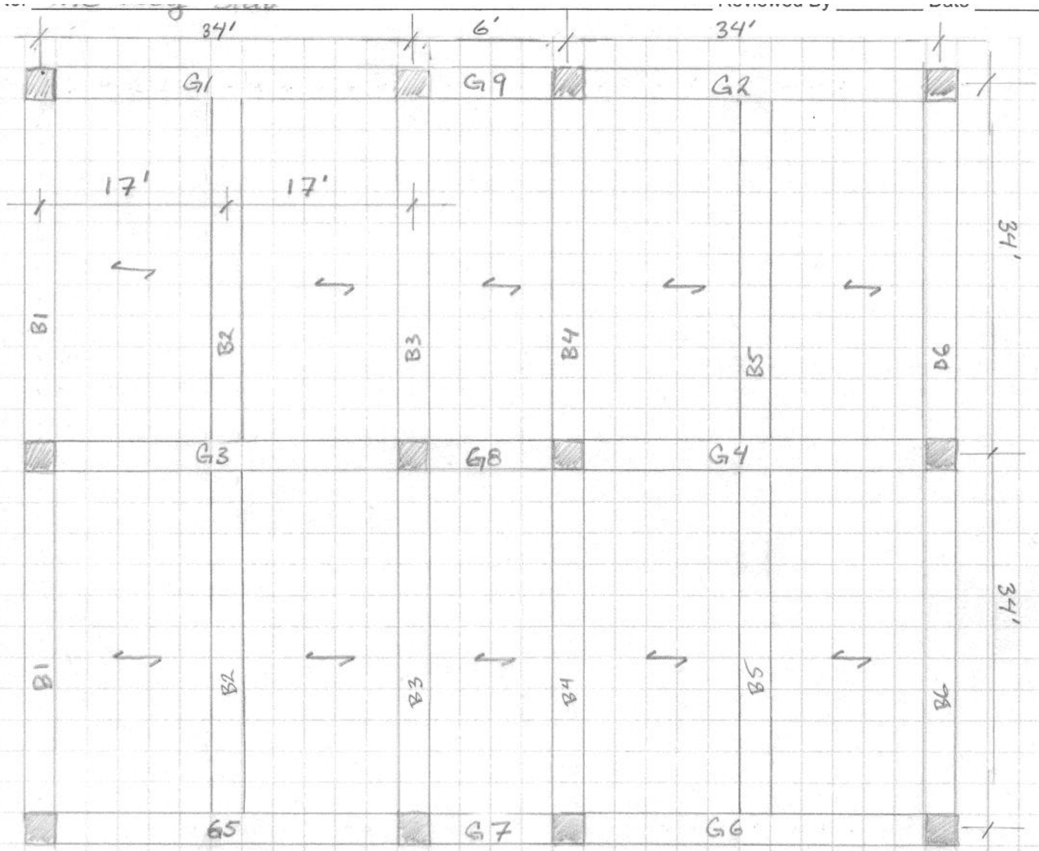
202.7^K

$(34 \cdot 20) = 680$ sf

-3.917 sf

676 sf

One-Way Slab



$f'_c = 3{,}000$.

Slab in living units.

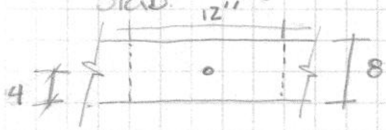


Table 9.5(a)

$h = \frac{l}{24}$ one end continuous.

$h = \frac{(17-1)(12)}{24} = 8''$

Deflection do not need check if using Table 9.5a

LL 40psf

DL SDL 15psf

SW 100psf

$w = 1.2(100+15) + 1.6(40) = 202\,\text{plf}$

$M_u = \frac{wl^2}{8} = \frac{202(16)^2}{8} = 6.46\,'K$

Ass: 12" Beams.

ME-01

Try 1#6 @ 12" oc $A_s = 0.44$

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

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.44(60)}{0.85(3)(12)} = 0.863$

$\beta_1 = 0.85 \quad f'_c \leq 4$ ksi

$c = a/\beta_1 = 0.863/0.85 = 1.01$ $\quad \varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{1.01}(4-1.01) = .0089 > 0.005$

$\phi = 0.9$

$M_n = (0.44)(60)(4 - .863/2)/12 = 7.85^{1K}$

$\phi M_n = 0.9(7.85) = 7.06^{1K} > 6.46^{1K}$ ok

Shrinkag and tempenture reinforcement

$A_s = 0.0018 A_g = 0.0018(12)(8) = 0.173 in^2 \rightarrow$ #4 @12" oc $A_s = 0.2 in^2$

