Rockville Metro Plaza II

121 Rockville Pike
Rockville, Maryland

Technical Report III

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Executive Summary

Figure 1: Rockville Pike Entrance - JMV

Rockville Metro II is the second part of a three phase project that will aid in revitalizing its community. The building is planned to bring new retail venues and Class A office space to the Rockville, MD area. In September of 2011, construction began on this ten story structure.

The structure was planned to have three levels of below grade parking. An initial geotechnical report concluded that the soil at this level would be adequate to support the structure on concrete footings alone. The only concern found was that the water level could exceed this elevation. Thus damp-proofing measures were taken in the design.

The entire structural system is built using cast-in-place concrete. The lower levels of the structure (parking and retail levels) use flat plate, two-way slabs with mild reinforcing to support the floors. Columns which bear these levels incorporate drop caps in order to better resist punching shear forces. The upper levels of the structure (the office spaces) also use a flat plate slab with mild reinforcing to support the floors. However, in order to facilitate a more flexible office space, larger column-to-column spans (40 feet) were designed. This required additional support of the slabs. To achieve this, wide, shallow post tensioned beams were added to the design. These aided in the control of deflection as well as reduced the potential for cracking. All live loading was determined using ASCE 7 as a guide.

In order to respond to the potential for lateral loads on the structure such as seismic and wind, shear walls were incorporated into the structural design. These walls were placed at the center of the structure about the elevator core. These walls were designed to be 12” thick with rebar reinforcing. ASCE 7 also aided in determining the loading conditions for these elements. The roof of the structure is specified as a green roof. MET II is set to achieve a LEED rating of Platinum, and the green roof is one of the attributes that will aid in this achievement.

In April of 2013, construction on MET II concluded, and MET II became the National Headquarters for Choice Hotels. The following report will describe the structural systems of MET II in more depth. The structure will be analyzed as originally designed and built. Cagley and Associates is responsible for the original design the structural system of MET II and has provided all structural drawings for this report.

Report Summary

The focus of Technical Report III is to assess the floor system of the structure as it was originally designed. Analysis of Rockville Metro Plaza II's typical office bay will compare the loading to the capacity of various members. Among those members assessed are the one-way floor slab and the typical beam supporting the slab as well as typical exterior and interior columns. Results found through this report show that all members assessed were designed within capacity to meet strength and serviceability requirements.

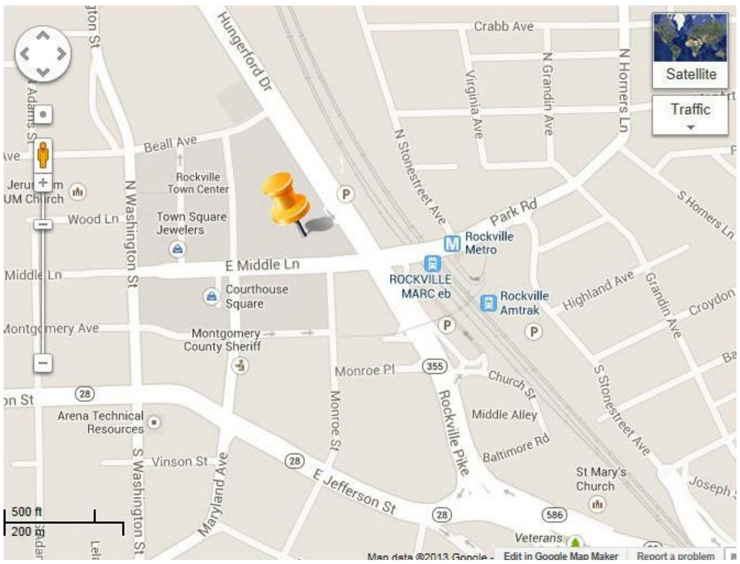
Figure 2: South West Corner – by JMV

The next progression of this report sought to assess possible floor systems alternatives. For this section, the following systems were analyzed for the typical office bay configuration: non-composite steel system, composite steel system, and hollow core slab system. In order to provide a more controlled result, the bay size was maintained at 20' x 40'. This report contains hand calculation for each design as well as computer results to support the findings. An evaluation of each floor system based on various criteria such as weight, cost, floor depth, etc. is also provided for comparison of systems.

The original concrete system yielded the shallowest floor system depth which is critical in order to apply to the height restrictions of the Metro D.C. area. Each of the steel options yielded favorable values across most comparison categories, though depth of these systems may be an issue since they are slightly deeper than the concrete option. It was concluded that a hollow core slab system would not be a feasible system due to the high cost and the difficulty of constructing areas of varying geometries (e.g. curved wall). A full summary and comparison of the floor systems may be found in the remainder of this report.

Site Location

Rockville Metro Plaza II is located in Rockville, Maryland, just 20 miles northwest of the heart of Washington D.C. The site sits prominently on Rockville Pike which is one of the main routes through the area. Across from the lot is the Rockville Metro stop. With such close proximity to these passage ways, this site boasts a transportation convenience for both employees and visitors alike.



The bustling Rockville area is primarily occupied by businesses, retail, restaurants, and high rise apartments. It is an ever expanding and reawakening locale, as new construction projects continually rejuvenate the lively scene. Upon visiting the area, it can be quite evident why Choice Hotels would decide to make MET II the site of their new North American Headquarters.

Figure 3: Map of Site Location – From "maps.google.com"

The new construction of MET II would be an addition to the current Rockville Metro Plaza I to the Northwest. This posed a complication during construction, for impact on MET I's daily function had to be minimized as much as possible. Excavation of the addition would be required to yield to the existing structure as well.

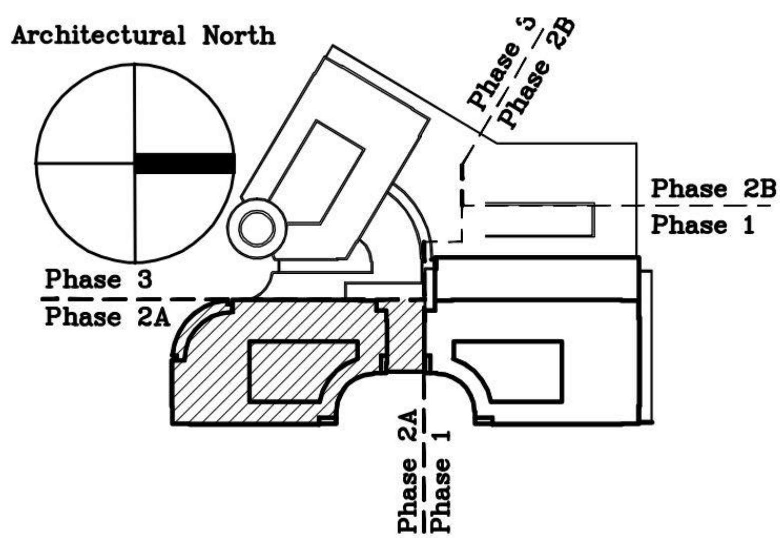


Figure 4: Map of Building Relations – by WDG Arch.

Design Codes

As defined on page S1.00 of the construction documents, the following codes are applicable to the design and construction of MET II's structural system and will also be used in the calculations included in this report:

- "The International Building Code-2009",
  International Code Council

- "Minimum Design Loads for Buildings and Other Structures" (ASCE 7),
  American Society of Civil Engineers

- "Building Code Requirements for Structural Concrete, ACI 318-02",
  American Concrete Institute

- "ACI Manual of Concrete Practice – Parts 1 Through 5",
  American Concrete Institute

- "Post Tensioning Manual",
  Post Tension Institute

Figure 5: Rockville Town Square Obelisk – by JMV

Gravity Loads

Floor Loads

Rockville Metro II utilizes multiple floor systems to comprise its structure. On the office levels, floors are generally comprised of one-way slab systems on a 20’ by 40’ bay. These slabs are carried by wide, shallow post tension beams which transfer loads to the building’s columns. On the parking levels below grade, a two-way slab system is used. These levels are mapped by 26’ x 20’ bays and thus better suited to be designed as two way slabs.

Garage Slab Loads

Within MET II, the below grade parking garage comprises levels P1, P2, and P3. OF these, 2 and 3 are elevated 8” slabs comprised of normal weight concrete and mild reinforcing.

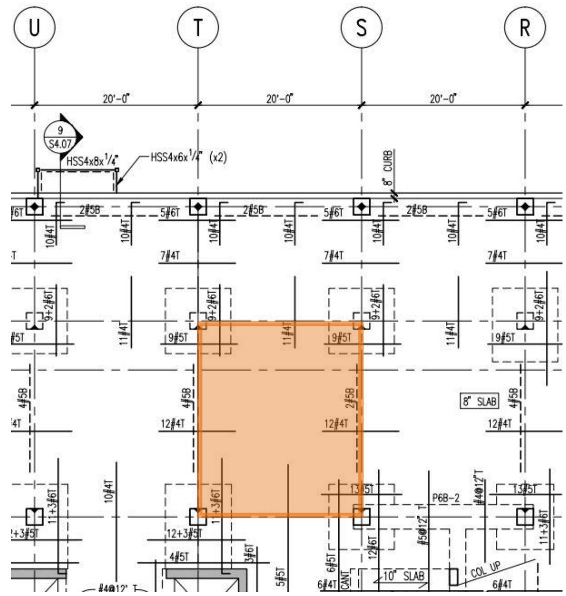


Figure 6: Plan of Garage Bay – by Cagley and Assoc.

These lower levels do not have the need for as large of an open space as compared to the office areas. The span here is governed by the diving aisle width that the International Building Code requires. Thus, the slab is designed to the 26’ x 20’ bay size. Since the aspect ratio is squarer, the section can be designed as a two-way slab system.

In terms of loading, the slab itself once again contributes most of the dead load on the floor system. Such items mechanical and lighting equipment are relatively light and are accounted for in the super imposed dead load. There is no flooring material installed on top of the slab and no hanging ceiling system below. The occupancy live load is defined in the IBC as a garage load of 40 psf (passenger vehicles only). However, the design uses a load of 50 psf which is the minimum load for truck and bus garages.

Table 1: Garage Loads

Type	Load Value (psf)
Slab	100
SDL	5
Live	50

Office Slab Loads

Within MET II, office space comprises the 4th through 11th floors. Due to the consistency in layout for level to level, a typical slab design is used for each level. This is comprised of an 8" normal weight concrete slab with mild reinforcing.

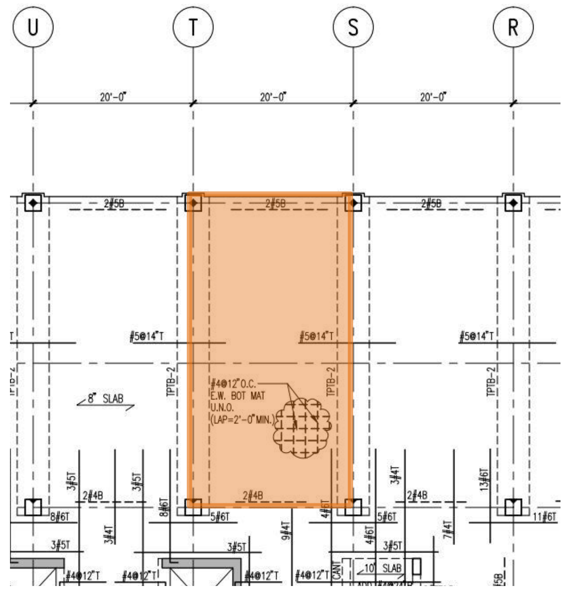


Figure 7: Plan of Office Bay – by Cagley and Assoc.

In order to create a larger open space in the layout, the typical bay is designed at 20' x 40' (as seen in figure 6 to the right). This open floor plan allows the tenant of the space to have more flexibility in how they want to organize the space. Due to the uneven aspect ratio of the bay, the slab acts as a one-way system. The slab is reinforced with a bottom mat made of #4 bars at 12" on center.

In terms of loading, the slab itself contributes most of the dead load on the floor system. Such items as flooring, hanging ceiling tiles, and mechanical/lighting equipment are relatively light and are accounted for in the super imposed dead load. The occupancy live load as designed and defined in the IBC is an office load of 80 psf with an additional 20 psf for the possibility of partitions installed in the space.

Table 2: Office Loads

Type	Load Value (psf)
Slab	100
SDL	5
Live (Occupant)	80
Live (Partition)	20

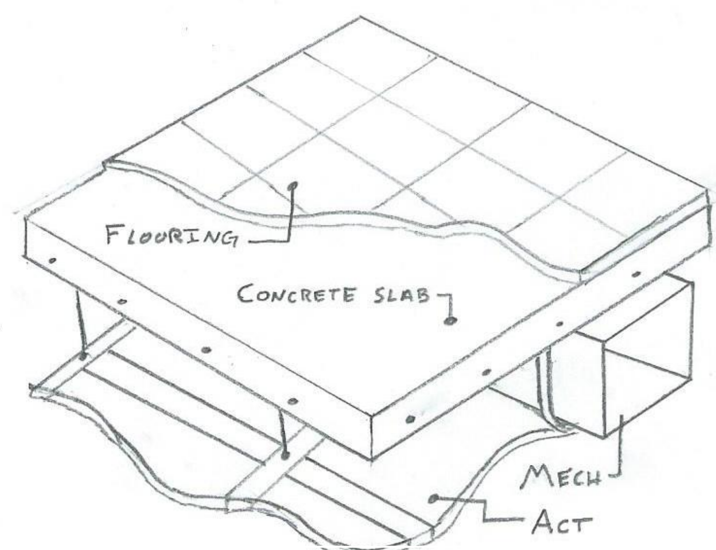


Figure 8: Cut Away of Typical Floor Slab – by JMV

Roof Slab Loads

In pursuit of a LEED rating, the roof of MET II was designated as a green roof composition. Green roofs are a more environmentally friendly alternative to the standard roof. They reduce heat island effects, reduce rainwater runoff (which lessens the potential for sewer overflow), and provide a habitat for birds and insects, as well as many other benefits. For the structure, however, this can equate to a heavier roof as there will be more mass present than that of a standard roof. The roof is designated as an extensive green roof which means that the vegetation will mainly grasses and similar small plants (e.g. sedum). These plants have relatively shallow root systems and thus do not require a deep soil base, as only a 4" depth is used.

In order to support the roof, a concrete slab is used in a similar configuration as seen on the office levels: an 8" concrete slab comprised of normal weight concrete and #4 bars as reinforcing. The bays are 40' x 20' and the roof slab act as a one-way system and wide, shallow post tension beams are provided to transfer the load to columns.

In terms of loading, the slab itself contributes most of the dead load on the floor system. Hanging loads for the ceiling below are accounted for in the super imposed dead load. The green roof also contributes to the dead load. Live loads are as governed by IBC and ASCE 7. The controlling load is a roof live load of 30 psf for ponding (as the snow load and occupant load were determined to b 17.5 psf and 20 psf respectively).

Table 3: Roof Composition

Item	Design Value (psf)
Vegetation	1
Soil	29
Filter/ Moisture Mat	2
Insulation	3
Roof Membrane	5
Slab	100
SDL	10

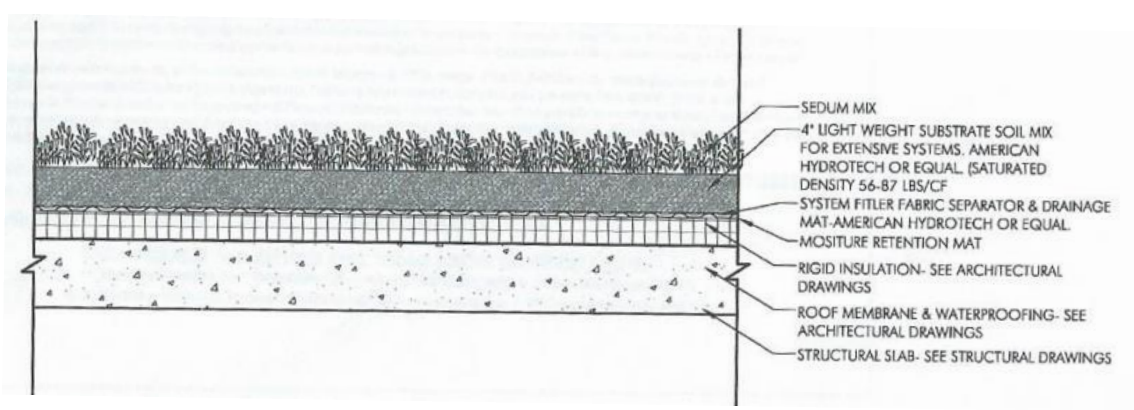


Figure 9: Green Roof Cross Section – by Studio 39

Exterior Wall Load

Rockville Metro II is enclosed by a wall system comprised of precast concrete panels and aluminum framed glass windows. This system is attached to the structural system's slabs and columns.

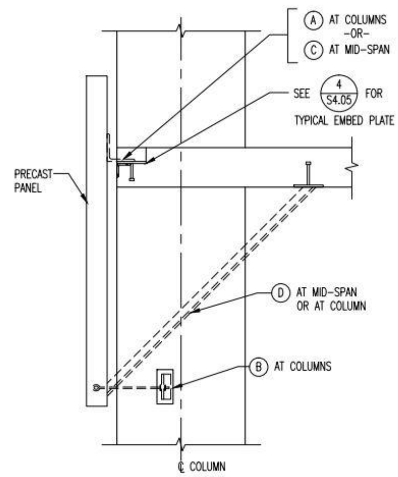


Figure 10: Precast Elevation Detail - by Cagley and Assoc.

Each precast panel spans between two exterior columns. Two connections are made at each column and to the slab at mid-span. These connections are both load bearing and non-load bearing (as seen in figure 9). The load bearing connections (i.e. support weight of panel) only occur at the columns. Other connections act to tie back the panel to the structure and to resist loads perpendicular to the panel. Figure 9 depicts the tie back connections and the fact that they occur at two different elevations at each connection point.

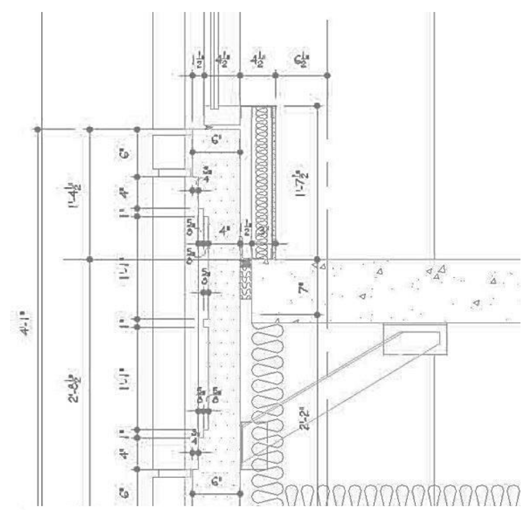


Figure 11: Wall Elevation Section - by Cagley and Assoc.

The aluminum framed window system is set between the precast panels, thus their load bears on the panels. Cold formed steel studs and the remaining wall components such as insulation and dry wall bear directly onto the concrete slab. In designing the structural system of the building, a line load of 500 plf was used by the structural engineer to estimate the load of the wall configuration. During the design stage, this load would be applied to the slab, and would in turn be transferred to the columns. In actuality, the load of the precast concrete panel is directly transferred to the columns. The only load the slab sees comes from lateral loads and from the interior wall components that are set directly on the slab.

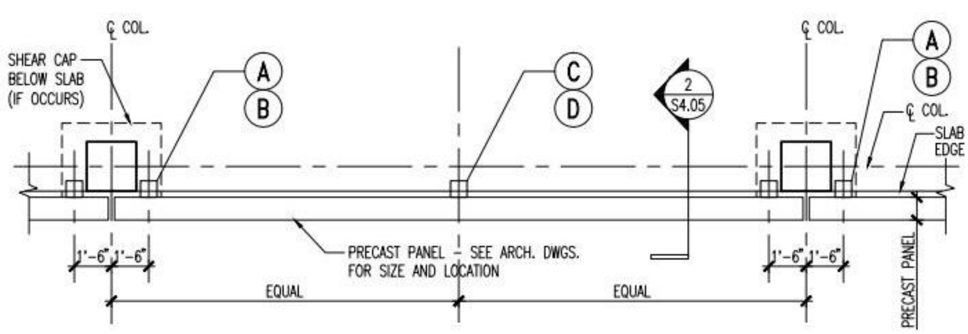


Figure 12: Precast Plan Detail – by Cagley and Assoc.

Gravity Load Summary

In comparing the design values provided on the structural documents to those listed in the International Building Code and ASCE 7, it is evident that all live load requirements were met or exceeded. The main areas of where this trend is evident are mechanical rooms and office areas. Each of these spaces were designed with higher live loads most likely due to the owner's specification, anticipated actual loading, or the simply the office's standard practice for good design. The comparison of live load values may be seen in Table 4 below.

ASCE 7 was used in calculating the flat roof snow load of the structure. Using this document as a guide, the same value as presented on the structural documents was derived. This calculation can be seen in Table 5 below. Snow drift was not considered in this report. The super-imposed values presented below in Table 6 are also as listed on the structural documents.

Table 4: Floor Live Loads

Area	As Designed (psf)	ASCE 7-05 (psf)
Corridors (first level)	100	100
Corridors (above first)	100	80
Lobbies	100	100
Marquees/Canopies	75	75
Mechanical Room	150 (U)	125
Offices	80 + 20 (partitions)	50 + 20 (partitions)
Parking Garage	50	40
Retail – First Floor	100	100
Stairs/Exit Ways	100 (U)	100
Storage (Light)	125 (U)	125

Table 5: Flat Roof Snow Load

Ground Snow Load	P_g=	25 psf
Snow Exposre Factor	C_e=	1.0
(Terrain Category B)		
Thermal Factor	C_t=	1.0
Importance Factor	I_s=	1.0
$P_f = 0.7*P_g*C_e*C_t*I_s*P_g =$		17.5 psf

Table 6: Superimposed Dead Loads

Area	Design Value (psf)
Floor	5
Roof	10

As-Built System - Gravity Loads

One-Way Concrete Slab & PT Beams

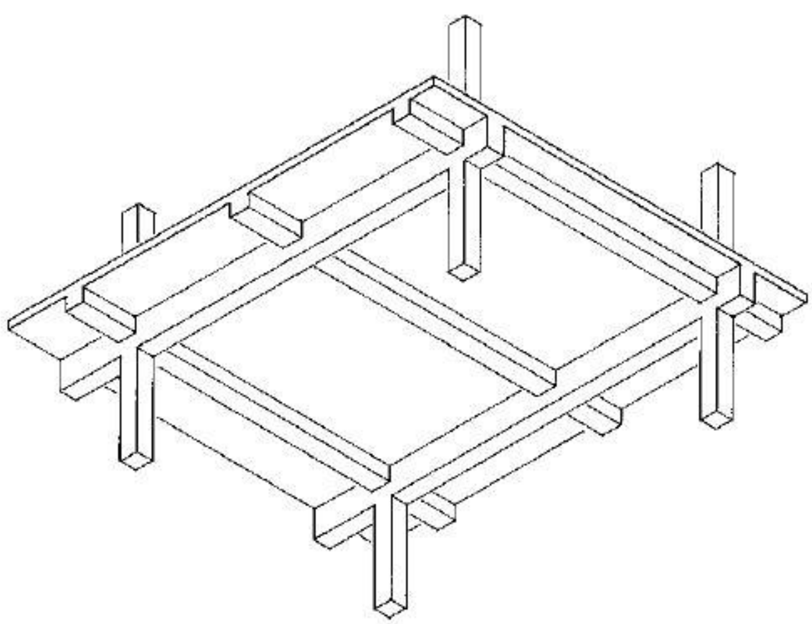


Figure 13: One-Way Concrete Slab & Beam
by RS Means

One-Way Concrete Slab & PT Beams

The originally designed structural system of Rockville Metro Plaza II's typical bay is a one-way slab and post tensioned beam system. This system is constructed entirely of reinforced concrete. It employs a one-way concrete slab that spans the North-South direction. The post tension beams span the perpendicular direction and are used to support the slab.

This system provides many benefits. The wide, shallow post tensioned beams allow for a shallow depth of the floor system. This in turn means more stories within the height restriction, and thus more leasable space. This characteristic also yields taller floor to ceiling heights which may be interpreted as an architecturally satisfying feature as the space will appear larger and possess a more open feel.

The monolithic construction style along with specific rebar detailing allow for the building's beam-to-column connections to act as moment frames and aid the main lateral system in withstanding lateral loads. The structure primarily employs concrete shear walls at its core as the main lateral force resisting system.

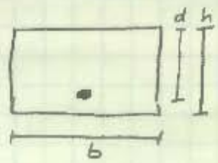
Another positive aspect of the system is that it uses concrete as the structural medium. Within the D.C. area, this medium is widely used and very familiar to most construction companies. This in turn equates to construction companies and their workers being quite knowledgeable and skilled regarding the construction of concrete buildings. They will be able to approach the project with confidence and construct the project in a competitive time frame.

One of the drawbacks to this system is that the mass of the building will be quite large. This in turn means that the structure must be designed to higher earthquake loads as opposed to a lighter system (e.g. steel). Also, the larger mass means that the foundation will have to be larger in order to accommodate the weight.

This system will be compared against alternatives within this report. The comparison section found in the latter portion of this document will further address the pro's and con's of this system when weighed against alternatives.

J.M.V. | Tech 3 Sys 1

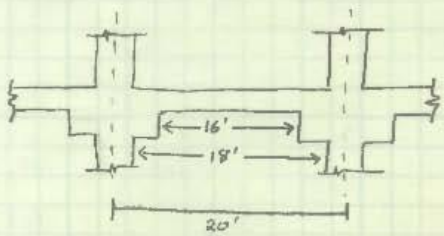
Typical Office Floor Slab - Mid-Span Moment



$d = 7"$
$h = 8"$
$b = 12"$

(1) #4 bar
3/4" clear cover
$d_b = 0.5"$

$f'_c = 4{,}500$ psi
$f_y = 60{,}000$ psi
$A_s = 0.20 \text{ in}^2$



Dead: $\frac{8}{12}" \times 150$ pcf $= 100$ psf
Super: 5 psf
Live: 80 psf + 20 psf

$1.2D + 1.6L =$
$1.2(105) + 1.6(100) = 286$ psf

Slab capacity:

$\beta_1 = 0.85 - \frac{0.05}{1000}(4500 - 4000) = 0.825$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.2)(60)}{0.85(4.5)(12)} = 0.261$ in $\qquad c = a/\beta_1 = 0.3169$

$\varepsilon_s = \frac{0.003}{c}(d - c) = \frac{0.003}{0.3169}(7 - 0.3169) = 0.0633 > 0.005 \quad \therefore \phi = 0.9$

$M_n = 0.85 f'_c a b (d - a/2) = 0.85(4.5)(0.261)(12)(7 - \frac{0.261}{2}) = 82.43$ kip in

$\phi M_n = 0.9(82.43)\frac{1}{12} = 6.18$ kip-ft

Moment from Loading

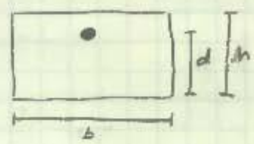
$M_u = \frac{w_u l_n^2}{14} = \frac{(0.286)(16)^2}{14} = 5.23$ kip-ft [End span]

$M_u = \frac{w_u l_n^2}{16} = \frac{0.286(16)^2}{16} = 4.58$ kip-ft [Interior Span]

$\phi M_n > M_u \quad \therefore$ ok (design confirmed)

J.M.V. | TECH 3 | SYS 1

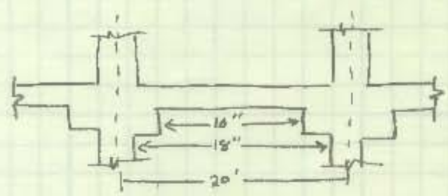
Typical Office Floor Slab - Moment at Beam Support



$h = 8"$
$d = 6.94"$
$b = 14"$

(1) #5 bar @ 14"
3/4" clear cover
$d_b = 0.625"$

$f'_c = 4{,}500$ psi
$f_y = 60{,}000$ psi
$A_s = 0.31\ in^2$



Dead:
$8/12" \times 150$ pcf $= 100$ psf
Super:
5 psf
Live:
80 psf + 20 psf

$1.2D + 1.6L =$
$1.2(105) + 1.6(100) = 286$ psf

slab capacity:

$\beta_1 = 0.85 - \frac{0.05}{1000}(4500-4000) = 0.825$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.31)(60)}{0.85(4.5)(14)} = 0.347$ $\qquad c = a/\beta_1 = 0.421$

$\varepsilon_s = \frac{0.003}{c}(d-c) = \frac{0.003}{0.421}(6.94 - 0.421) = 0.0465 > 0.005 \quad \therefore \phi = 0.9$

$M_n = 0.85 f'_c ab(d - a/2) = 0.85(4.5)(0.347)(14)(6.94 - \frac{0.347}{2}) = 125.85$ kip-in

$\phi M_n = (0.9)(125.85)\frac{1}{12} = 9.439$ kip ft

Moment from Loading

$M_u = \frac{w_u l_n^2}{24} = \frac{(0.286)(16)^2}{24} = 3.05$ Kip-ft [Exterior edge]

$M_u = \frac{w_u l_n^2}{10} = \frac{(0.286)(16)^2}{10} = 7.32$ Kip-ft [Exterior of 1st interior]

$M_u = \frac{w_u l_n^2}{12} = \frac{(0.286)(16)^2}{12} = 6.10$ Kip-ft [Interior]

$\phi M_n > M_u \quad \therefore$ ok (design confirmed)

J. M.V. | Tech 3 | Sys 1

Deflection of Slab

Minimum Thickness - Table 9.5(a)

$$h = \ell/28 = 20/28\,(12) = 8.57'' \quad > \quad 8'' \text{ provided}$$

Max Permissible deflection -

$$\ell/360 = \frac{20(12)}{360} = 2/3\,'' \quad \text{[immediate defl. due to LL]}$$

Actual Deflection -

$$n = E_s/E_c = \frac{29{,}000}{57000\sqrt{4500}} = 7.58$$

$$\text{centroid} = \frac{(8)(12)(4) + (7.58-1)(0.2)(1)}{8(12) + 7.58(0.2)} = 3.96'' \quad \therefore \ \bar{y}_{top} = 4.04'',\ \bar{y}_{bot} = 3.96''$$

$$I_g = \frac{12(8)^3}{12} + (8)(12)(0.04)^2 + (7.58-1)\left[\frac{\pi(0.25)^4}{4} + (0.2)(3-0.04)^2\right] = 523.7\ in^4$$

$$M_{cr} = \frac{f_r I_g}{y_t} = \frac{7.5\sqrt{4500}\,(523.7)}{(4.04)(12)(1000)} = 5.435\ k\cdot ft$$

Negative moments: let D = 105 psf & L = 100 psf

$$\text{Dead:} \quad \frac{(0.105)(16)^2}{11} = -2.44\ kip\,ft$$

$$\text{Live+Dead:} \quad \frac{(0.205)(16)^2}{11} = -4.77\ kip\,ft$$

[Note: only interior spans considered]

Positive moments: let D = 105 psf & L = 100 psf

$$\text{Dead:} \quad \frac{(0.105)(16)^2}{14} = 1.92\ kip\,ft$$

$$\text{Live+Dead:} \quad \frac{(0.205)(16)^2}{14} = 3.75\ kip\,ft$$

Immediate deflections $\Delta i = k \frac{5}{48} \frac{M_a \ell^2}{E_c I_e}$ where $k = 1.2 - 0.2\, M_o/M_a$

$$\Delta i_D = \left[1.2 - 0.2\left(3.36/1.92\right)\right]\left(5/48\right)\frac{(1.92)(20)^2}{(3824)(523.7)} = 0.059''$$

$$\Delta i_{D+L} = \left[1.2 - 0.2\left(6.56/3.75\right)\right]\left(5/48\right)\frac{(3.75)(20)^2}{(3824)(523.7)} = 0.115''$$

$$\Delta i_L = 0.115 - 0.059 = 0.0560'' < 0.66'' \quad \therefore \underline{ok}$$

Long term deflection

$$\lambda = \frac{\xi}{1 + 50\rho'} = \frac{2.0}{1+0} \quad \text{[maximum possible value : 5yr case]}$$

$$\Delta_{\infty LS} = \lambda\,(\Delta i)_{sus} = (2.0)(0.5\,\Delta i_L) = 0.056\ in$$

$$\Delta_{\infty D} = \lambda(\Delta i)_D = (2.0)(0.059) = 0.118\ in$$

$$\Delta_{\infty} = \Delta i_D + \Delta i_L + \Delta_{\infty D} + \Delta_{\infty LS} = 0.059 + 0.056 + 0.118 + 0.056$$

$$\Delta_{\infty} = 0.17'' < 0.66'' \quad \therefore ok$$

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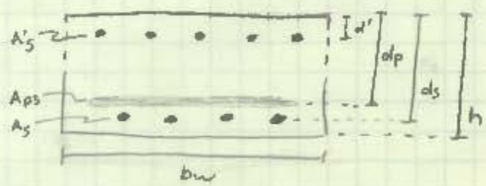
Post tension Beam

single drape
1.5" clear cover
#4 stirrups



Top bars : (5) #9
Tendons : (20) 7 strands unbonded
Bottom bar : (4) #9

Geometry

$h = 20"$
$b_w = 48"$
$b_{eff} = 120"$

$d' = 2.564"$
$d_p = 17"$
$d_s = 17.436"$

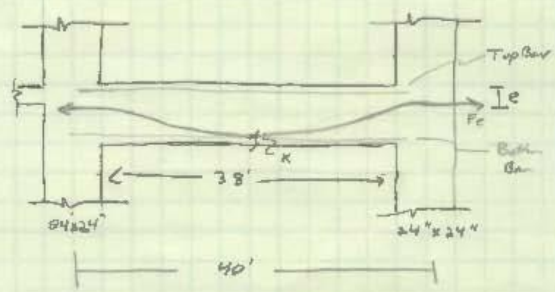
$A'_s = 5 \text{ in}^2$
$A_{ps} = 3.06 \text{ in}^2$
$A_s = 4 \text{ in}^2$

$L = 40'$
$L_n = 38'$
$h_f = 8"$

Materials

$f_y = 60$ ksi
$f_{pu} = 270$ ksi
$f'_c = 4.5$ ksi

Elevation view



Loading

Live = 1250 plf
Super = 100 plf
Slab = 1600 plf
Self = 1000 plf
Fe = 540 kips

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Beam Values for Analysis

$A'_s = (5)(1.00) = 5\ in^2$

$A_{ps} = (20)(0.153) = 3.06\ in^2$

$A_s = (4)(1.00) = 4\ in^2$

$$b_{eff} = \min \begin{cases} \tfrac{1}{4}\text{span} = \tfrac{1}{4}(40 \times 12) = 120\ in \\ b_w + 16h_f = 48 + 16(8) = 176\ in \\ b_w + \text{clear} = 48 + (24 \times 12 - 48) = 240\ in \end{cases}$$

Live = $(100\ psf \times 20\ ft) = 2000\ plf$ → Reduced $L_r = L_o\left(.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$

$K_{LL}A_T = 40 \times 40$

$\therefore L_r = 2000(0.625) = 1250$

Super = $(5\ psf \times 20\ ft) = 100\ plf$

Slab = $(150\ pcf \times 8/12 \times [20-4]) = 1600\ plf$

Self = $(150\ pcf \times 20/12 \times 4\ ft) = 1000\ plf$

$d' = 1.5''(\text{clear}) + 0.5''(\#4\ \text{stirrup}) + \tfrac{1}{2} \times 1.128(\#9) = 2.564\ in$

$d_s = 20 - d' = 17.436''$

$d_p = 20'' - 3''(\text{given value}) = 17''$

$\beta_1 = 0.85 - \frac{0.005}{1000}(f'_c - 4000) = 0.825$

J.M.V. | TECH 3 | SYS 1

Solve for f_{ps}:

$\frac{F_e}{A_{ps}} = f_{pe} \geq 0.5 f_{pu}$ $\rightarrow$ $176.5 = \frac{540}{3.06} \geq 0.5(270) = 135$ ok

Analyze as unbonded tendons

Span to depth ratio: $38 \times 12 / 20 = 22.8 \leq 35$

Thus f_{ps} may be approximated by following equation

$$f_{ps} = f_{pe} + 10{,}000 + \frac{f'_c}{100 \rho_p} \qquad \text{where } \rho_p = \frac{A_{ps}}{b\, d_p}$$

$$= 176.5 + 10 + 4.5/[100 \times 3.06/(148 \times 17)] = 198.5 \text{ KSI}$$

$$f_{ps} \leq f_{py},\ f_{se} + 60{,}000 \qquad \text{where } f_{py} \approx 0.85 f_{pu}$$

$$198.5 \leq 229.5,\ 136.5 \qquad \text{ok}$$

Solve for a:

$$a = \frac{A_{ps} f_{ps} + A_s f_y - A'_s f_y}{0.85 f'_c\, b_{eff}}$$

$$= \frac{(3.06)(198.5) + (4)(60) - (5)(60)}{0.85\,(4.5)(120)} = 1.193 \text{ in} \leq h_f = 8'' \quad \text{ok}$$

$$c = a/\beta_1 = 1.193/0.825 = 1.446 \text{ in}$$

$\frac{0.003}{1.446}(17 - 1.446) = 0.03 > 0.005$
$\therefore \phi = 0.9$

Solve for ϕM_n:

$$M_n = A_{ps} f_{ps} (d_p - a/2) + A_s f_y (d - a/2) + A'_s f_y (d' - a/2)$$

$$= (3.06)(198.5)\left(17 - \tfrac{1.193}{2}\right) + (4)(60)\left(17.436 - \tfrac{1.193}{2}\right) + (5)(60)\left(2.564 - \tfrac{1.193}{2}\right)$$

$$= 14595.38 \text{ kip in}$$

$$\phi M_n = 0.9\,(14595.38)\, \tfrac{1}{12} = 1094.65 \text{ kip-ft}$$

Solve for M_u:

$$M_u = \frac{w_u \ell_n^2}{8} = \frac{5.24\,(38)^2}{8} = 945.8 \text{ kip-ft}$$

Check Design:

$\phi M_n > M_u$ $\quad 1094 > 945$ $\quad$ ok

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Solve for f_{ps}:

$F_e/A_{ps} = f_{pe} \geq 0.5 f_{pu} \rightarrow 176.5 = 540/3.06 \geq 0.5(270) = 135$ ok

Analyze as unbonded tendons

Span to depth ratio : $38 \times 12/20 = 22.8 \leq 35$

Thus f_{ps} may be approximated by following equation:

$f_{ps} = f_{pe} + 10{,}000 + \frac{f'_c}{100\rho_p}$ where $\rho_p = \frac{A_{ps}}{b_w d_p}$

$= 176.5 + 10 + 4.5/100(3.06/[38 \times 12]) = 198.5$ ksi

$f_{ps} \leq f_{py}, \; f_{se} + 60{,}000$ where $f_{py} \approx 0.85 f_{pu}$

$198.5 \leq 229.5, \; 136.5$ ok

Solve for a: (neglect top steel)

$$a = \frac{A_{ps} f_{ps} + A_s f_y}{0.85 f'_c b_{eff}}$$

$$= \frac{(3.06)(198.5) + (4)(60)}{0.85(4.5)(120)} = 1.85 \leq h_f = 8'' \text{ ok}$$

$c = a/\beta_1 = 1.85/0.825 = 2.24$ $\frac{0.003}{2.24}(17 - 2.24) = 0.02 \gg 0.005 \therefore \phi = 0.9$

Solve for ϕM_n:

$M_n = A_{ps} f_{ps}(d_p - a/2) + A_s f_y (d - a/2)$

$= (3.06)(198.5)(17 - \frac{1.85}{2}) + (4)(60)(17.436 - \frac{1.85}{2})$

$= 13728.36$ kip-in

$\phi M_n = 0.9(13728.36)\frac{1}{12} = 1029.63$ kip-ft

Solve for M_u:

$M_u = \frac{w_u l_n^2}{8} = \frac{5.24(38)^2}{8} = 945.8$ kip-ft

Check Design:

$\phi M_n > M_u$ $1030 > 946$ ok

Project Name:	Rockville Metro II
Job Number:	Tech 3
Date:	10/7/2013
Engineer:	JMV
Column ID:	U7

Level	Load Type	Height (ft)	Size (in x in)	Trib, A_T (ft^2)	Cum Trib (ft^2)	*K_{LL} (IBC)	Design LL (psf)		Allowable Reduction	Reduced LL (psf)	Slab t_s (in)	P_{Slab} (kips)	P_{Drop} (kips)	P_{Beams} (kips)	P_{Facade} (kips)	P_{Col} (kips)	SDL (psf)	P_{SDL} (kips)	P_{LL} (kips)	P_{DL} (kips)	ΣP_{LL} (kips)	ΣP_{DL} (kips)	ΣP_{Total} (kips)	ΣP_u (kips)
PH Roof	Roof	18	0 x 0	0	0	4	0		0.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.0
Roof	Mech	12	24 x 24	405	405	4	35	(U)	0.00	35	8	41	0	12	10	7.2	50	20	14.2	90	14.18	90	104.1	130.6
11	Floor	12	24 x 24	405	810	4	100		0.51	51	8	41	0	12	10	7.2	10	4	20.8	74	34.97	164	198.7	252.4
10	Floor	12	24 x 24	405	1215	4	100		0.47	47	8	41	0	12	10	7.2	10	4	18.8	74	53.81	237	291.3	371.0
9	Floor	12	24 x 24	405	1620	4	100		0.44	44	8	41	0	12	10	7.2	10	4	17.7	74	71.48	311	382.7	487.8
8	Floor	12	24 x 24	405	2025	4	100		0.42	42	8	41	0	12	10	7.2	10	4	16.9	74	88.36	385	473.3	603.3
7	Floor	12	24 x 24	405	2430	4	100		0.40	40	8	41	0	12	10	7.2	10	4	16.3	74	104.6	459	563.3	717.9
6	Floor	12	24 x 24	405	2835	4	100		0.40	40	8	41	0	12	10	7.2	10	4	16.2	74	120.8	532	653.3	832.3
5	Floor	12	24 x 24	405	3240	4	100		0.40	40	8	41	0	12	10	7.2	10	4	16.2	74	137	606	743.2	946.7
4	Floor	12	24 x 24	405	3645	4	100		0.40	40	8	41	0	12	10	7.2	10	4	16.2	74	153.2	680	833.2	1061.1
P3	Garage	18	24 x 24	140	3785	4	50		**0.80**	40	8	14	0	0	10	10.8	10	1	5.6	36	158.8	716	875.0	1113.5
P1	Retail	9	24 x 24	140	3925	4	100	(U)	0.00	100	9	16	0	0	0	5.4	10	1	14	23	172.8	739	911.5	1163.0
BP1	Garage	9	24 x 24	140	4065	4	50		**0.80**	40	8	14	0	0	0	5.4	10	1	5.6	21	178.4	760	937.9	1196.9
BP2	Garage	9	24 x 24	140	4205	4	50		**0.80**	40	8	14	0	0	0	5.4	10	1	5.6	21	184	780	964.3	1230.8
BP3-SOG					4205														0	0	184	780	**964.3**	1230.8

*K_{LL}, the live load element factor, was obtained from IBC Table 1607.9.1:

Interior columns	4
Exterior columns w/o cant slab	4
Edge columns w/ cant slab	3
Corner columns w/ cant slab	2

Foundation Design:

Allowable bearing (psf):	10000
Required footing area (ft^2):	96.43
Square Footing Size:	9.82

*LL reduction calculated using IBC eqn 16-1.