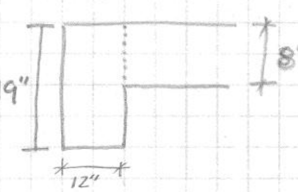
Slab in Corrador LL: 100psf

$w = 1.2(100+15) + 1.6(100)$

$= 298$ plf

$M_u = \frac{298(16)^2}{8} = 9.53$ use 2#6 @ 6" oc

Beam B1, B2



LL 40psf

DL = SDL 15psf
Slab 100psf
SW 137.5plf

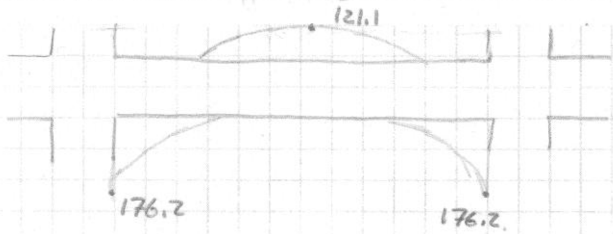
$w = [1.2(15+100) + 1.6(40)] 8' +$
$1.2(137.5) = 1.78^{K}/ft$

Table 9.5a

$h = \frac{l}{21}$ Both ends Continuous

$\frac{33(12)}{21} = 18.85 \rightarrow 19"$

ME-01

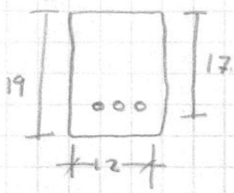


$M_{max}+ \frac{wl^2}{16} = \frac{1.78(33)^2}{16} = 121.1^{1k}$

$M(-)\ \frac{wl^2}{11}\ \frac{1.78(33)^2}{11} = 176.2^{1k}$

$V = \frac{wl^2}{2} = \frac{1.78(33)}{2} = 29.87^{k}$

At midspan $M_u = 121.1$



$A_s = \frac{121.1}{4(17)} = 1.78$ Try 3#7 $A_s = 1.8 in^2$

$M_n = A_s f_y (d - a/2) = 1.8(60)(17 - \frac{3.53}{2})/12 = 137.1^{1k}$

$a = \frac{A_s f_y}{0.85 f'_c\, b} = \frac{1.8(60)}{0.85(3)(12)} = 3.53$

$\beta_1 = 0.85$

$c = a/\beta_1 = 3.53/0.85 = 4.15$

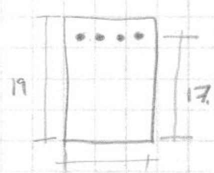
$\varepsilon_s = \frac{0.003}{4.15}(17-4.15) = 0.0093 > .005 \rightarrow \phi = 0.9$

$\phi M_n = 0.9(137.1) = 123.4 \geq 121.1$ ok

$A_{s,min}$ | $\frac{3\sqrt{f'_c}}{f_y} b_w d = 0.56 in^2$

$\frac{200 b_w d}{f_y} = 0.68 in^2$

At supports $M_u = 176.2^{1k}$



$A_s = \frac{176.2}{4(17)} = 2.59$

Try 4#8 $A_s = 3.16$

$a = 6.196$ $\varepsilon_s = 0.004$

$\beta_1 = 0.85$ $\phi = 0.814$

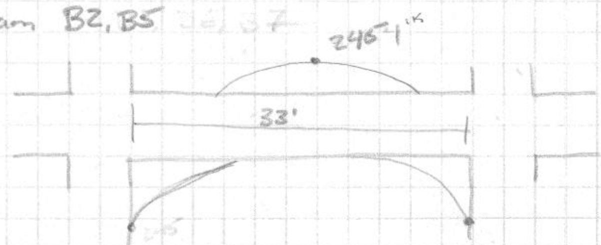
$C = 7.290$ $\phi M_n = 178.8 \geq 176.2$ ok

ME-01

$A_{s\,min}$ | $\frac{3\sqrt{f'_c}}{f_y} b_w d = 0.56$

$\frac{200 b_w d}{f_y} = 0.68$

Beam B2, B5



DL = SDL 15psf
Slab 100psf
SW 137.5plf (19'd Beam)
LL 40psf

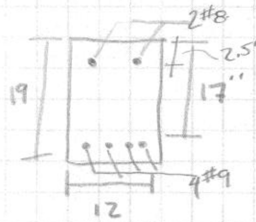
$M_u(+) = \frac{wl^2}{16} = 245^{'K}$

$M_u(-) \; \frac{wl^2}{11} = 356.4^{'K}$

$V = \frac{wl}{2} = 59.4^{K}$

$w = [1.2(15+100) + 1.6(40)]\,17'$
$+ 1.2(137.5) =$
$= 3.6^{K}/ft$

At Midspan $M_u = 245^{'K}$



$A_s = \frac{M_u}{4d} = \frac{245}{4(17)} = 3.6$

Top steel will be required.