*LL reduction is **not** to be used:

1. For LL > 100 psf at the roof level. (For LL>100 for floors other than roof, the max allowable reduction is 20%.)
2. For LL <= 100 psf in public assembly areas.
3. For any roof loading.
4. For parking garages, the max allowable reduction is 20% for columns supporting **2** or more floors. Otherwise, it is 0%.

T.E.C.- Building "A"

Project Name: Rockville Metro II
Job Number: Tech 3
Date: 10/7/2013
Engineer: JMV

Column ID: S3

Level	Load Type	Height (ft)	Size (in x in)	Trib, A_T (ft^2)	Cum Trib (ft^2)	*K_{LL} (IBC)	Design LL (psf)		Allowable Reduction	Reduced LL (psf)	Slab t, (in)	P_{Slab} (kips)	P_{Drop} (kips)	P_{Beams} (kips)	P_{Facade} (kips)	P_{Col} (kips)	SDL (psf)	P_{SDL} (kips)	P_{LL} (kips)	P_{DL} (kips)	ΣP_{LL} (kips)	ΣP_{DL} (kips)	ΣP_{Total} (kips)	ΣP_u (kips)
PH Roof	Roof	18	24 x 24	250	250	4	35	(U)	0.00	35	8	25				10.8	15	4	8.75	40	8.75	40	48.3	70.2
Roof	Roof			250	500	4	35	(U)	0.00	35	8	25		18		0	50	13	8.75	56	17.5	95	112.6	162.8
Roof	Mech	12	24 x 24	320	820	4	150	(U)	0.00	150	8	32				7.2	100	32	48	71	65.5	166	231.8	304.3
11	Floor	12	24 x 24	570	1390	4	100		0.45	45	8	57		18		7.2	30	17	25.7	99	91.22	266	356.8	464.6
10	Floor	12	24 x 24	570	1960	4	100		0.42	42	8	57		18		7.2	10	6	23.9	88	115.1	353	468.6	608.3
9	Floor	12	24 x 24	570	2530	4	100		0.40	40	8	57		18		7.2	10	6	22.8	88	137.9	441	579.3	750.3
8	Floor	12	24 x 24	570	3100	4	100		0.40	40	8	57		18		7.2	10	6	22.8	88	160.7	529	690.0	892.3
7	Floor	12	24 x 24	570	3670	4	100		0.40	40	8	57		18		7.2	10	6	22.8	88	183.5	617	800.7	1034.2
6	Floor	12	24 x 24	570	4240	4	100		0.40	40	8	57		18		7.2	10	6	22.8	88	206.3	705	911.4	1176.2
5	Floor	12	24 x 24	570	4810	4	100		0.40	40	8	57		18		7.2	10	6	22.8	88	229.1	793	1022.1	1318.1
4	Floor	12	24 x 24	570	5380	4	100		0.40	40	8	57		18		7.2	10	6	22.8	88	251.9	881	1132.8	1460.1
P3	Garage	18	24 x 24	430	5810	4	50		**0.80**	40	8	43	4			10.8	10	4	17.2	63	269.1	943	1212.5	1562.6
P1	Retail	9	24 x 24	430	6240	4	100	(U)	0.00	100	9	48	4			5.4	10	4	43	62	312.1	1006	1317.9	1706.4
BP1	Garage	9	24 x 24	430	6670	4	50		**0.80**	40	8	43	4			5.4	10	4	17.2	57	329.3	1063	1392.2	1802.4
BP2	Garage	9	24 x 24	430	7100	4	50		**0.80**	40	8	43	4			5.4	10	4	17.2	57	346.5	1120	1466.5	1898.5
BP3-SOG					7100														0	0	346.5	1120	**1466.5**	1898.5

*K_{LL}, the live load element factor, was obtained from IBC Table 1607.9.1:

Interior columns	4
Exterior columns w/o cant slab	4
Edge columns w/ cant slab	3
Corner columns w/ cant slab	2

*LL reduction calculated using IBC eqn 16-1.
*LL reduction is **not** to be used:

1. For LL > 100 psf at the roof level. (For LL>100 for floors other than roof, the max allowable reduction is 20%.)
2. For LL <= 100 psf in public assembly areas.
3. For any roof loading.
4. For parking garages, the max allowable reduction is 20% for columns supporting **2** or more floors. Otherwise, it is 0%.

Foundation Design:

Allowable bearing (psf):	10000
Required footing area (ft^2):	146.65
Square Footing Size:	12.11

T.I.C.- Building "A"

Project Name: Rockville Metro II
Job Number: Tech 3
Date: 10/7/2013
Engineer: JMV

Column ID: U6.2

Level	Load Type	Height (ft)	Size (in x in)	Trib, A_T (ft^2)	Cum Trib (ft^2)	*K_{LL} (IBC)	Design LL (psf)		Allowable Reduction	Reduced LL (psf)	Slab t, (in)	P_{Slab} (kips)	P_{Drop} (kips)	P_{Beams} (kips)	P_{Facade} (kips)	P_{Col} (kips)	SDL (psf)	P_{SDL} (kips)	P_{LL} (kips)	P_{DL} (kips)	ΣP_{LL} (kips)	ΣP_{DL} (kips)	ΣP_{Total} (kips)	ΣP_u (kips)
PH Roof	Roof				0	4	35	(U)	0.00	35	8	0				0	15	0	0	0	0	0	0.0	0.0
Roof	Roof				0	4	35	(U)	0.00	35	8	0				0	50	0	0	0	0	0	0.0	0.0
Roof	Mech				0	4	150	(U)	0.00	150	8	0				0	100	0	0	0	0	0	0.0	0.0
11	Floor				0	4	100		0.00	100	8	0				0	30	0	0	0	0	0	0.0	0.0
10	Floor				0	4	100		0.00	100	8	0				0	10	0	0	0	0	0	0.0	0.0
9	Floor				0	4	100		0.00	100	8	0				0	10	0	0	0	0	0	0.0	0.0
8	Floor				0	4	100		0.00	100	8	0				0	10	0	0	0	0	0	0.0	0.0
7	Floor				0	4	100		0.00	100	8	0				0	10	0	0	0	0	0	0.0	0.0
6	Floor				0	4	100		0.00	100	8	0				0	10	0	0	0	0	0	0.0	0.0
5	Floor				0	4	100		0.00	100	8	0				0	10	0	0	0	0	0	0.0	0.0
4	Floor				0	4	100		0.00	100	8	0				0	10	0	0	0	0	0	0.0	0.0
P3	Garage	18	24 x 24	385	385	4	50		0.00	50	8	39	4			10.8	10	4	19.3	58	19.25	58	76.8	99.9
P1	Retail	9	24 x 24	385	770	4	100	(U)	0.00	100	9	43	4			5.4	10	4	38.5	57	57.75	115	172.3	229.8
BP1	Garage	9	24 x 24	385	1155	4	50		0.80	40	8	39	4			5.4	10	4	15.4	52	73.15	167	239.8	317.0
BP2	Garage	9	24 x 24	385	1540	4	50		0.80	40	8	39	4			5.4	10	4	15.4	52	88.55	219	307.4	404.3
BP3-SOG					1540														0	0	88.55	219	**307.4**	404.3

*K_{LL}, the live load element factor, was obtained from IBC Table 1607.9.1:

Interior columns	4
Exterior columns w/o cant slab	4
Edge columns w/ cant slab	3
Corner columns w/ cant slab	2

*LL reduction calculated using IBC eqn 16-1.
*LL reduction is **not** to be used:
1. For LL > 100 psf at the roof level. (For LL>100 for floors other than roof, the max allowable reduction is 20%.)
2. For LL <= 100 psf in public assembly areas.
3. For any roof loading.
4. For parking garages, the max allowable reduction is 20% for columns supporting **2** or more floors. Otherwise, it is 0%.

Foundation Design:

Allowable bearing (psf):	10000
Required footing area (ft^2):	30.74
Square Footing Size:	5.54

T.G.C.- Building "A"

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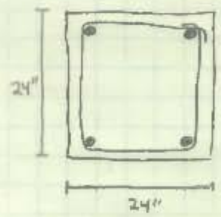
Column Analysis



24"x24" (4)#11, #4 ties @ 22" oc

$f'_c = 4000$ psi $f_y = 60$ ksi 1.5" CLR

Check at level 6 [applies for typ. interior & exterior]

$A_s = 4(1.56) = 6.24\ in^2$

Pure Axial (P_o)

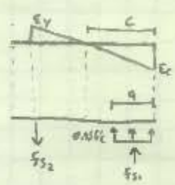
$\varepsilon_y = f_y/E_s = 0.00207$

$P_o = 0.85 f'_c (A_g - A_{st}) + A_s f_y = 0.85(4)(24\times24 - 6.24) + (60)(6.24) = 2311.6$ kips

Balanced Strain (M_b, P_b)

$c = \frac{0.003}{0.003 + \varepsilon_y} d_2 = \frac{0.003}{0.003 + 0.00207}(24 - 1.5 - 1/2 - 1.41/2) = 12.6"$

$a = \beta_1 c = 0.85(12.6) = 10.7"$



$\varepsilon_{s1} = \frac{0.003}{12.6}(12.6 - 2.705) = 0.0023 > \varepsilon_y \quad \therefore 60$ ksi

$\varepsilon_{s2} = \frac{0.003}{12.6}(12.6 - 23.47) = -0.0025 > \varepsilon_y \quad \therefore -60$ ksi

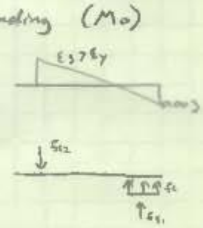
$P_b = 0.85(4)(0.85)(12.6)(24) + 2(1.56)(60) + 2(1.56)(-60)$

$P_b = 873.9$ kips

$M_b = 0.85(4)(0.85)(12.6)(24)\left[12 - \frac{10.7}{2}\right] + (3.12)(60)[12 - 2.7] + (3.12)(-60)[12 - 21.3]$

$M_b = 697.0$ kip-ft

Pure Bending (M_o)



$f_{s1} = \frac{0.003}{c}(c - 2.705)(29000)$ Assume f_{s1} does not yield and ε_{s2} does

$f_{s2} = -60$ ksi

$\Sigma F = 0: \ 0.85(4)(24)(0.85)c + 2(1.56)f_{s1} + 2(1.56)f_{s2}$

$\therefore c = 2.70"$

$f_{s1} = -3.86 < 60 \ \therefore ok$

$\varepsilon_{s2} = \varepsilon_{s2} < -0.00207 \ \therefore ok$

$M_o = 0.85(4)(24)(0.85\times2.70)\left(12 - \frac{0.85\times2.70}{2}\right) + (3.12)(-3.86)(12-2.7) + (3.12)(-60)(12-21.3)$

$M_o = 305.11$ kip-ft

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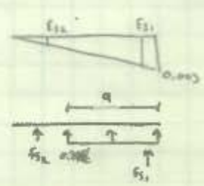
Pure Tension (T_o)

$T_o = \Sigma A_{si} f_{si}$ $f_{si} = -60$ ksi

$= 4(1.56)(-60) = -374.4$ kips

Point Between P_o & P_b

choose $c = h = 24''$



$\varepsilon_{s1} = \frac{0.003}{24}(24 - 2.7) = 0.00266 > \varepsilon_y \therefore f_{s1} = 60$ ksi

$\varepsilon_{s2} = \frac{0.003}{24}(24 - 21.7) = 0.0002875 < \varepsilon_y \therefore f_{s2} = 8.34$ ksi

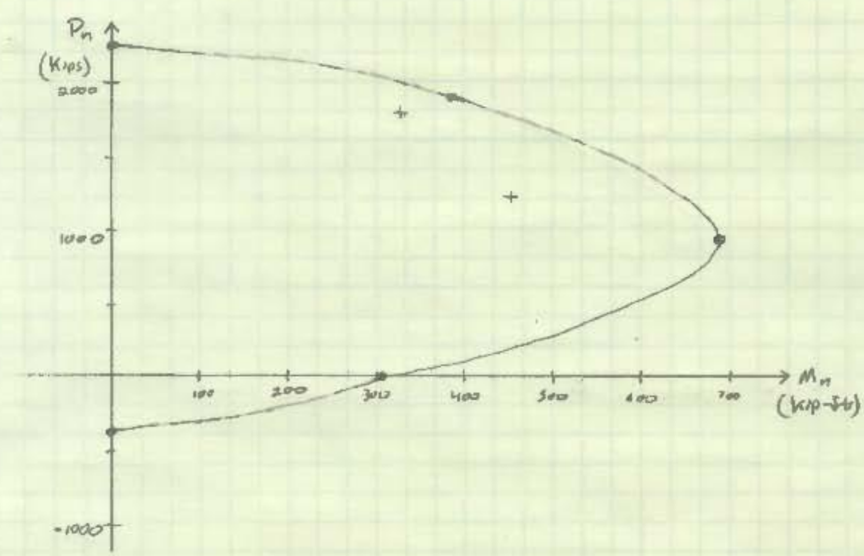
$P_n = 0.85(4)(24)(0.85)(24) + (3.12)(60) + (3.12)(8.34)$

$P_n = 1877.9$ kips

$M_n = 0.85(4)(24)(0.85)(24)\left[12 - \frac{0.85(24)}{2}\right] + (3.12)(60)(12 - 2.7) + (3.12)(8.34)(12 - 21.3)$

$M_n = 374.6$ kip-ft

Interaction Curve



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Column Capacity - Exterior Column (U-7)

Check for Max Combined Axial & Flexural under Gravity Loading

P_u = 832 kips [from Column takedown calculation]

M_u = 283.7 kip-ft [from PT beam calculation, let $M_u = 0.3 M_o$]

Using interaction curve: Let ϕ = 0.65 (compression controlled)

Let $P_u/\phi = P_n$ = 1280 kips

Let $M_u/\phi = M_n$ = 436.5 kip-ft

Point is within the curve ∴ ok

Column Capacity - Interior Column (S-3)

Check for Max Combined Axial & Flexure under Gravity Loading

P_u = 1176.2 kips [from column takedown calculation]

M_u ≈ 220.0 kip-ft [estimated from beam & slab calculations]

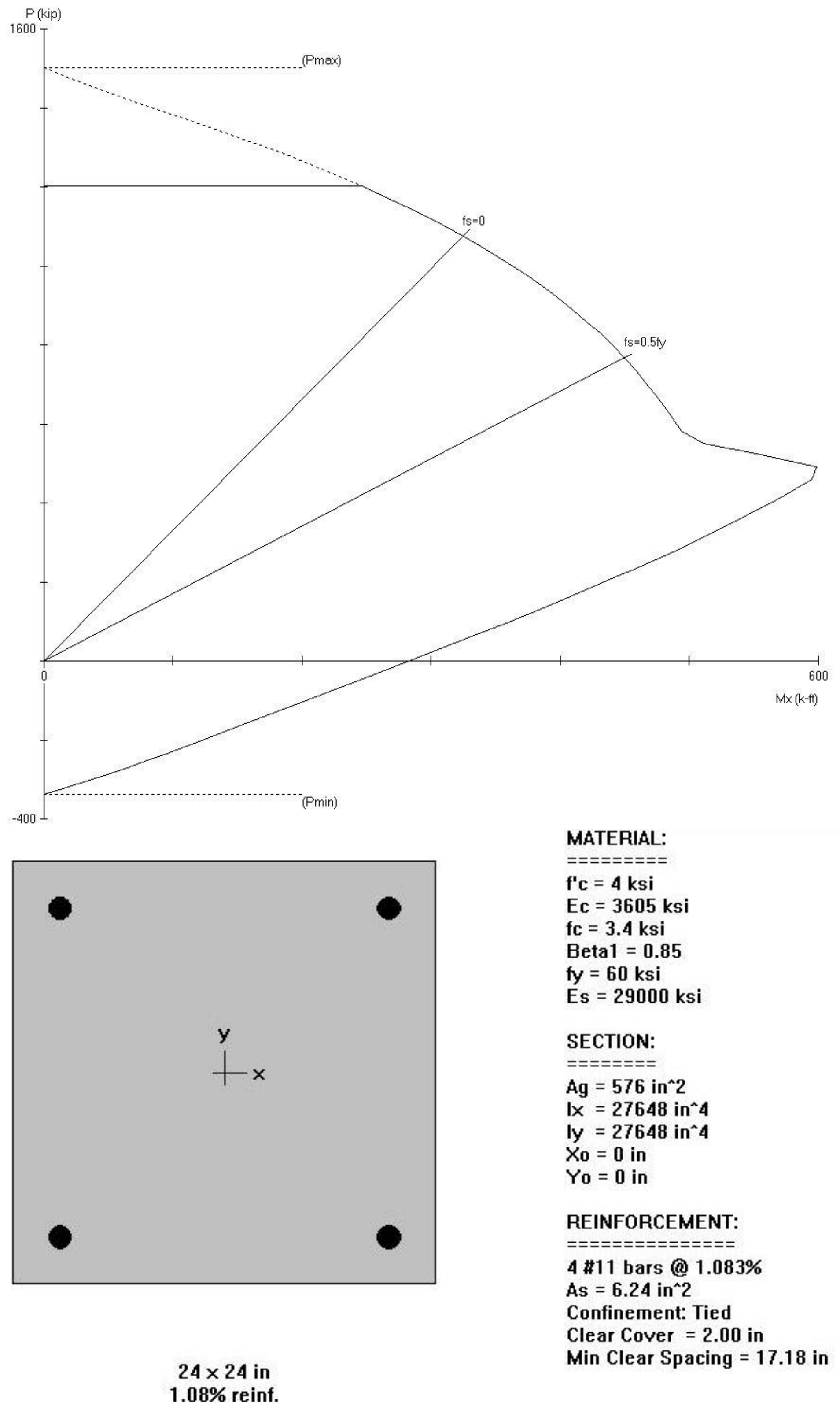
Using interaction curve: Let ϕ = 0.65 [compression controlled]