Try 4#9 with 2#8
$A_s = 4$ $\quad A'_s = 1.58$

d' = 2.5
d = 17

$M_n = A'_s E_s(.003\frac{c-d'}{c})(d-d') + 0.85 f'_c\, b\, \beta_1 c(d - a/2)$

$A_s f_y = .85 f'_c\, b\, \beta_1 c - A'_s E_s .003(\frac{c-d'}{c})$

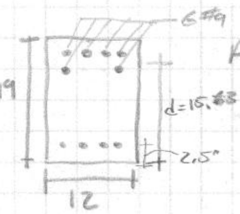
$C = 6.11 \quad a = \beta_1 c \quad \beta_1 = .85 \quad a = 5.19$

$\varepsilon_s = 0.003(\frac{c-d'}{c}) = 0.00177 \le 0.002$

$\phi M_n = 259.8^{'K} \ge 245^{'K}$ ok. $\phi = 0.9$

ME-01

At support $M_u = 356.4^{1K}$



$A_s = \frac{M_u}{4d} = \frac{356.4}{4(17)} = 5.24$

Try 6#9 (2 layers) with 4#10

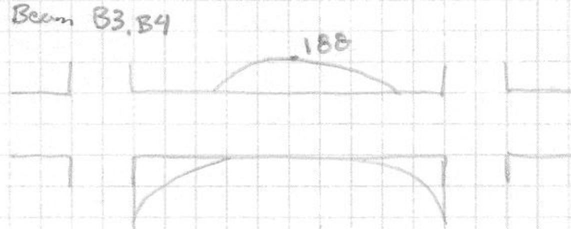
$A_s = 6 \text{ in}^2$ $A_s' = 5.08 \text{ in}^2$

$C = 5.13$ $\beta_1 = 0.85$ $a = 4.36$

$\varepsilon_s' = .003\left(\frac{c-d'}{c}\right) = 0.00154 \leq 0.002$

$\phi M_n = 357.7^{1K} \geq 356.4^{1K}$ ok $\phi = 0.9$

Beam B3, B4



DL = SDL 15 psf
Slab 100 psf
SW = 137.5 plf

LL 40 psf
100 psf

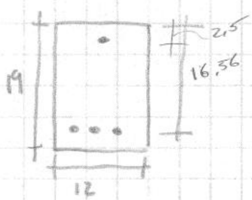
$M_u(+) = \frac{wl^2}{16} = 188^{1K}$

$M_u(-) = \frac{wl^2}{11} = 274.2$

$V = \frac{wl}{2} = 45.7^K$

$w = 1.2(15+100)11.5 + 1.2(137.5) + 1.6(40 \cdot 8.5' + 100 \cdot 3')$
$2.77^{K}/\text{ft}$

At Midspan $M_u = 188^{1K}$



$A_s = \frac{M_u}{4d} = \frac{188}{4(17)} = 2.76 \text{ in}^2$

Try 3#9 with 1#7

$A_s = 3 \text{ in}^2$ $A_s = 0.6 \text{ in}^2$

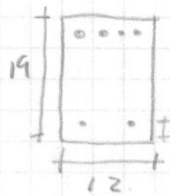
$C = 5.78$ $\beta_1 = 0.85$ $a = 4.91$

$\varepsilon_s' = .003\left(\frac{c-d}{c}\right) = 0.0017 < .002$

$\phi M_n = 190.3^{K} \geq 188^{1K}$ ok.

ME-01

At supports. $M_u = 274.2^{'K}$



$A_s = \frac{M_u}{4d} = \frac{279.2}{4(17)} = 4.03$

Try 4 #10 with 2 #9

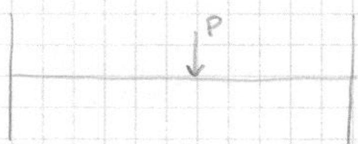
$A_s = 5.08$ $A_s' = 2 in^2$

$C = 7.32$ $\beta_1 = 0.85$ $a = 6.22$

$\varepsilon_s' = 0.00197$ $\varepsilon_t = .00379 < .005 \rightarrow \phi = 0.8$

$\phi M_n = 277^{'K} \geq 274.2^{'K}$ OK

Girder G1, G2, G3, G4



$P = 2\frac{wl}{2} = 2 \cdot 59.4 = 118.8^K$

$w = 1.2(137.5) = 165$ plf

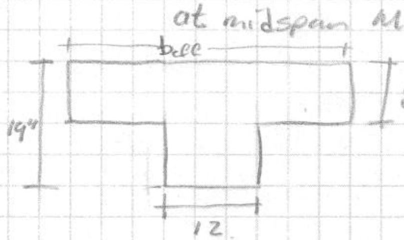
from computer analysis

$M_u^+ = 509^{'K}$

$M_u(-) = 517^{'K}$

$V_u = 61.9^K$

at midspan $M_u = 509$



b_{eff} | $1/4$ span $= 1/4\ 33(12) = \boxed{99}$
$b_w + 16h_f = 12 + 16 \cdot 8 = 144$
$b_w + \sum \frac{1}{2} CD = 12 + 33 \cdot 12 - 13 = 396$

Check T-Beam

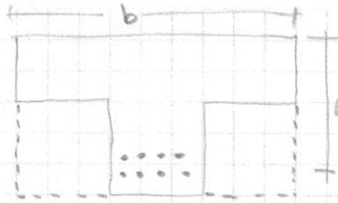
$M_{uT\text{-}B} = \phi .85 f'_c\ b \cdot h_f(d - h_f/2)$

$0.9 \cdot .85(3)(99)(8)(17 - 8/2)/12 = 1969^{'K} > 509$ No T-Beam
Rect Beam with b = 99"

Try 8 #9

$A_s = 8 in^2$

ME-01



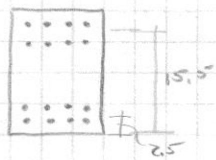
$a = 1.9 \quad \beta_1 = 0.85 \quad c = 2.24$

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

$\phi M_n = 0.9 \cdot 8(60)(15.5 - \frac{1.9}{2})/12 = 524 > 509$ OK

At support $M_u = 517$

Try 4#10, 4#9 with 8 #9



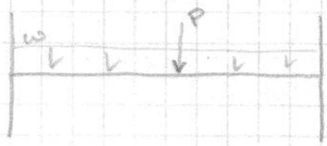
$a = 4.94 \quad \beta_1 = 0.85 \quad c = 5.77$

$\varepsilon'_s = 0.0017 < .002$

$\varepsilon_t = .0056 \rightarrow \phi\, 0.9$

$\phi M_n = 532 > 517$ OK

Girder G5, G6.



$P = \frac{wl}{2} = 59.4$

$w = 1.2(137.5 + 15 \cdot 11) = 0.363$ klf

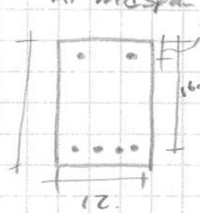
From computer analysis.

$M_{u+} = 270^{1k}$

$M_{u-} = 287^{1k}$

$V_u = 35.9^k$

At midspan $M_u = 270$

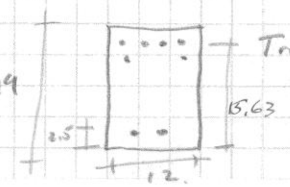


Try 4 #10 with 2#9

$\phi M_n = 277 > 270$ OK

∴ See Calc for B3, B4 at supports for detail calculation.

At support $M_u = 287$



Try 6 #8 with 2#9

$A_s = 4.74\,in^2 \quad A'_s = 2\,in^2$

ME-01

$C = 6.321$ $\beta_1 = 0.85$ $a = 5.373$

$\varepsilon'_s = 0.003 > .002$

$\varepsilon_t = 0.00486 \longrightarrow \phi = 0.888$

$\phi M_n = 296.2'^k > 287'^k$ ok

∴ for all Beams and Girders #3 stirrups were assumed but was not calculated for this Report

∴ Torsion in exterior beams were not consider in this report, but is a design concern.

Unless noticed, all images and figures were created by Ryan English.

Appendix D: Revision of Technical Report 1.

Wind/seismic Load Calculation

Under carful review of the calculation of the wind and seismic load calculation it was found that an analytical error was found in the wind load analysis. It was found that the wrong leeward wind pressure was used in the calculation of the base shear and over turning moment. After recalculation of the wind load, for the E-W direction the base shear was found to be 245 kips with an over turning moment of 8,188 kip-ft and for the N-S direction the base shear was found to be 249 kips with an over turning moment of 7,989 kip-ft. The seismic load was check for errors and none was found. At the same time the seismic load was re-compared to the values for the structural document and was less than 5% off form their values. The seismic base shear was 1355 kips with an over turning moment of 63,704 kip-ft.

Snow Drift

Snow drift was analysis and the calculation can be found in Appendix B.