Let $P_u/\phi = P_n$ = 1809.5 kips

Let $M_u/\phi = M_n$ = 338.5 kip-ft

Point is within the curve ∴ ok

Column Interaction Diagram – Typical Exterior/Interior Column



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STRUCTUREPOINT - spColumn v4.81 (TM)                                              P
Licensed to: Penn State University. License ID: 59919-1033951-4-22545-2CF68       1
untitled.col                                                                      0


General Information:
====================
  File Name: untitled.col
  Project:
  Column:                                 Engineer:
  Code:     ACI 318-11                    Units: English

  Run Option: Investigation               Slenderness: Not considered
  Run Axis:   X-axis                      Column Type: Structural

Material Properties:
====================
  f'c   = 4 ksi                           fy   = 60 ksi
  Ec    = 3605 ksi                        Es   = 29000 ksi
  Ultimate strain = 0.003 in/in
  Beta1 = 0.85

Section:
========
  Rectangular: Width = 24 in              Depth = 24 in

  Gross section area, Ag =  576 in^2
  Ix =  27648 in^4                        Iy =  27648 in^4
  rx =  6.9282 in                         ry =  6.9282 in
  Xo =  0 in                              Yo =  0 in

Reinforcement:
==============
  Bar Set: ASTM A615
  Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)
  ---- --------- -----------   ---- --------- -----------   ---- --------- -----------
  #  3      0.38        0.11   #  4      0.50        0.20   #  5      0.63        0.31
  #  6      0.75        0.44   #  7      0.88        0.60   #  8      1.00        0.79
  #  9      1.13        1.00   # 10      1.27        1.27   # 11      1.41        1.56
  # 14      1.69        2.25   # 18      2.26        4.00

  Confinement: Tied; #3 ties with #10 bars,  #4 with larger bars.
  phi(a) = 0.8,  phi(b) = 0.9,  phi(c) = 0.65

  Layout: Rectangular
  Pattern: Sides Different  (Cover to longitudinal reinforcement)
  Total steel area: As = 6.24 in^2 at rho = 1.08%
  Minimum clear spacing = 17.18 in

                  Top       Bottom         Left        Right
                --------   --------     --------     --------
    Bars         2  #11     2  #11       0  # 3       0  # 3
    Cover(in)         2          2            2            2

Control Points:
===============
                        Axial Load P     X-Moment     Y-Moment NA depth Dt depth    eps_t   Phi
  Bending about                  kip         k-ft         k-ft       in       in
  --------------------- ------------ ------------ ------------ -------- -------- -------- ------
   X @ Max compression        1502.5         0.00         0.00    68.62    21.30 -0.00207  0.650
     @ Allowable comp.        1202.0       246.08         0.00    23.72    21.30 -0.00031  0.650
     @ fs = 0.0               1074.8       324.90         0.00    21.30    21.30  0.00000  0.650
     @ fs = 0.5*fy             767.8       449.57         0.00    15.83    21.30  0.00103  0.650
     @ Balanced point          561.3       497.74         0.00    12.60    21.30  0.00207  0.650
     @ Tension control         482.0       605.74         0.00     7.99    21.30  0.00500  0.900
     @ Pure bending             -0.0       282.88         0.00     2.70    21.30  0.02064  0.900
     @ Max tension            -337.0        -0.00        -0.00     0.00    21.30  9.99999  0.900

  -X @ Max compression        1502.5         0.00         0.00    68.62    21.30 -0.00207  0.650
     @ Allowable comp.        1202.0      -246.08        -0.00    23.72    21.30 -0.00031  0.650
     @ fs = 0.0               1074.8      -324.90        -0.00    21.30    21.30  0.00000  0.650
     @ fs = 0.5*fy             767.8      -449.57         0.00    15.83    21.30  0.00103  0.650
     @ Balanced point          561.3      -497.74         0.00    12.60    21.30  0.00207  0.650
     @ Tension control         482.0      -605.74         0.00     7.99    21.30  0.00500  0.900
     @ Pure bending             -0.0      -282.88        -0.00     2.70    21.30  0.02064  0.900
     @ Max tension            -337.0        -0.00        -0.00     0.00    21.30  9.99999  0.900


  *** End of output ***
```

J.M.V. TECH 3 SYS1

Foundation Analysis

Note:

- Consider Axial Gravity Load from Column only
- From Geotech Report - soil bearing capacity = 10 ksf

Typical Exterior Column (U-7)

ΣP_{tot} = 964.3 kips

Required footing area = ΣP_{tot}/Allowable = 964.3/10 ksf = 96.43 ft²

Area provided = 13.5' x 8' = 108 ft² > 96.4 ft² ∴ ok

Typical Interior Column (S-3)

ΣP_{tot} = 1466.5 kips

Required footing area = ΣP_{tot}/Allowable = 1466.5/10 ksf = 146.7 ft²

Area provided = 12.5' x 12.5' = 156.25 ft² > 146.7 ft² ∴ ok

Typical Garage Column (U-6.2)

ΣP_{tot} = 307.4 kips

Required footing area = ΣP_{tot}/Allowable = 307.4/10 ksf = 30.74 ft²

Area Provided = 5.5' x 5.5' = 30.25 ft² ~ 30.7 ∴ ok

[account for by overestimate of load]

- Summary

- Footing Spot checks show that the footings are designed within requirements for gravity loading. These should be checked lateral load cases as well for thorough analysis. Also, rectangular shape other than squares would have added considerations based on type of loading.

Alternative System One

Steel Frame & Non-Composite Concrete Deck

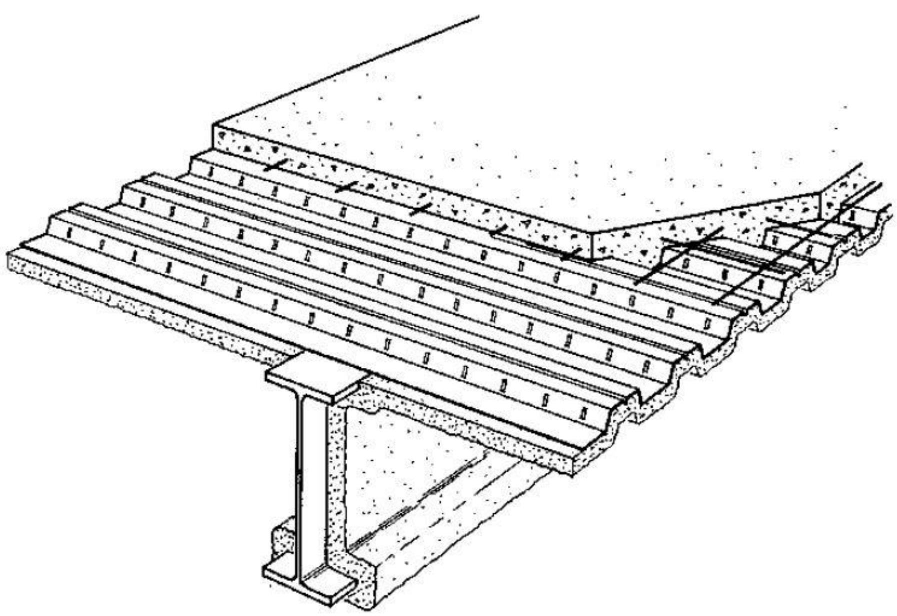


Figure 14: Steel Framing & Non Composite Concrete Deck
by RS Means

Steel Frame & Non-Composite Concrete Deck

One possible alternative floor system is a non-composite concrete deck supported by steel framing. In this scenario, the columns, beams, and girders of the structure would consist of various wide flange members. The metal decking would span the beams and girders and act as the formwork for the concrete that would be cast on top of the decking. For this system, light weigh concrete with a topping thickness of 3.25" was selected in order to achieve a 2 hour fire rating without having to fireproof the underside of the metal decking. This thinner topping also aids in minimizing the depth of this system.

This system offers many benefits to the overall design of the structure. One major benefit is that this system significantly reduces the overall weight of the building. This is accomplished though the implementation of a thinner floor slab and the use of economical steel members in place of heavy concrete elements. By reducing the overall weight of the structure, the earthquake design load on the structure is reduced as well as the size of footings in the foundation.

A steel system in place of concrete would also affect the lateral force resisting system of the structure. In place of a concrete shear wall, the logical choices of braced frames and moment frames constructed of steel members would need to be investigated.

The use of this system also has potential disadvantages. For example, the following calculations found that the minimum floor depth of a non-composite system would be 6.25" deeper than the original design. This would affect the architectural design of the interior spaces, as the floor to ceiling height would be reduced (in order to maintain the same story heights).

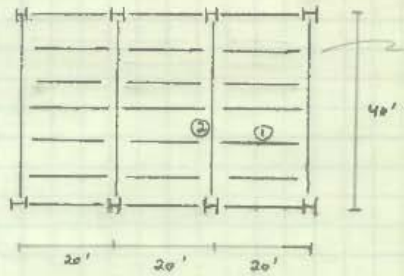
Another disadvantage of this system is that the lighter weight of the floor can give way to vibration. This serviceability aspect can make for an unpleasant space if not dealt with properly. Further investigation would need to be done in order to assess if the amount of vibration that this system would experience.

Also note that two permutation of this system will be calculated: beams running in the short direction and beams running in the long direction. Due to the similarities in cost and weight (as well as the latter possessing better constructability potential - few pieces to erect), the long span beam option will be used in the comparison section.

A more complete comparison of this system against the original system and the other alternatives may be found in the latter portion of this document.

J.M.V. | Tech 3 | Sys 1A

Alternative System 1 – Non-Composite Decking & Steel Framing



Six equal beam spacings

LL = 80 psf + 20 partitions
SDL = 5 psf
Beam sw ≈ 3 psf [Girder ≈ 5 psf]
Slab = 115 × 0.354 = 41 psf
Decking = 2 psf

$W_u = 1.2(41+2+3+5) + 1.6(100) = 221.2\ psf$

• Decking : use 2C20 , 5 1/4" slab , LWC [see Vulcraft Catalogue]

- assume 3 span condition & clear span 7' [since 7' > 40/6 = 6.67']

$F_b = 30000 \rightarrow 170\ psf > 148\ psf$ ∴ ok
$Def = \ell/240 \rightarrow 147\ psf > 100\ psf$ ∴ ok

Construction clear span max = 10'-9" > 6.67' ∴ ok

Note: slab thickness chosen for fire proofing reasons

• Typical Beams ①

Strength: $\frac{w_u \ell^2}{8} = M_u \rightarrow \frac{(221.2)(6.67)(20)^2}{8} = M_u = 73.7\ \text{kip ft}$

Serviceability: $\frac{5 w_{LL} L^4}{384 E I_x} = \Delta_{LL} = \ell/360 \rightarrow \frac{5(0.1 \times 6.67)(20)^4 (12)^3}{384(29000)(20 \times 12/360)} = I_x = 124.4\ in^4$

Choose W12×19 [$I_x = 130\ in^4$, $\phi M_n = 92.6$ kip-ft]
(deflection controls)

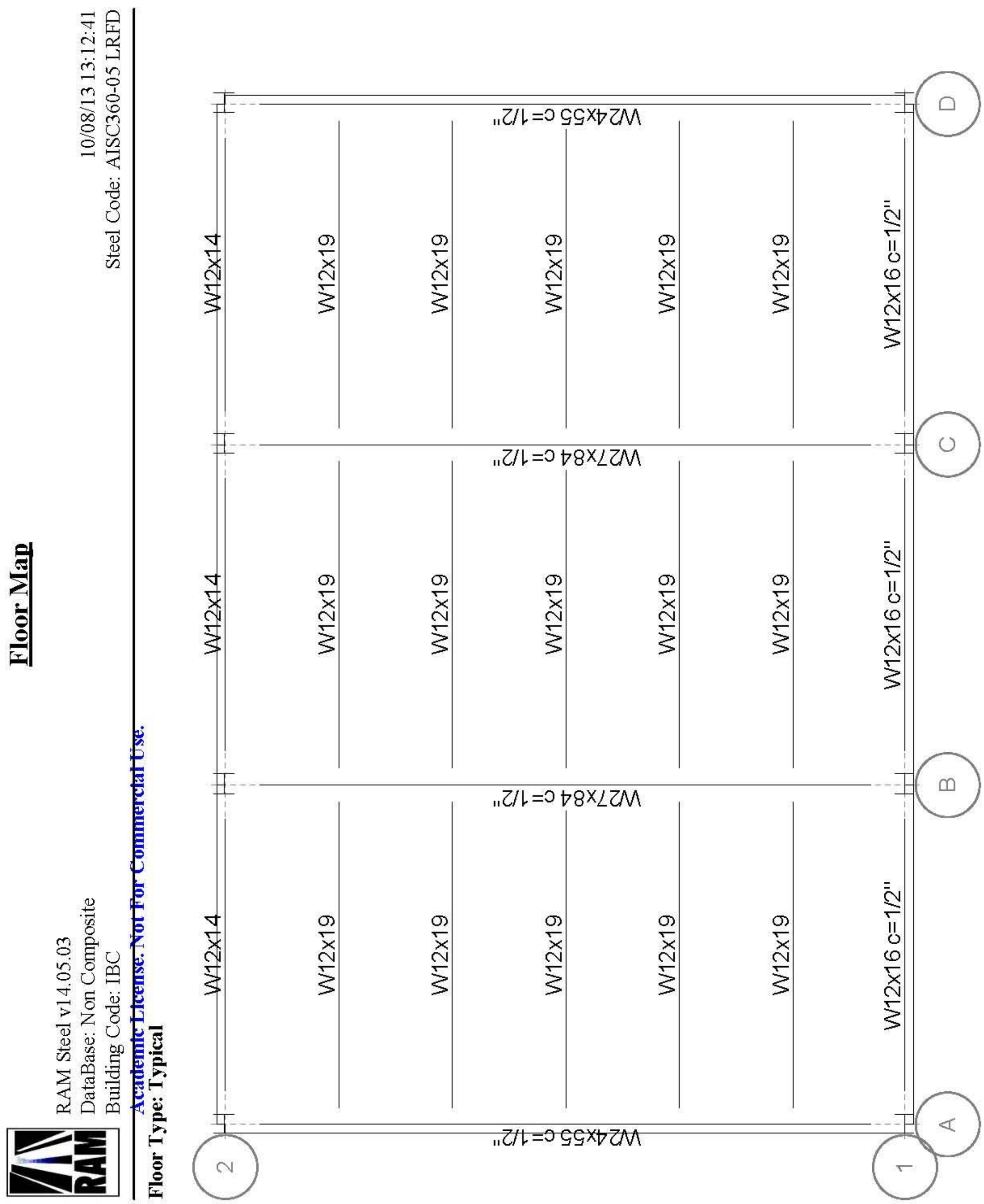
• Typical Girder ②

strength : $M_u = [1.2(56) + 1.6(70.0)](6.67)(20)(4.5 \times 6.67) = 717.5$ kip-ft

Serviceability : $\frac{5 w_{LL} L^4}{384 E I_x} = \Delta_{LL} = \ell/360 \rightarrow \frac{5(0.\ldots)(40)^4 (12)^3 (20)}{384(29000)(40 \times 12/360)} = I_x = 2085.6\ in^4$

Choose W24×84 [$I_x = 2370\ in^4$, $\phi M_n = 840$ kip-ft]

[Note: • partitions considered non reducible , $LLR = 0.25 + 15/\sqrt{1600}$
• Girder met W24×76 values (barely) but was sized up to account for loading type assumptions (point loads vs smear load)
• W24×84 chosen over W27×84 for depth consideration]


Floor Map
RAM Steel v14.05.03
DataBase: Non Composite
Building Code: IBC
10/08/13 13:12:41
Steel Code: AISC360-05 LRFD
Academic License. Not For Commercial Use.
Floor Type: Typical
W24x55 c=1/2"
W27x84 c=1/2"
W27x84 c=1/2"
W24x55 c=1/2"
W12x14
W12x19
W12x16 c=1/2"
A
B
C
D
1
2

Gravity Beam Design

RAM Steel v14.05.03.00
DataBase: Non Composite 10/09/13 15:33:55
Building Code: IBC Steel Code: AISC360-05 LRFD

Floor Type: Typical Beam Number = 27

SPAN INFORMATION (ft): I-End (20.00,20.00) J-End (40.00,20.00)

Beam Size (Optimum) = W12X19 Fy = 50.0 ksi
Total Beam Length (ft) = 20.00
Mp (kip-ft) = 102.92

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type	PartL
1	0.000	0.287	0.000	---	NonR	0.000
	20.000	0.287	0.000			0.000
2	0.000	0.033	0.533	0.0%	Red	0.133
	20.000	0.033	0.533			0.133
3	0.000	0.019	0.000	---	NonR	0.000
	20.000	0.019	0.000			0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 14.73 kips 1.00Vn = 86.01 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.2DL+1.6LL	73.7	10.0	0.0	1.00	0.90	92.62
Controlling		1.2DL+1.6LL	73.7	10.0	0.0	1.00	0.90	92.62

REACTIONS (kips):

	Left	**Right**
DL reaction	3.39	3.39
Max +LL reaction	6.67	6.67
Max +total reaction (factored)	14.73	14.73

DEFLECTIONS:

Dead load (in)	at	10.00 ft	=	-0.324	L/D	=	741
Live load (in)	at	10.00 ft	=	-0.637	L/D	=	377
Net Total load (in)	at	10.00 ft	=	-0.960	L/D	=	250

Gravity Beam Design

RAM Steel v14.05.03.00
DataBase: Non Composite 10/09/13 15:33:55
Building Code: IBC Steel Code: AISC360-05 LRFD

Floor Type: Typical **Beam Number = 2**

SPAN INFORMATION (ft): I-End (20.00,0.00) J-End (20.00,40.00)

Beam Size (Optimum) = W27X84 Fy = 50.0 ksi
Total Beam Length (ft) = 40.00
Mp (kip-ft) = 1016.6
7

POINT LOADS (kips):

Dist	DL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	PartL
6.667	3.39	5.33	33.9	0.00	0.00	0.0	0.00	0.0	1.33
6.667	3.39	5.33	33.9	0.00	0.00	0.0	0.00	0.0	1.33
13.333	3.39	5.33	33.9	0.00	0.00	0.0	0.00	0.0	1.33
13.333	3.39	5.33	33.9	0.00	0.00	0.0	0.00	0.0	1.33
20.000	3.39	5.33	33.9	0.00	0.00	0.0	0.00	0.0	1.33
20.000	3.39	5.33	33.9	0.00	0.00	0.0	0.00	0.0	1.33
26.667	3.39	5.33	33.9	0.00	0.00	0.0	0.00	0.0	1.33
26.667	3.39	5.33	33.9	0.00	0.00	0.0	0.00	0.0	1.33
33.333	3.39	5.33	33.9	0.00	0.00	0.0	0.00	0.0	1.33
33.333	3.39	5.33	33.9	0.00	0.00	0.0	0.00	0.0	1.33

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type	PartL
1	0.000	0.084	0.000	---	NonR	0.000
	40.000	0.084	0.000			0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 61.22 kips 1.00Vn = 368.46 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.2DL+1.6LL	730.6	20.0	6.7	1.05	0.90	915.00
Controlling		1.2DL+1.6LL	730.6	20.0	6.7	1.05	0.90	915.00

REACTIONS (kips):

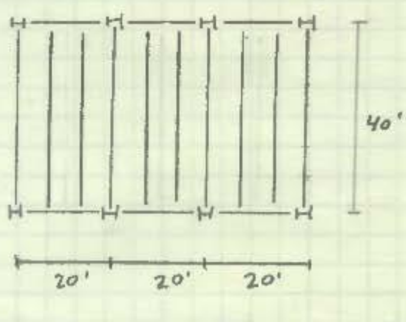
	Left	**Right**
DL reaction	18.63	18.63
Max +LL reaction	24.29	24.29
Max +total reaction (factored)	61.22	61.22

DEFLECTIONS: (Camber = 1/2)

Dead load (in)	at	20.00 ft	=	-0.752	L/D =	639
Live load (in)	at	20.00 ft	=	-0.993	L/D =	483
Net Total load (in)	at	20.00 ft	=	-1.245	L/D =	386

J.M.V. | TECH 3 | SYS 1.2A

Alternative System 1.2 - Non Composite Decking & Steel Framing



Equal Beam Spacings
facade load = 0.5 klf
LL = 80 psf + 20 psf (partitions)
SDL = 5 psf
Beam SW ≈ 7.5 psf [Girder ≈ 2 psf]
Slab = 115 × 0.354 = 41 psf
decking = 2 psf

$w_u = 1.2(41+2+7.5+5) + 1.6(100) = 226.6$ psf

• Decking

- use 2C20, 5 1/4" slab, LWC [see vulcraft catalogue]
- assume three span condition & that clear span = 7' [since 7' > 20/3 = 6.67']

$F_b = 30000 \rightarrow$ 170 psf >
$D_{LL} = l/240 \rightarrow$ 147 psf > 100 psf ∴ ok

construction clear span max = 10'9" > 6.67' ∴ ok

• Typical Beams

strength: $\frac{w_u l^2}{8} = M_u \rightarrow \frac{(0.2366)(6.67)(40)^2}{8} = M_u = 285.1$ kip·ft

serviceability: $\frac{5 w_{LL} L^4}{384 E I_x} = \Delta_{LL} = l/360 \rightarrow \frac{5(0.0922)(40)^4(12)^3(6.67)}{384(29000)(40 \times 12/360)} = I_x = 913.6 \text{ in}^4$

Beam LL reduction = $0.25 + \frac{15}{\sqrt{6.67 \times 40 \times 2}} = 89.94\%$

Choose W21x48 [$I_x = 959 \text{ in}^4$, $\phi M_n = 398$ kip·ft]

[Note: - Partitions considered non reducible
- results verified by following computer results]

- Exterior Girder, Typical

$$w_u = \left[1.2\,(41+2+7.5+5+2) + 1.6\,(20+80\times .7803)\right](20) + 1.2(500) = 4.62 \text{ klf}$$

Girder LL reduction $= 0.25 + \frac{15}{\sqrt{800}} = 78.03\%$

strength: $\frac{w_u L^2}{8} = M_u \rightarrow \frac{(4.62)(20)^2}{8} = 230.9$ Kip-ft

servicibility: $\frac{5 w_{LL} L^4}{384 E I_x} = \Delta_{LL} = L/360 \rightarrow \frac{5(0.082)(20)^4(12)^3(20)}{384(29000)(20\times 12/360)} = I_x = 307.0$ Kip-ft

Choose W 16x36 $[I_x = 448 \text{ in}^4,\ \phi M_n = 240 \text{ kip-ft}]$

- Interior Girder, Typical

$$W_u = \left[1.2\,(41+2+7.5+5+2) + 1.6\,(20+80\times 0.625)\right](40) = 7.24 \text{ klf}$$

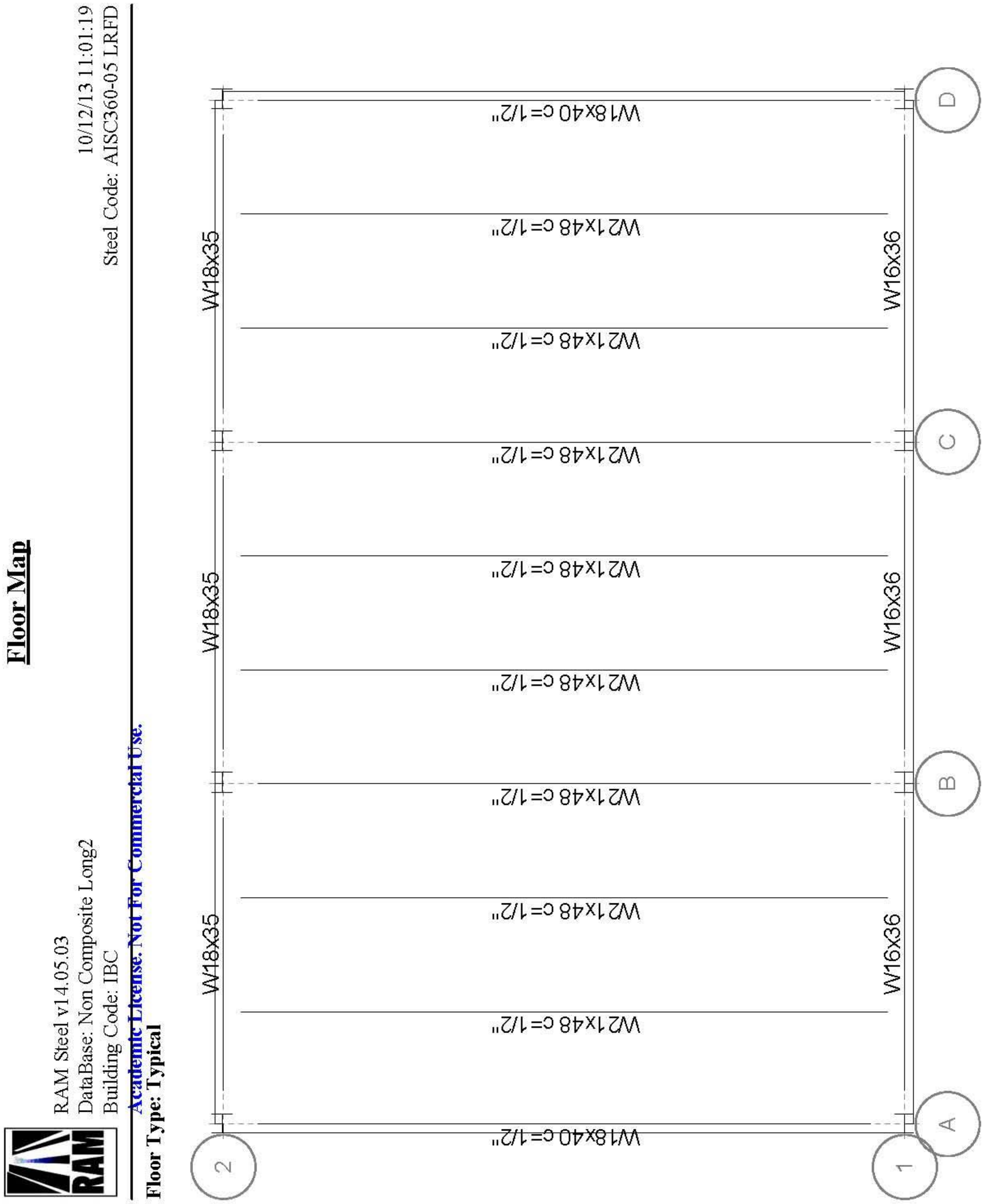
Girder LL reduction $= 0.25 + \frac{15}{\sqrt{1600}} = 0.625$

strength: $\frac{w_u L^2}{8} = M_u \rightarrow \frac{(7.24)(20)^2}{8} = M_u = 362$ Kip-ft

servicibility: $\frac{5 W_{LL} L^4}{384 E I_x} = \Delta_{LL} = L/360 \rightarrow \frac{5(0.07\times 40)(20)^4(12)^3}{384(29000)(20\times 12/360)} = I_x = 521.4 \text{ in}^4$

Choose W 21x48 $[I_x = 959 \text{ in}^4,\ \phi M_n = 398 \text{ kip-ft}]$

[Note: - while a W18x35 works for the typical exterior girder, the section chosen provides a shallower depth, despite economy.
- partitions considered non reducible
- results verified by following computer analysis]


Floor Map
RAM Steel v14.05.03
DataBase: Non Composite Long2
Building Code: IBC
Academic License. Not For Commercial Use.
Floor Type: Typical
10/12/13 11:01:19
Steel Code: AISC360-05 LRFD
W18x40 c=1/2"
W21x48 c=1/2"
W18x35
W16x36
A
B
C
D
1
2

Gravity Beam Design

RAM Steel v14.05.03.00
DataBase: Non Composite Long2 10/12/13 11:01:19
Building Code: IBC Steel Code: AISC360-05 LRFD

Floor Type: Typical **Beam Number = 37**

SPAN INFORMATION (ft): I-End (26.67,0.00) J-End (26.67,40.00)

Beam Size (Optimum) = W21X48 Fy = 50.0 ksi
Total Beam Length (ft) = 40.00
Mp (kip-ft) = 445.83

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type	PartL
1	0.000	0.287	0.000	---	NonR	0.000
	40.000	0.287	0.000			0.000
2	0.000	0.033	0.533	10.0%	Red	0.133
	40.000	0.033	0.533			0.133
3	0.000	0.048	0.000	---	NonR	0.000
	40.000	0.048	0.000			0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 28.45 kips 1.00Vn = 216.30 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.2DL+1.6LL	284.5	20.0	0.0	1.00	0.90	397.95
Controlling		1.2DL+1.6LL	284.5	20.0	0.0	1.00	0.90	397.95

REACTIONS (kips):

	Left	**Right**
DL reaction	7.36	7.36
Max +LL reaction	12.26	12.26
Max +total reaction (factored)	28.45	28.45

DEFLECTIONS: (Camber = 1/2)

Dead load (in)	at	20.00 ft	=	-0.762	L/D	=	630
Live load (in)	at	20.00 ft	=	-1.270	L/D	=	378
Net Total load (in)	at	20.00 ft	=	-1.532	L/D	=	313

Gravity Beam Design

RAM Steel v14.05.03.00
DataBase: Non Composite Long2 10/12/13 11:01:19
Building Code: IBC Steel Code: AISC360-05 LRFD

Floor Type: Typical **Beam Number = 6**

SPAN INFORMATION (ft): I-End (20.00,0.00) J-End (40.00,0.00)

Beam Size (Optimum) = W16X36 Fy = 50.0 ksi
Total Beam Length (ft) = 20.00
Mp (kip-ft) = 266.67

POINT LOADS (kips):

Dist	DL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	PartL
6.667	7.36	10.67	11.3	0.00	0.00	0.0	0.00	0.0	2.67
13.333	7.36	10.67	11.3	0.00	0.00	0.0	0.00	0.0	2.67

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type	PartL
1	0.000	0.500	0.000	0.0%	Red	0.000
	20.000	0.500	0.000			0.000
2	0.000	0.022	0.000	---	NonR	0.000
	20.000	0.022	0.000			0.000
3	0.000	0.003	0.041	11.3%	Red	0.010
	20.000	0.003	0.041			0.010
4	0.000	0.036	0.000	---	NonR	0.000
	20.000	0.036	0.000			0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 35.71 kips 1.00Vn = 140.71 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.2DL+1.6LL	225.6	10.0	6.7	1.00	0.90	228.45
Controlling		1.2DL+1.6LL	225.6	10.0	6.7	1.00	0.90	228.45

REACTIONS (kips):

	Left	**Right**
DL reaction	12.96	12.96
Max +LL reaction	12.60	12.60
Max +total reaction (factored)	35.71	35.71

DEFLECTIONS:

Dead load (in)	at	10.00 ft	=	-0.433	L/D =	554
Live load (in)	at	10.00 ft	=	-0.471	L/D =	510
Net Total load (in)	at	10.00 ft	=	-0.904	L/D =	265

Alternative System Two

Steel Frame & Composite Concrete Deck

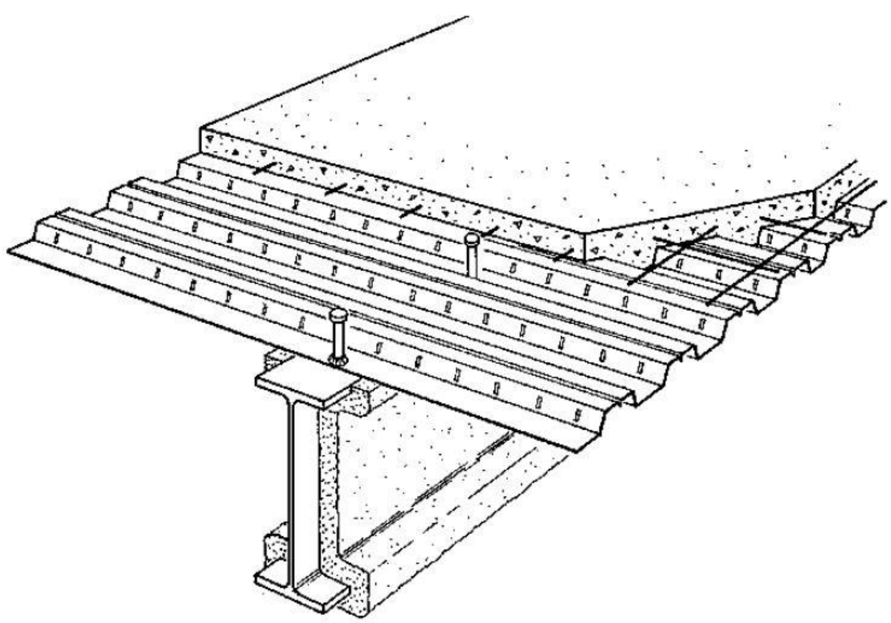


Figure 15: Steel Framing & Non Composite Concrete Deck
by RS Means

Steel Frame & Composite Concrete Deck

Another possible alternative floor system is a composite concrete deck supported by steel framing. In this scenario, the columns, beams, and girders of the structure would consist of various wide flange members. The metal decking would span the beams and girders and act as the formwork for the concrete that would be cast on top of the decking. Shear studs would be welded to the top flange of supporting members. These studs will act to transfer shear force from the beam into the slab. This will in turn increase the moment capacity of the section and allow for a smaller wide flange section (relative to the non-composite option). For this system, light weigh concrete with a topping thickness of 3.25" was selected in order to achieve a 2 hour fire rating without having to fireproof the underside of the metal decking. This thinner topping also aids in minimizing the depth of this system.

One major benefit of this system is that this it significantly reduces the overall weight of the building. This is accomplished though the implementation of a thinner floor slab and the use of economical steel members in place of heavy concrete elements. By reducing the overall weight of the structure, the earthquake design load on the structure is reduced as well as the size of footings in the foundation.

A steel system in place of concrete would also affect the lateral force resisting system of the structure. In place of a concrete shear wall, the logical choices of braced frames and moment frames constructed of steel members would need to be investigated.

The use of this system also has potential disadvantages. For example, the following calculations found that the minimum floor depth of a non-composite system would be 3.25" deeper than the original design. This would affect the architectural design of the interior spaces, as the floor to ceiling height would be reduced (in order to maintain the same story heights). Another disadvantage of this system is that the lighter weight of the floor can give way to vibration. This serviceability aspect can make for an unpleasant space if not dealt with properly. Further investigation would need to be done in order to assess if the amount of vibration that this system would experience.

Also note that two permutation of this system will be calculated: beams running in the short direction and beams running in the long direction. Due to the similarities in cost and weight (as well as the latter possessing better constructability potential - few pieces to erect), the long span beam option will be used in the comparison section. A more complete comparison of this system against the original system and the other alternatives may be found in the latter portion of this document.

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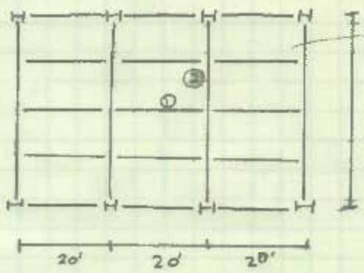
Alternative System 2 - Composite Decking & Steel Framing



Four Equal Beam Spacings

LL = 80 psf + 20 psf partitions
SDL = 5 psf
Beam SW ≈ 5 psf [Girder ≈ 7 psf]
Slab + Decking = 42 psf

• Decking 2VLI20 with 3.25" LWC ~~topping~~ [see Vulcraft catalogue]

-assume 3 span condition & clear span of 10' [since $10' = 40/4$]

Allowable super imposed deck load = 142 psf

142 > 105 psf ∴ ok

Max. Unshored clear span = 10'-11"

10'-11" < 10' ∴ ok

Note - thickness of deck/slab chosen for fireproofing reasons in addition to strength/service

• For Composite design, assume the following:

- 3/4" diameter shear studs
- fluted section will not contribute to strength
- consider partitions as non-reducible

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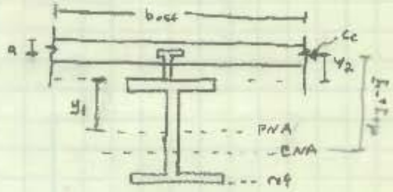
①



$w_u = 1.2(42+5+5) + 1.6(100) = 222.4$ psf

$b_{eff} = 2 \times \left[\min \begin{cases} 10/2 \times 12 \\ 20 \times 12/8 \end{cases} \right] \begin{matrix} = 120'' \\ = 60'' \leftarrow \end{matrix}$

$F_y = 50$ ksi $\quad f'_c = 3$ ksi

$M_u = \frac{w_u l^2}{8} = \frac{(222.4 \times 10)(20)^2}{8} = 111.2$ kip-ft

Max concrete force = 0.85(3 ksi)(60 in)(3.25 in) = 497 kips

Stud strength = 17.1 kips per stud [65 ksi, 3.5 in, 3/4" diam]

Consider Table 3-19: let a = 1.5" Y_2 = 4.5"

W10x12 → Capacity = 118 kips > 111.2 kips ∴ ok
ΣQ_n = 156 kips → 156/17.1 = 9.1 ∴ (10) studs per side

$a_{act} = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{156}{0.85(3)(60)} = 1.02 < 1.5$ ok

∴ W10x12 will work for composite action

- check unshored strength: $\phi M_p = 46.9$ kip-ft

$w_u = 1.4D = 1.4(42+5)(10) + 1.4(12) = 0.675$ klf

$w_u = 1.2D + 1.6L = 1.2(42+5)(10) + 1.2(12) + 1.6(20)(10) = 0.898$ klf

$M_u = \frac{w_u l^2}{8} = \frac{0.898(20)^2}{8} = 44.9$ klf < 46.9 ∴ ok

- check wet concrete deflection: $\Delta_{wc} = L/240 = 1''$

$W_{wc} = 42(10) + 26 = 0.446$ klf

$\Delta_{wc} = \frac{5wL^4}{384EI_x} = \frac{5(0.446)(20)^4(12)^3}{384(29000)(53.8)} \approx 1''$ ∴ ok [but consider camber]

- check LL deflection: $\Delta_{LL} = L/360 = 2/3''$

$W_{LL} = 100 \times 10 = 1$ klf

$I_{LB} = 201$ @ $Y_2 = 4.5''$ & $\Sigma Q_n = 156$ kips

$\Delta_{LL} = \frac{5wL^4}{384EI_x} = \frac{5(1.0)(20)^4(12)^3}{384(29000)(201)} = 0.62'' < 0.66''$ ∴ ok

- Summary: W10x12 (20)

[Note: computer result yeilded W10x12 (16)
variance attributed to approximating with tables]

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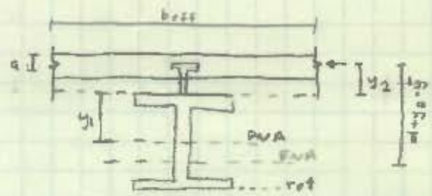
(2)



$w_u = 1.2(42+5+1.2+3) + 1.6(70.0) = 173.44$ psf

$b_{eff} = 2 \times \left[\min \begin{cases} 20/2 \times 12 \\ 40 \times 12/8 \end{cases} \right] \begin{matrix} = 240'' \\ = 120'' \leftarrow \end{matrix}$

$F_y = 50$ ksi $\quad f'_c = 3$ ksi

$M_u = \frac{w_u l^2}{8} = \frac{[173.44(20)](40)^2}{8} = 693.76$ kip-ft

Max concrete force = $0.85\,(3\text{ ksi})\,(120\text{ in})\,(7.25'') = 995$ kips

Stud strength = 17.1 Kip per stud $\quad$ [65 ksi, 3.5 in, 3/4" diam]

Consider Table 3-19 $\quad$ let $a = 1.5''$ $\quad Y_2 = 4.5$

W21x50 → capacity = 729 kip-ft > 693.76 kip-ft ∴ ok
$\Sigma Q_n = 473$ kips → $473/17.1 = 27.7$ ∴ (28) per side

$a_{act} = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{473}{0.85(3)(120)} \approx 1.5$ ∴ ok

- check unshored strength: $\quad \phi M_p = 413$ kip-ft

$w_u = 1.4D = 1.4(47+1.2)(20) + 1.4(50) = 1.42$ klf

$w_u = 1.2D + 1.6L = 1.2(47+1.2)(20) + 1.2(50) + 1.6(20)(20) = 1.857$ klf

$M_u = \frac{w_u l^2}{8} = \frac{1.857(40)^2}{8} = 371.36$ kip-ft < 413 ∴ ok

- check wet concrete deflection: $\quad \Delta_{wc} = l/240 = 2''$

$w_{wc} = 42(20) + 50 = 0.890$ klf

$\Delta_{wc} = \frac{5wL^4}{384 E I_x} = \frac{5(0.89)(40)^4(12)^3}{384\,(29000)\,(\ldots)} = 1.80'' < 2''$ ∴ ok
[but consider camber]

- check LL deflection: $\quad \Delta_{LL} = l/360 = 1\frac{1}{3}''$

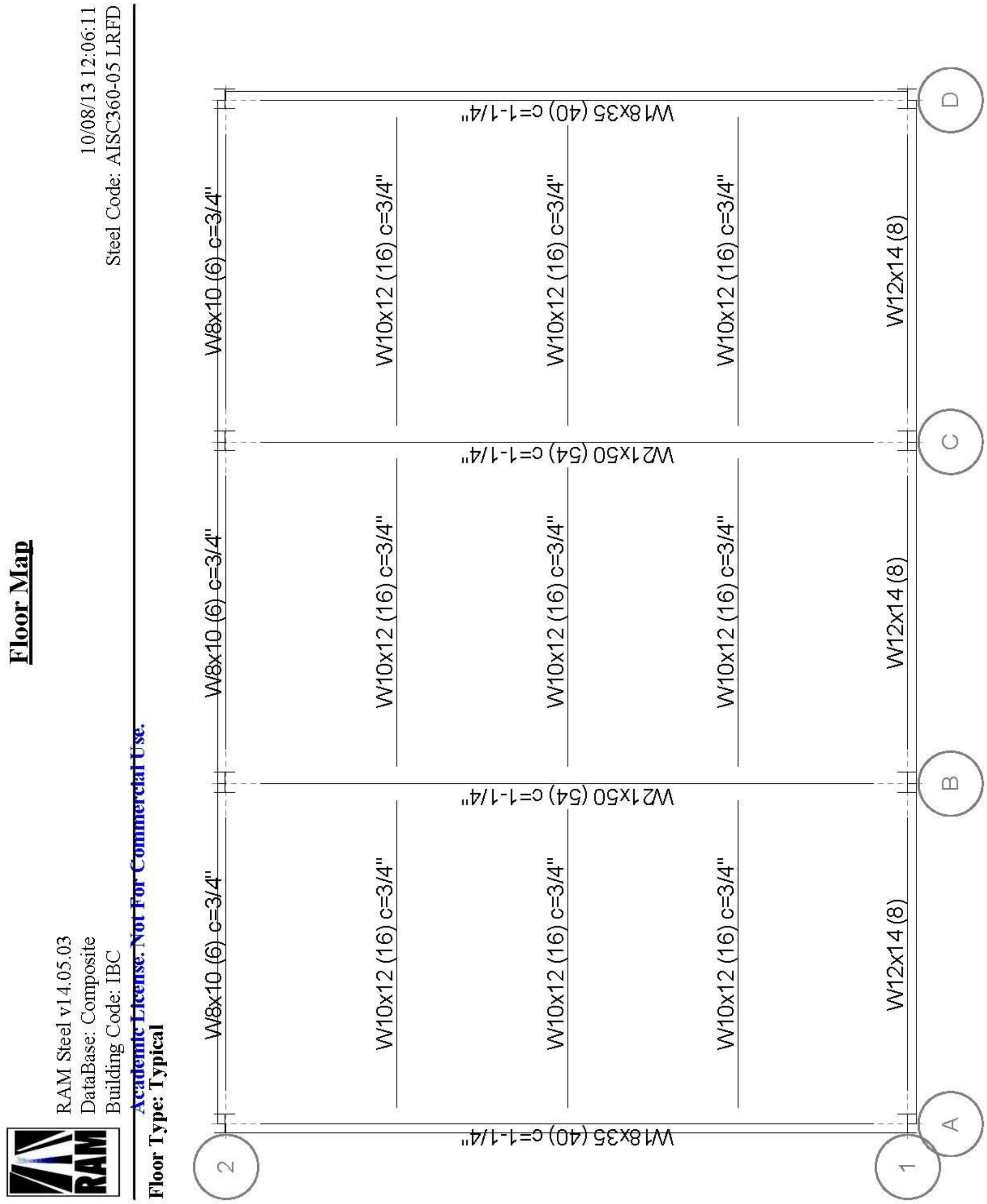
$w_{LL} = 70 \times 20 = 1.4$ klf

$I_{LB} = 2260$ @ $Y_2 = 4.5''$ & $\Sigma Q_n = 473$ kips

$\Delta_{LL} = \frac{5wL^4}{384 E I_{LB}} = \frac{5(1.4)(40)^4(12)^3}{384\,(29000)(2260)} = 1.23'' < 1\frac{1}{3}''$ ∴ ok
May Apply camber to offset

- Summary: W21x50 (56) [+ xx camber]

[Note: computer result yielded W21x50 (54)
variance attributed to hand calc's over
approximating with tables in manual]


Floor Map
RAM Steel v14.05.03
DataBase: Composite
Building Code: IBC
Academic License. Not For Commercial Use.
Floor Type: Typical
10/08/13 12:06:11
Steel Code: AISC360-05 LRFD
W18x35 (40) c=1-1/4"
W21x50 (54) c=1-1/4"
W21x50 (54) c=1-1/4"
W18x35 (40) c=1-1/4"
W8x10 (6) c=3/4"
W8x10 (6) c=3/4"
W8x10 (6) c=3/4"
W10x12 (16) c=3/4"
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W12x14 (8)
W12x14 (8)
W12x14 (8)
A
B
C
D
1
2

Gravity Beam Design

RAM Steel v14.05.03.00
DataBase: Composite 10/09/13 18:41:59
Building Code: IBC Steel Code: AISC360-05 LRFD

Floor Type: Typical **Beam Number = 38**

SPAN INFORMATION (ft): I-End (20.00,20.00) J-End (40.00,20.00)

Beam Size (Optimum) = W10X12 Fy = 50.0 ksi
Total Beam Length (ft) = 20.00

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Deck Label	Office	Office
Concrete thickness (in)	3.25	3.25
Unit weight concrete (pcf)	110.00	110.00
f'c (ksi)	3.00	3.00
Decking Orientation	perpendicular	perpendicular
Decking type	VULCRAFT 2.0VL	VULCRAFT 2.0VL

beff (in)	=	60.00	Y bar(in)	=	11.77
Mnf (kip-ft)	=	141.70	Mn (kip-ft)	=	127.39
C (kips)	=	136.82	PNA (in)	=	9.77
Ieff (in4)	=	246.74	Itr (in4)	=	273.24
Stud length (in)	=	3.50	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 17.1 Rg = 1.00 Rp = 0.60
\# of studs: Max = 20 Partial = 16 Actual = 16
Number of Stud Rows = 1 Percent of Full Composite Action = 77.30

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.420	0.420	0.000	---	NonR	0.000	0.000
	20.000	0.420	0.420	0.000			0.000	0.000
2	0.000	0.050	0.000	0.800	0.0%	Red	0.200	0.000
	20.000	0.050	0.000	0.800			0.200	0.000
3	0.000	0.012	0.012	0.000	---	NonR	0.000	0.000
	20.000	0.012	0.012	0.000			0.000	0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 21.78 kips 1.00Vn = 56.26 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.4DL	30.2	10.0	0.0	1.00	0.90	46.90
	Init DL	1.4DL	30.2	10.0	---	---		
	Max +	1.2DL+1.6LL	108.9	10.0	---	---	0.90	114.65
Controlling		1.2DL+1.6LL	108.9	10.0	---	---	0.90	114.65

REACTIONS (kips):

	Left	Right
Initial reaction	4.32	4.32
DL reaction	4.82	4.82
Max +LL reaction	10.00	10.00
Max +total reaction (factored)	21.78	21.78

DEFLECTIONS: (Camber = 3/4)

Initial load (in)	at	10.00 ft	=	-0.996	L/D	=	241
Live load (in)	at	10.00 ft	=	-0.503	L/D	=	477
Post Comp load (in)	at	10.00 ft	=	-0.528	L/D	=	454
Net Total load (in)	at	10.00 ft	=	-0.774	L/D	=	310

Gravity Beam Design

RAM Steel v14.05.03.00
DataBase: Composite 10/09/13 18:41:59
Building Code: IBC Steel Code: AISC360-05 LRFD

Floor Type: Typical **Beam Number = 3**

SPAN INFORMATION (ft): I-End (40.00,0.00) J-End (40.00,40.00)

Beam Size (Optimum) = W21X50 Fy = 50.0 ksi
Total Beam Length (ft) = 40.00

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Deck Label	Office	Office
Concrete thickness (in)	3.25	3.25
Unit weight concrete (pcf)	110.00	110.00
f'c (ksi)	3.00	3.00
Decking Orientation	parallel	parallel
Decking type	VULCRAFT 2.0VL	VULCRAFT 2.0VL

beff (in)	=	120.00	Y bar(in)	=	19.60
Mnf (kip-ft)	=	885.00	Mn (kip-ft)	=	805.37
C (kips)	=	461.78	PNA (in)	=	20.38
Ieff (in4)	=	2507.25	Itr (in4)	=	2905.76
Stud length (in)	=	3.50	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 17.1 Rg = 1.00 Rp = 0.75
\# of studs: Full = 86 Partial = 54 Actual = 54
Number of Stud Rows = 1 Percent of Full Composite Action = 62.83

POINT LOADS (kips):

Dist	DL	CDL	RedLL	Red%	NonRL L	StorLL	Red%	RoofLL	Red%	PartL	
10.000	4.82	4.32	8.00	31.7	0.00	0.00	0.0	0.00	0.0	2.00	0.00
10.000	4.82	4.32	8.00	31.7	0.00	0.00	0.0	0.00	0.0	2.00	0.00
20.000	4.82	4.32	8.00	31.7	0.00	0.00	0.0	0.00	0.0	2.00	0.00
20.000	4.82	4.32	8.00	31.7	0.00	0.00	0.0	0.00	0.0	2.00	0.00
30.000	4.82	4.32	8.00	31.7	0.00	0.00	0.0	0.00	0.0	2.00	0.00
30.000	4.82	4.32	8.00	31.7	0.00	0.00	0.0	0.00	0.0	2.00	0.00

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.050	0.050	0.000	---	NonR	0.000	0.000
	40.000	0.050	0.050	0.000			0.000	0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 54.36 kips 1.00Vn = 237.12 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.4DL	255.7	20.0	10.0	1.11	0.90	347.30
	Init DL	1.4DL	255.7	20.0	---	---		
	Max +	1.2DL+1.6LL	720.9	20.0	---	---	0.90	724.83
Controlling		1.2DL+1.6LL	720.9	20.0	---	---	0.90	724.83

REACTIONS (kips):

Gravity Beam Design

RAM Steel v14.05.03.00
DataBase: Composite 10/09/13 18:41:59
Building Code: IBC Steel Code: AISC360-05 LRFD

Floor Type: Typical **Beam Number = 3**

SPAN INFORMATION (ft): I-End (40.00,0.00) J-End (40.00,40.00)

Beam Size (Optimum) = W21X50 Fy = 50.0 ksi
Total Beam Length (ft) = 40.00

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Deck Label	Office	Office
Concrete thickness (in)	3.25	3.25
Unit weight concrete (pcf)	110.00	110.00
f'c (ksi)	3.00	3.00
Decking Orientation	parallel	parallel
Decking type	VULCRAFT 2.0VL	VULCRAFT 2.0VL

beff (in)	=	120.00	Y bar(in)	=	19.60
Mnf (kip-ft)	=	885.00	Mn (kip-ft)	=	805.37
C (kips)	=	461.78	PNA (in)	=	20.38
Ieff (in4)	=	2507.25	Itr (in4)	=	2905.76
Stud length (in)	=	3.50	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 17.1 Rg = 1.00 Rp = 0.75
\# of studs: Full = 86 Partial = 54 Actual = 54
Number of Stud Rows = 1 Percent of Full Composite Action = 62.83

POINT LOADS (kips):

Dist	DL	CDL	RedLL	Red%	NonRL L	StorLL	Red%	RoofLL	Red%	PartL	
10.000	4.82	4.32	8.00	31.7	0.00	0.00	0.0	0.00	0.0	2.00	0.00
10.000	4.82	4.32	8.00	31.7	0.00	0.00	0.0	0.00	0.0	2.00	0.00
20.000	4.82	4.32	8.00	31.7	0.00	0.00	0.0	0.00	0.0	2.00	0.00
20.000	4.82	4.32	8.00	31.7	0.00	0.00	0.0	0.00	0.0	2.00	0.00
30.000	4.82	4.32	8.00	31.7	0.00	0.00	0.0	0.00	0.0	2.00	0.00
30.000	4.82	4.32	8.00	31.7	0.00	0.00	0.0	0.00	0.0	2.00	0.00

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.050	0.050	0.000	---	NonR	0.000	0.000
	40.000	0.050	0.050	0.000			0.000	0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 54.36 kips 1.00Vn = 237.12 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.4DL	255.7	20.0	10.0	1.11	0.90	347.30
	Init DL	1.4DL	255.7	20.0	---	---		
	Max +	1.2DL+1.6LL	720.9	20.0	---	---	0.90	724.83
Controlling		1.2DL+1.6LL	720.9	20.0	---	---	0.90	724.83

REACTIONS (kips):

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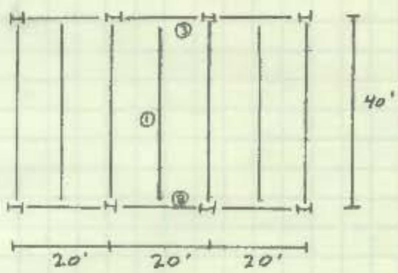
	Left	Right
Initial reaction	13.95	13.95
DL reaction	15.45	15.45
Max +LL reaction	22.39	22.39
Max +total reaction (factored)	54.36	54.36

DEFLECTIONS: (Camber = 1-1/4)

Initial load (in)	at	20.00 ft	=	-1.756	L/D	=	273
Live load (in)	at	20.00 ft	=	-1.123	L/D	=	427
Post Comp load (in)	at	20.00 ft	=	-1.199	L/D	=	400
Net Total load (in)	at	20.00 ft	=	-1.705	L/D	=	282

J.M.V. | TECH 3 | SYS 2.2A

Alternative System 2.2A - Composite Decking & Steel Framing



Equal beam spacing

Facade = 500 plf
LL = 80 psf + 20 psf (partitions)
SDL = 5 psf
Beam SW ≈ 5 psf [Girder =
Slab + decking = 42 psf

• Decking 2VLI 20 with 3.25" LWC topping [see vulcraft catalogue]

- assume 3 span condition & clear span of 10' [since 10' = 20'/2]

- Allowable superimposed deck load = 142 psf > 105 psf ∴ ok

- Max unshored clear span = 10'-11" < 10' ∴ ok

Note - thickness of deck chosen for Fireproofing & depth reasons in addition to strength / serviceability

- For Composite design, assume the following:

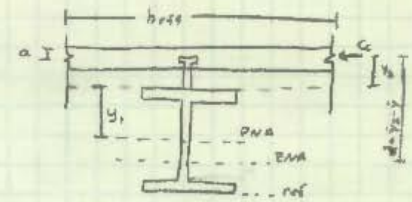
- 3/4" diameter shear studs

- Fluted section will not contribute to strength

- Consider partitions as non-reducible

J.M.V. TECH 3 SYS 2.2A

①



$w_u = [1.2(42+3+5) + 1.6(20+50)]10 = 1.744$ klf

LL reduction $= 0.25 + \frac{15}{\sqrt{1600}} = 62.5\%$

$b_{eff} = 2 \times \left[\min \begin{Bmatrix} 20/2 \times 12 \\ 40 \times 12/8 \end{Bmatrix} \right] \begin{matrix} = 240'' \\ = 120'' \leftarrow \end{matrix}$

$F_y = 50$ ksi

$F'_c = 3$ ksi

$M_u = \frac{w_u l^2}{8} = \frac{1.744(40)^2}{8} = 348.8$ Kip-ft

Max Concrete Force $= 0.85(3 \text{ ksi})(120 \text{ in})(3.25'') = 995$ kips

Stud strength $= 17.1$ kip per stud [65 ksi, 3.5", 3/4" diam]

Consider table 3-19 let $a = 1.5''$ $Y_2 = 4.5''$

W18x35 → capacity $= 397$ kip-ft > 348.8 kip-ft ∴ ok

$\Sigma Q_n = 194$ kips → $194/17.1 = 11.3$ ∴ (12) studs per side

$a_{act} = \frac{\Sigma Q_n}{0.85 f'c\, b_{eff}} = \frac{194}{0.85(3)(120)} = 0.64'' < 1.5''$ ∴ ok

- check unshored strength : $\phi M_p = 249$ kip-ft

$w_u = 1.4D = 1.4(42+5)(10) + 1.4(31) = 0.701$ klf

$w_u = 1.2D + 1.6L = 1.2(47)(10) + 1.2(31) + 1.6(20)(10) = 0.921$ klf

$M_u = \frac{w_u l^2}{8} = \frac{(0.921)(40)^2}{8} = 184.2$ Kip-ft < 249 kip-ft ∴ ok

- check wet concrete deflection : $\Delta_{wc} = L/240 = 2''$

$w_{wc} = 42(10) + 35 = 455$ plf

$\Delta_{wc} = \frac{5wL^4}{384EI_x} = \frac{5(0.455)(40)^4(12)^3}{384(29000)(510)} = 1.77'' < 2''$ ∴ ok

- check LL deflection : $\Delta_{LL} = L/360 = 1\frac{1}{3}''$

$W_{LL} = 70 \times 10 = 0.7$ klf

$I_{LB} = 1282 \text{ in}^4$ @ $a = 0.64''$

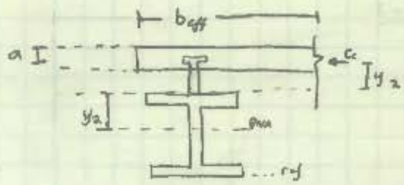
$\Delta_{LL} = \frac{5wL^4}{384EI_{LB}} = \frac{5(0.7)(40)^4(12)^3}{384(29000)(1282)} = 1.1'' < 1.33''$ ∴ ok

Summary : W18x35 (24)

[Note : computer result yeilded W18x35 (22)
- variance attributed to approximating with tables]

J.M.V. TECH 3 SYS 2.2A

②



$P_u = [1.2(42+3.5+5+1.5)(10) + 1.2(35) + 1.6(20+62.4)(10)](20)$
$= 39.696$ kip

LL reduction $= 0.25 + 15/\sqrt{800} = 78.03\%$

$w_u = 500$ plf

$b_{eff} = \min \begin{cases} 40/2 \times 12 + 6 = 246'' \\ 20 \times 12/8 + 6 = 36'' \leftarrow \end{cases}$

Assume 6" overhang

$M_u = \frac{PL}{4} + \frac{wL^2}{8} = \frac{(39.696)(20)}{4} + \frac{(0.5)(20)^2}{8} = 224.8$ kip-ft

Max concrete force $= 0.85(3\text{ ksi})(36\text{ in})(3.25\text{ in}) = 298$ kips

Stud strength $= 17.1$ kips per stud [65 ksi, 3.5 in, 3/4" diam]

Consider table 3-19 let $a = 1.5''$ $Y_2 = 4.5''$

W 16x26 → capacity $= 263.5$ (interpolated) > 224.8 kip-ft ∴ ok
$\Sigma Q_n = 136.8$ → choose (8) studs ∴ $\Sigma Q_n = 136.8$

$a_{act} = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{136.8}{0.85(3)(36)} = 1.49'' < 1.5''$ ∴ ok

- check unshored strength : $\phi M_p = 166$ kip-ft

$P_u = 1.4D = [1.4(42+5+3.5+1.5)(20)(10) + 1.4(35)(20)] = 15.4$ kips

$P_u = 1.2D + 1.6L = [1.2(47+5)(20)(10) + 1.2(35)(20) + 1.6(20)(10)(20)] = 19.72$ kips ←

$M_u = \frac{w_u l^2}{8} = \frac{(19.72)(20)}{4} = 98.6$ kip-ft < 166 kip-ft ∴ ok

- check wet concrete deflection : $\Delta_{wc} = L/240 = 1''$

$w_{wc} = 26$ plf $P_{wc} = 35(20) + 42(10)(20) = 9.1$ kips

$\Delta_{wc} = \frac{PL^3}{48EI} + \frac{5wL^4}{384EI_x} = \frac{5(0.026)(20)^4(12)^3}{384(29000)(301)} + \frac{(9.1)(20)^3(12)^3}{48(29000)(301)} = 0.31'' < 1''$ ∴ ok

- check LL deflection : $\Delta_{LL} = L/360 = 2/3''$

$P_{LL} = (82.4)(20)(10) = 16.5$ kips $I_{LB} = 607$ by interpolation

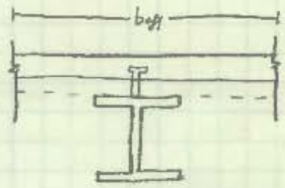
$\Delta_{LL} = \frac{PL^3}{48EI} = \frac{(20+62.4)(10\times 20)(20)^3(12)^3}{48(29000)(607)} = 0.27'' < 0.67''$ ∴ ok

- Summary : W 16x26 (16)

[Note : computer results yielded W16x26 (14)
– variance attributed to approx. with tables]

J. M. V. | TECH 3 | SYS 2.2A

(3)



$P_u = [1.2(42+5+8)(10) + 1.2(35) + 1.6(20+50)(10)](40)$
$= 71.44$ kips

LL reductn $= 0.25 + 15/\sqrt{1600} = 62.5\%$

$b_{eff} = 2 \times \left[\min \begin{Bmatrix} 40/2 \times 12 \\ 20 \times 12/8 \end{Bmatrix} \right] \begin{matrix} = 480'' \\ = 60'' \leftarrow \end{matrix}$

$M_u = \frac{PL}{4} = \frac{71.44(20)}{4} = 357.2$ kip-ft

Max concrete force $= 0.85(3\,ksi)(60\,in)(3.25\,in) = 497$ kips

stud strength = 17.1 kips per stud [65 ksi, 3.5 in, 3/4" diam]

Consider table 3-19 let $a = 1.5''$ $Y_2 = 4.5''$

W 18x35 → capacity = 397 kip-ft > 357.2 kip-ft ∴ ok
$\Sigma Q_n = 194$ kips → $194/17.1 = 11.3$ ∴ choose (12) studs per side

$a_{act} = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{194}{0.85(3)(60)} = 1.27'' < 1.5''$ ∴ ok

- check unshored strength : $\phi M_p = 249$ kip-ft

$P_u = 1.4D = [1.4(42+5+3.5+1.5)(40) + 1.4(35)](40) = 30.8$ kips

$P_u = 1.2L + 1.6D = [1.2(47+5)(10)(40) + 1.2(35)(40) + 1.6(20)(10)(40) = 39.44$ kips ←

$M_u = \frac{PL}{4} = \frac{39.44(20)}{4} = 197.2$ kip-ft < 249 kip-ft ∴ ok

- check wet concrete deflection : $\Delta_{wc} = L/240 = 1''$

$w_{wc} = 35$ plf $P_{wc} = 35(40) + 42(10)(40) = 18.2$ kips

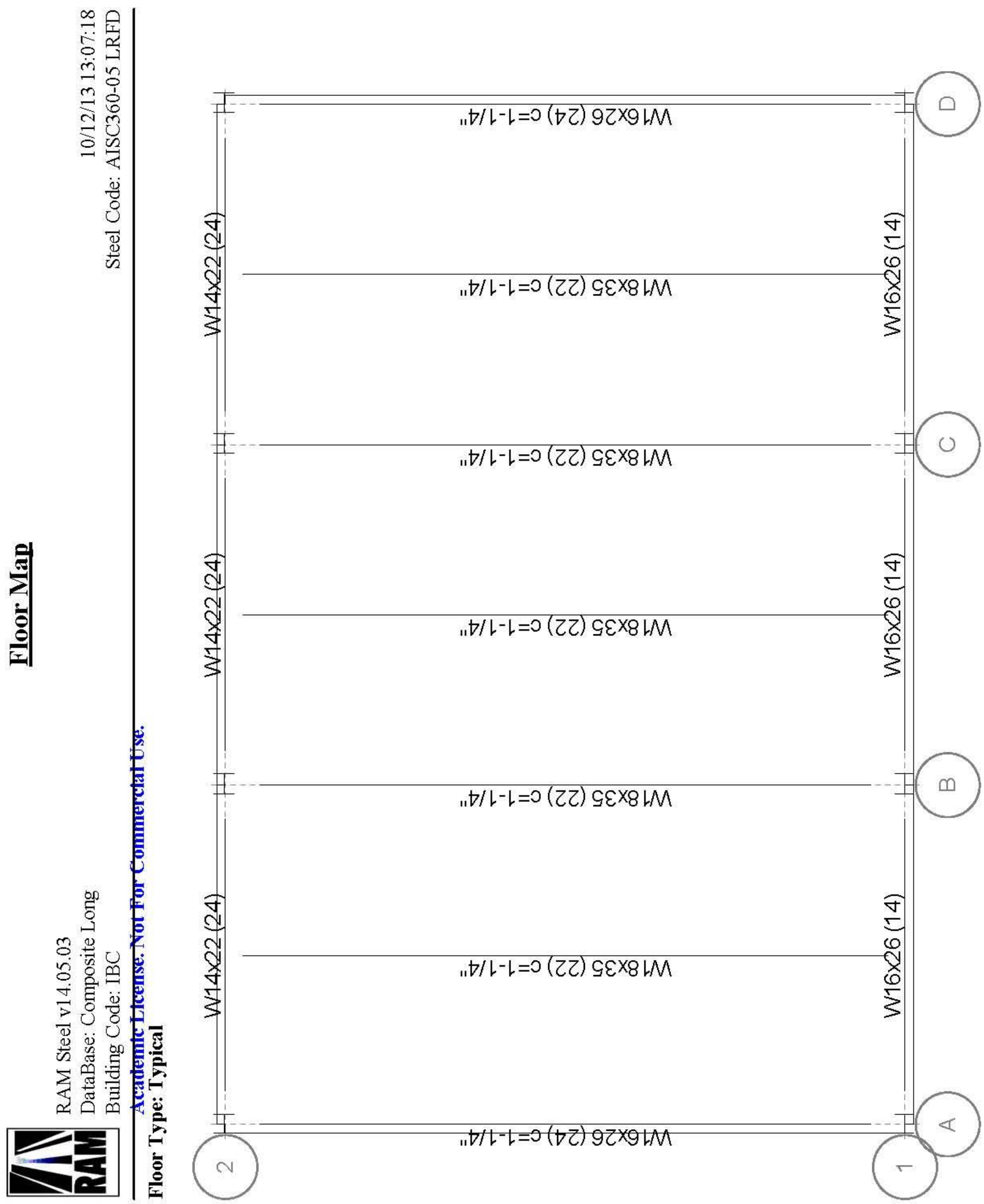
$\Delta_{wc} = \frac{PL^3}{48EI_x} + \frac{5wL^4}{384EI_x} = \frac{(18.2)(20)^3(12)^3}{48(29000)(510)} + \frac{5(0.035)(20)^4(12)^3}{384(29000)(510)} = 0.36'' < 1''$ ∴ ok

- check LL deflection : $\Delta_{LL} = L/360 = 2/3''$

$W_{LL} = (70.0)(40)(10) = 28.0$ kips $I_{LB} = 1010\ in^4$

$\Delta_{LL} = \frac{PL^3}{48EI} = \frac{[(20+50)(40)(10)](20)^3(12)^3}{48(29000)(1010)} = 0.28'' < 0.66''$ ∴ ok

- Summary : W18x35 (24)


Floor Map
RAM Steel v14.05.03
DataBase: Composite Long
Building Code: IBC
Academic License. Not For Commercial Use.
Floor Type: Typical
10/12/13 13:07:18
Steel Code: AISC360-05 LRFD
W16x26 (24) c=1-1/4"
W18x35 (22) c=1-1/4"
W18x35 (22) c=1-1/4"
W18x35 (22) c=1-1/4"
W18x35 (22) c=1-1/4"
W18x35 (22) c=1-1/4"
W16x26 (24) c=1-1/4"
W14x22 (24)
W14x22 (24)
W14x22 (24)
W16x26 (14)
W16x26 (14)
W16x26 (14)
A
B
C
D
1
2

Gravity Beam Design

RAM Steel v14.05.03.00
DataBase: Composite Long 10/12/13 13:07:18
Building Code: IBC Steel Code: AISC360-05 LRFD

Floor Type: Typical **Beam Number = 47**

SPAN INFORMATION (ft): I-End (30.00,0.00) J-End (30.00,40.00)

Beam Size (Optimum) = W18X35 Fy = 50.0 ksi
Total Beam Length (ft) = 40.00

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Deck Label	Office	Office
Concrete thickness (in)	3.25	3.25
Unit weight concrete (pcf)	110.00	110.00
f'c (ksi)	3.00	3.00
Decking Orientation	perpendicular	perpendicular
Decking type	VULCRAFT 2.0VL	VULCRAFT 2.0VL

beff (in)	=	120.00	Y bar(in)	=	17.97
Mnf (kip-ft)	=	569.01	Mn (kip-ft)	=	444.57
C (kips)	=	188.13	PNA (in)	=	15.12
Ieff (in4)	=	1233.49	Itr (in4)	=	1707.04
Stud length (in)	=	3.50	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 17.1 Rg = 1.00 Rp = 0.60
# of studs: Max = 40 Partial = 22 Actual = 22
Number of Stud Rows = 1 Percent of Full Composite Action = 36.53

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.420	0.420	0.000	---	NonR	0.000	0.000
	40.000	0.420	0.420	0.000			0.000	0.000
2	0.000	0.050	0.000	0.800	22.0%	Red	0.200	0.200
	40.000	0.050	0.000	0.800			0.200	0.200
3	0.000	0.035	0.035	0.000	---	NonR	0.000	0.000
	40.000	0.035	0.035	0.000			0.000	0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 38.49 kips 1.00Vn = 159.30 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.2DL+1.6LL	173.1	20.0	0.0	1.00	0.90	249.37
	Init DL	1.4DL	127.3	20.0	---	---		
	Max +	1.2DL+1.6LL	384.9	20.0	---	---	0.90	400.11
Controlling		1.2DL+1.6LL	384.9	20.0	---	---	0.90	400.11

REACTIONS (kips):

	Left	Right
Initial reaction	13.09	13.09
DL reaction	10.09	10.09
Max +LL reaction	16.49	16.49
Max +total reaction (factored)	38.49	38.49

DEFLECTIONS: (Camber = 1-1/4)

Initial load (in)	at	20.00 ft	=	-1.771	L/D	=	271
Live load (in)	at	20.00 ft	=	-1.327	L/D	=	362
Post Comp load (in)	at	20.00 ft	=	-1.408	L/D	=	341
Net Total load (in)	at	20.00 ft	=	-1.928	L/D	=	249

Gravity Beam Design

RAM Steel v14.05.03.00
DataBase: Composite Long 10/12/13 13:07:18
Building Code: IBC Steel Code: AISC360-05 LRFD

Floor Type: Typical **Beam Number = 6**

SPAN INFORMATION (ft): I-End (20.00,0.00) J-End (40.00,0.00)

Beam Size (Optimum) = W16X26 Fy = 50.0 ksi
Total Beam Length (ft) = 20.00

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Deck Label	Office	Office
Concrete thickness (in)	3.25	3.25
Unit weight concrete (pcf)	110.00	110.00
f'c (ksi)	3.00	3.00
Decking Orientation	parallel	parallel
Decking type	VULCRAFT 2.0VL	VULCRAFT 2.0VL

beff (in)	=	36.00	Y bar(in)	=	13.85
Mnf (kip-ft)	=	340.79	Mn (kip-ft)	=	284.61
C (kips)	=	119.72	PNA (in)	=	12.64
Ieff (in4)	=	640.58	Itr (in4)	=	837.07
Stud length (in)	=	3.50	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 17.1 Rg = 1.00 Rp = 0.75
\# of studs: Full = 36 Partial = 14 Actual = 14
Number of Stud Rows = 1 Percent of Full Composite Action = 40.13

POINT LOADS (kips):

Dist	DL	CDL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	PartL	CLL
10.000	10.09	9.09	16.00	1.8	0.00	0.00	0.0	0.00	0.0	4.00	4.00

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.500	0.000	0.000	0.0%	Red	0.000	0.000
	20.000	0.500	0.000	0.000			0.000	0.000
2	0.000	0.021	0.021	0.000	---	NonR	0.000	0.000
	20.000	0.021	0.021	0.000			0.000	0.000
3	0.000	0.003	0.000	0.041	1.8%	Red	0.010	0.010
	20.000	0.003	0.000	0.041			0.010	0.010
4	0.000	0.026	0.026	0.000	---	NonR	0.000	0.000
	20.000	0.026	0.026	0.000			0.000	0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 29.22 kips 0.90Vn = 105.97 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.2DL+1.6LL	90.2	10.0	10.0	1.65	0.90	165.75
	Init DL	1.4DL	67.0	10.0	---	---		
	Max +	1.2DL+1.6LL	255.2	10.0	---	---	0.90	256.15
Controlling		1.2DL+1.6LL	255.2	10.0	---	---	0.90	256.15

REACTIONS (kips):

	Left	Right
Initial reaction	7.12	7.12
DL reaction	10.55	10.55
Max +LL reaction	10.35	10.35
Max +total reaction (factored)	29.22	29.22

DEFLECTIONS:

Initial load (in)	at	10.00 ft	=	-0.320	L/D	=	751
Live load (in)	at	10.00 ft	=	-0.315	L/D	=	761
Post Comp load (in)	at	10.00 ft	=	-0.428	L/D	=	561
Net Total load (in)	at	10.00 ft	=	-0.748	L/D	=	321

Alternative System Three

Steel Frame & Hollow Core Concrete Planks

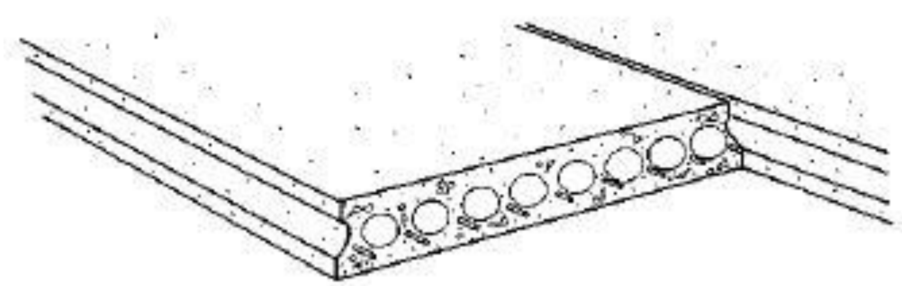


Figure 16: Steel Framing & Hollow Core Concrete Planks
by RS Means

Steel Frame & Hollow Core Concrete Planks

The third possible alternative floor system that will be investigated is hollow core concrete planks supported by steel framing. In this scenario, the columns, beams, and girders of the structure would consist of various wide flange members. The floor would be comprised of precast hollow core planks which would span the 20' dimension of the typical bay. For this system, normal weigh concrete with a topping thickness of 2.0" was selected in order to achieve a 2 hour fire rating as well as increase the strength of the section and lessen potential vibration.

This system offers limited benefits to the overall design of the structure. One major benefit is that this system significantly reduces the overall weight of the building. This is accomplished though the implementation of a lighter floor slab and the use of economical steel members in place of heavy concrete elements. By reducing the overall weight of the structure, the earthquake design load on the structure is reduced as well as the size of footings in the foundation.

A steel system in place of concrete would also affect the lateral force resisting system of the structure. In place of a concrete shear wall, the logical choices of braced frames and moment frames constructed of steel members would need to be investigated.

The use of this system also has potential disadvantages. For example, the following calculations found that the minimum floor depth of a non-composite system would be 38" deep, which is significantly deeper than the original design. This would most likely reduce the amount of leasable space, as one floor would need to be removed from the design in order to still meet height requirements of the local.

Research shows that another disadvantage of this system that it is susceptible to vibration (even with the 2" topping). This serviceability aspect can make for an unpleasant space if not dealt with properly. Further investigation would need to be done in order to assess if the amount of vibration that this system would experience.

This system relies on regular, repetitive geometries. Thus, another significant disadvantage is that this system would be hard to design, construct, and install properly given the irregularities that exist in many spaces of the architectural design.

A more complete comparison of this system against the original system and the other alternatives may be found in the latter portion of this document.

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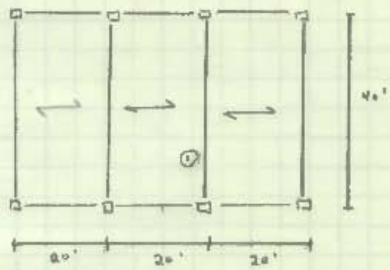
Alternative System 3 - Hollow Core Planks



LL = 80 psf + 20 psf (partitions)
SDL = 5 psf + 15 psf (topping)
Total = 120 psf

DL = 74 psf

Beam LL reducible by $.25 + 15/\sqrt{1600}$

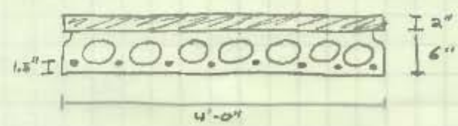
• Hollow Core Planks [4HC6-2] [see PCI Handbook]

• 4'-0" x 6" , 66-S [20' span]

allowable load = 136 psf < 120 psf ∴ ok

[Note: f'_c = 5000 psi f_{pu} = 270,000 psi]

Section view



① • Beam Design (40' span)

Strength: $\frac{w_u \ell^2}{8} = \frac{(0.232 \times 20)(40)^2}{8} = 928$ Kip-ft

$w_u = 1.2D + 1.6L = 1.2(74 + 6 + 5 + 15) + 1.6(50 + 20) = 232$ psf

Serviceability: $\frac{5 w_{LL} L^4}{384 EI_x} = \Delta_{LL} = L/360 \rightarrow \frac{5(0.1 \times 20)(40)^4(12)^3}{384(29000)(40 \times 12/360)} = I_x = 2979.3 \text{ in}^4$

∴ Choose W 30x90 [most economical]

[I_x = 3610 in⁴ , ϕM_n = 1060 kip-ft]

[Note: The following sections may be considered for height value, but are not as economical:
W 27x94 , W 24x103 , W 21x111]

[Above results are verified via computer analysis]

Strand Pattern Designation
76-S

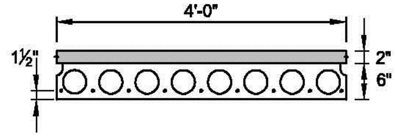
S = straight
Diameter of strand in 16ths
No. of Strand (7)

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
444 – Safe superimposed service load, psf
0.1 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

HOLLOW-CORE
4'-0" x 6"
Normal Weight Concrete



f'_c = 5,000 psi
f_{pu} = 270,000 psi

Section Properties

	Untopped		Topped	
A =	187	in.^{2}	283	in.^{2}
I =	763	in.^{4}	1,640	in.^{4}
y_b =	3.00	in.	4.14	in.
y_t =	3.00	in.	3.86	in.
S_b =	254	in.^{3}	396	in.^{3}
S_t =	254	in.^{3}	425	in.^{3}
wt =	195	plf	295	plf
DL =	49	psf	74	psf
V/S =	1.73	in.		

4HC6

Table of safe superimposed service load (psf) and cambers (in.) **No Topping**

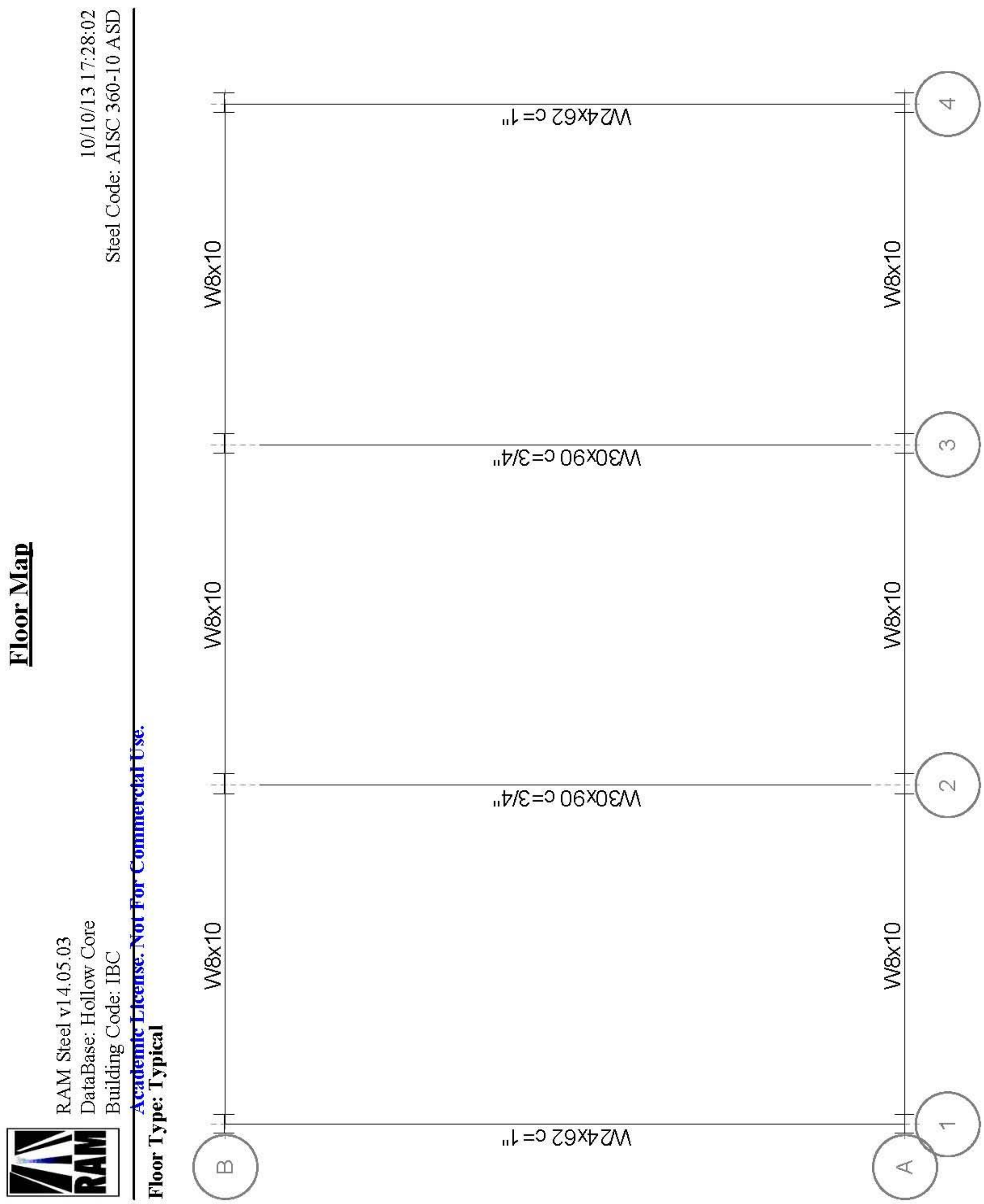
Strand Designation Code	Span, ft																				
	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	444	382	333	282	238	203	175	151	131	114	100	88	77	68	59	52	46	40	33	28	
	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7		
	0.2	0.2	0.2	0.2	0.3	0.3	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.3	–0.5	–0.7	–0.9	–1.2	–1.5	–1.9	
76-S		445	388	328	278	238	205	178	155	136	120	105	93	82	73	65	57	49	42	36	31
		0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	0.0	–0.1	–0.3	–0.4	–0.6
		0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.7	–0.9	–1.2	–1.6	–2.0
96-S		466	421	386	338	292	263	229	201	177	157	139	124	110	99	88	78	68	60	53	46
		0.3	0.3	0.3	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	–0.1
		0.3	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.3	0.2	0.1	–0.1	–0.3	–0.6	–0.9	–1.3
87-S		478	433	398	362	322	290	264	240	212	188	167	149	134	119	107	95	85	76	68	60
		0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3
		0.4	0.5	0.5	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.3	0.2	0.0	–0.3	–0.6
97-S		490	445	407	374	346	311	276	242	220	203	186	166	148	133	119	107	96	86	78	70
		0.4	0.4	0.5	0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.9	0.9	0.9	0.8	0.7	0.6
		0.5	0.6	0.6	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	0.9	0.9	0.8	0.7	0.5	0.3	0.1	–0.2

4HC6 + 2

Table of safe superimposed service load (psf) and cambers (in.) **2 in. Normal Weight Topping**

Strand Designation Code	Span, ft																		
	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	470	396	335	285	244	210	182	158	136	113	93	75	59	46	34				
	0.2	0.2	0.2	0.2	0.2	02	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2				
	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2				
76-S		461	391	334	287	248	216	188	163	137	115	95	78	63	50	38	27		
		0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	–0.0	–0.1	–0.3		
		0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2	–1.5		
96-S			473	424	367	319	279	245	216	186	160	137	116	98	82	68	55	43	33
			0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	–0.1
			0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	–0.1	–0.3	–0.5	–0.7	–1.0	–1.4	–1.7
87-S			485	446	415	377	331	292	258	224	195	169	147	127	109	94	80	67	55
			0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3
			0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.4	0.2	0.1	–0.1	–0.3	–0.5	–0.8	–1.2
97-S			494	455	421	394	357	327	288	251	219	192	168	146	127	110	95	82	70
			0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.9	0.9	0.9	0.8	0.7	0.6
			0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.2	0.0	–0.2	–0.5	–0.8

Strength is based on strain compatibility; bottom tension is limited to $7.5\sqrt{f'_c}$; see pages 2–7 through 2–10 for explanation.


Floor Map
RAM Steel v14.05.03
DataBase: Hollow Core
Building Code: IBC
Academic License. Not For Commercial Use.
Floor Type: Typical
10/10/13 17:28:02
Steel Code: AISC 360-10 ASD
W24x62 c=1"
W30x90 c=3/4"
W30x90 c=3/4"
W24x62 c=1"
W8x10
W8x10
W8x10
W8x10
W8x10
W8x10
1
2
3
4
A
B

Gravity Beam Design

RAM Steel v14.05.03.00
DataBase: Hollow Core 10/10/13 17:28:02
Building Code: IBC Steel Code: AISC 360-10 ASD

Floor Type: Typical **Beam Number = 8**

SPAN INFORMATION (ft): I-End (20.00,0.00) J-End (20.00,40.00)

Beam Size (Optimum) = W30X90 Fy = 50.0 ksi
Total Beam Length (ft) = 40.00
Mp (kip-ft) = 1179.1
7

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type	PartL
1	0.000	1.480	0.000	---	NonR	0.000
	40.000	1.480	0.000			0.000
2	0.000	0.400	1.600	37.5%	Red	0.400
	40.000	0.400	1.600			0.400
3	0.000	0.090	0.000	---	NonR	0.000
	40.000	0.090	0.000			0.000

SHEAR: Max Va (DL+LL) = 67.40 kips Vn/1.67 = 249.07 kips

MOMENTS:

Span	Cond	LoadCombo	Ma kip-ft	@ ft	Lb ft	Cb	Ω	Mn / Ω kip-ft
Center	Max +	DL+LL	674.0	20.0	0.0	1.00	1.67	706.09
Controlling		DL+LL	674.0	20.0	0.0	1.00	1.67	706.09

REACTIONS (kips):

	Left	Right
DL reaction	39.40	39.40
Max +LL reaction	28.00	28.00
Max +total reaction	67.40	67.40

DEFLECTIONS: (Camber = 3/4)

Dead load (in)	at	20.00 ft	=	-1.084	L/D	=	443
Live load (in)	at	20.00 ft	=	-0.770	L/D	=	623
Net Total load (in)	at	20.00 ft	=	-1.104	L/D	=	435

Floor System Comparison Tables

Weight, Cost, Summary

Floor System Weight Per Ft^2

System 1 - Cast In Place Concrete				
Material	**Unit**	**Unit Wt.**	**Quantity**	**Total**
Mild Reinforcing, #4, slab	lbs	1	1208	1208
Mild Reinforcing, #9, beam	lbs	1	1230	1230
Mild Reinforcing, #5, slab	lbs	1	543	543
Post tensioned strands	lbs	1	34.7	34.7
Concrete, NWC, 4.5 ksi	pcf	145	693.3	100533.3
			Sum	103549
			PSF	129.44

System 1A - Non-Composite Steel				
Material	**Unit**	**Unit Wt.**	**Quantity**	**Total**
Concrete, LWC, 3 ksi	pcf	110	282.96	31125.6
WWF W2.1xW2.1	pcsf	29	8	232
Steel Decking - 2C20	psf	1.97	800	1576
Beams - W 12x19	plf	19	120	2280
Beams - W 24x84	plf	84	40	3360
			Sum	38573.6
			PSF	48.22

System 1.2A - Non-Composite Steel				
Material	**Unit**	**Unit Wt.**	**Quantity**	**Total**
Concrete, LWC, 3 ksi	pcf	110	282.96	31125.6
WWF W2.1xW2.1	pcsf	29	8	232
Steel Decking - 2C20	psf	1.97	800	1576
Beams - W 18x35	plf	35	20	700
Beams - W 21x48	plf	48	80	3840
			Sum	37473.6
			PSF	46.84

System 2A - Composite Steel				
Material	**Unit**	**Unit Wt.**	**Quantity**	**Total**
Concrete, LWC, 3 ksi	pcf	110	304.56	33501.6
WWF W2.1xW2.1	csf	29	8	232
Steel Decking - 2VLI20	psf	1.97	800	1576
Beams - W 10x12	plf	12	100	1200
Beams - W 18x35	plf	35	40	1400
Steel Studs	lbs/ea	0.51	104	53.04
			Sum	37962.64
			PSF	47.45

System 2.2A - Composite Steel				
Material	**Unit**	**Unit Wt.**	**Quantity**	**Total**
Concrete, LWC, 3 ksi	pcf	110	304.56	33501.6
WWF W2.1xW2.1	pcsf	29	8	232
Steel Decking - 2VLI20	psf	1.97	800	1576
Beams - W 18x35	plf	35	100	3500
Steel Studs	lbs/ea	0.51	68	34.68
			Sum	38844.28
			PSF	48.56

System 3A - Hollow Core Planks				
Material	**Unit**	**Unit Wt.**	**Quantity**	**Total**
Concrete, NWC, 3 ksi	pcf	145	133.3	19333.33
Hollow Core Planks	psf	49	800	39200.00
WWF	pcsf	21	8	168.00
Beams - W 30x90	plf	90	80	7200.00
Beams - W 8x10	plf	10	40	400.00
			Sum	66301.33
			PSF	82.88

Cost Per Square Foot Estimates

System 1 - Cast In Place Concrete						
Material	**Unit**	**Mat.**	**Inst.**	**Quantity**	**Total Mat.**	**Total Inst.**
Forms, slab, in place to 15' high, 4 uses	SF	1.18	3.83	640.00	755.20	2451.20
Forms, beams, __ wide, 4 uses	SFCA	0.81	5.45	214.00	173.34	1166.30
Mild Reinforcing, #4, slab	Ton	1000.00	560.00	1.28	1280.00	716.80
Mild Reinforcing, #9, beam	Ton	1000.00	600.00	1.23	1230.00	738.00
Mild Reinforcing, #5, slab	Ton	1000.00	560.00	0.54	543.00	304.08
Post tensioned strands	Lbs.	1.06	1.20	34.70	36.78	41.64
Concrete, ready mix, NWC, 4.5 ksi	CY	105.00	0.00	25.70	2698.89	0.00
Place and Vibrate Concrete, Elevated slab, 8", crane	CY	0.00	20.14	15.81	0.00	318.51
Place and Vibrate Concrete, Elevates Beams, 8", crane	CY	0.00	53.55	9.89	0.00	529.55
Finish Floor	SF	0.00	0.49	800.00	0.00	392.00
Cure with sprayed membrane curing compound	CSF	10.95	6.15	8.00	87.60	49.20
				Totals	6804.81	6707.28
				$/SF	8.51	8.38
				Adjusted Tot. $/SF		16.87

System 1A - Non-Composite Steel						
Material	**Unit**	**Mat.**	**Inst.**	**Quantity**	**Total Mat.**	**Total Inst.**
Concrete, ready mix, LWC, 3 ksi, 110 pcf	CY	133.00	0.00	10.48	1393.84	0.00
WWF W2.1xW2.1	CSF	17.35	25.50	8.00	138.80	204.00
Steel Decking - 2C20	SF	1.79	0.50	800.00	1432.00	400.00
Beams - W 12x19	LF	28.00	4.93	120.00	3360.00	591.60
Beams - W 24x84	LF	122.00	5.30	40.00	4880.00	212.00
Place and Vibrate Concrete, Elevates slab, 6", pump	CY	0.00	20.14	10.48	0.00	211.07
Finish Floor	SF	0.00	0.49	800.00	0.00	392.00
Cure with sprayed membrane curing compound	CSF	10.95	6.15	8.00	87.60	49.20
				Totals	11292.24	2059.87
				$/SF	14.12	2.57
				Adjusted Tot. $/SF		17.05

System 1.2A - Non-Composite Steel						
Material	**Unit**	**Mat.**	**Inst.**	**Quantity**	**Total Mat.**	**Total Inst.**
Concrete, ready mix, LWC, 3 ksi, 110 pcf	CY	133.00	0.00	10.48	1393.84	0.00
WWF W2.1xW2.1	CSF	17.35	25.50	8.00	138.80	204.00
Steel Decking - 2C20	SF	1.79	0.50	800.00	1432.00	400.00
Beams - W 18x35	LF	51.00	5.96	20.00	1020.00	119.20
Beams - W 21x48	LF	70.00	5.38	80.00	5600.00	430.40
Place and Vibrate Concrete, Elevates slab, 6", pump	CY	0.00	20.14	10.48	0.00	211.07
Finish Floor	SF	0.00	0.49	800.00	0.00	392.00
Cure with sprayed membrane curing compound	CSF	10.95	6.15	8.00	87.60	49.20
				Totals	9672.24	1805.87
				$/SF	12.09	2.26
				Adjusted Tot. $/SF		14.66

	Mat.	**Inst.**	**Total**
Location Factors	103.2	96.5	100.6

System 2A - Composite Steel						
Material	**Unit**	**Mat.**	**Inst.**	**Quantity**	**Total Mat.**	**Total Inst.**
Concrete, ready mix, LWC, 3 ksi, 110 pcf	CY	133.00	0.00	11.28	1500.24	0.00
WWF W2.1xW2.1	CSF	17.35	25.50	8.00	138.80	204.00
Steel Decking - 2VLI20	SF	2.05	0.50	800.00	1640.00	400.00
Beams - W 10x12	LF	17.50	7.23	100.00	1750.00	723.00
Beams - W 18x35	LF	51.00	5.96	40.00	2040.00	238.40
Steel Studs	Ea.	2.68	1.62	104.00	278.72	168.48
Place and Vibrate Concrete, Elevates slab, 6", pump	CY	0.00	20.14	11.28	0.00	227.18
Finish Floor	SF	0.00	0.49	800.00	0.00	392.00
Cure with sprayed membrane curing compound	CSF	10.95	6.15	8.00	87.60	49.20
				Totals	7435.36	2402.26
				$/SF	9.29	3.00
				Adjusted Tot. $/SF		12.49

System 2.2A - Composite Steel						
Material	**Unit**	**Mat.**	**Inst.**	**Quantity**	**Total Mat.**	**Total Inst.**
Concrete, ready mix, LWC, 3 ksi, 110 pcf	CY	133.00	0.00	11.28	1500.24	0.00
WWF W2.1xW2.1	CSF	17.35	25.50	8.00	138.80	204.00
Steel Decking - 2VLI20	SF	2.05	0.50	800.00	1640.00	400.00
Beams - W 18x35	LF	51.00	5.96	100.00	5100.00	596.00
Steel Studs	Ea.	2.68	1.62	68.00	182.24	110.16
Place and Vibrate Concrete, Elevates slab, 6", pump	CY	0.00	20.14	11.28	0.00	227.18
Finish Floor	SF	0.00	0.49	800.00	0.00	392.00
Cure with sprayed membrane curing compound	CSF	10.95	6.15	8.00	87.60	49.20
				Totals	8648.88	1978.54
				$/SF	10.81	2.47
				Adjusted Tot. $/SF		13.54

System 3A - Hollow Core Planks						
Material	**Unit**	**Mat.**	**Inst.**	**Quantity**	**Total Mat.**	**Total Inst.**
Concrete, ready mix, NWC, 3 ksi, 150 pcf	CY	97.00	0.00	4.94	479.01	0.00
Hollow Core Planks	SF	6.30	1.99	800.00	5040.00	1592.00
WWF	CSF	14.50	23.00	8.00	116.00	184.00
Beams - W 30x90	LF	136.00	4.77	80.00	10880.00	381.60
Beams - W 8x10	LF	14.60	7.23	40.00	584.00	289.20
Place and Vibrate Concrete, Elevates slab, 6", pump	CY	0.00	20.14	4.94	0.00	99.46
Finish Floor	SF	0.00	0.49	800.00	0.00	392.00
Cure with sprayed membrane curing compound	CSF	10.95	6.15	8.00	87.60	49.20
				Totals	17186.61	2987.46
				$/SF	21.48	3.73
				Adjusted Tot. $/SF		25.77

	Mat.	**Inst.**	**Total**
Location Factors	103.2	96.5	100.6

Comparison of Floor Systems

Criteria		Existing	Alternatives		
		One-Way Slab with PT Beams	Non-Composite Steel	Composite Steel	Hollow Core Planks
General					
	Weight (PSF)	129.4	46.8	48.6	82.9
	Slab Weight (PSF)	100.0	38.9	38.9	74.0
	Overall Depth	20"	26.25"	23.25"	38"
	Slab Depth	8"	5.25"	5.25"	8"
	Cost ($/SF)	16.87	14.66	13.54	25.77
Architectural					
	Fire Rating	2HR	2HR - Beams Protected	2HR - Beams Protected	2HR - Beams Protected
	Add. Fireproofing	Not Required	Required	Required	Required
	MEP Coordination	Easy	Moderate	Moderate	Difficult
	Other Considerations	-	Reduced Floor-to-Ceiling Height	Reduced Floor-to-Ceiling Height	Non-Rectangular/ Geometry Difficult
Structural					
	Gravity	-	Reduce needed column capacity	Reduce needed column capacity	Reduce needed column capacity
	Foundation	-	Large reduction of footing size	Large reduction of footing size	Reduction of footing size
	Lateral	Moment Frame/ Shear Wall	Moment Frame/ Braced Frame	Moment Frame/ Braced Frame	Moment Frame/ Braced Frame
Serviceabitity					
	Vibration	Minimal	Very Likely	Likely	Likely
Construction					
	Formwork	Yes	Minimal	Minimal	Minimal
	Constructability	Medium	Easy	Easy	Easy
	Lead Time	Standard	Standard	Standard	Long
Further Investigate					
	Feasible	-	No	Yes	No

Appendix A

Gravity Load Documentation

J.M.V. | TECH | CODES

Design Codes - used in original design

- "International Building Code - 2009"
  International Code Council
- "Minimum Design Loads for Buildings and Other Structures" (ASCE 7)
  American Society of Civil Engineers
- "Building Code Requirements for Structural Concrete, ACI 318-02"
  American Concrete Institute
- "ACI Manual of Concrete Practice - Parts 1 through 5"
  American Concrete Institute
- "Post Tensioning Manual"
  Post Tension Institute

J.M.V. Tech Data

General Building Geometry

Level	Height	Sq. Ft.	Purpose
Roof	120'-10"	22,102	Mechanical
11	109'-1"	22,102	Office
10	97'-4"	23,058	Office
9	85'-7"	23,058	Office
8	73'-10"	23,058	Office
7	62'-1"	23,058	Office
6	50'-4"	23,058	Office
5	38'-7"	23,058	Office
4	25'-9"	23,058	Office
P6	15'-11"	24,893	Parking
PL	0'-0"	23,360	Retail/Lobby
P3	B.G.	30,388	Parking
P2	B.G.	30,388	Parking
P1	B.G.	30,388	Parking

J. M. V. TECH 2 GRAVITY

Live Loads

Area	Designed (psf)	ASCE 7-05 (psf)
Corridors (1st level)	100	100
Corridor (above 1st)	100	80
Lobbies	100	100
Marquees / Canopies	75	75
Mech. Rooms	150 (U)	125
Offices	80 + 20 (partition)	50 + 20 (partitions)
Parking garage	50 *	40 *
Retail - First Floor	100	100
Stairs / Exitways	100 (U)	100
Storage (Light)	125 (U)	125

Notes:

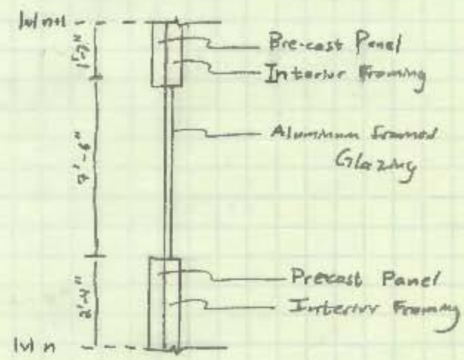
(U) = unreducible

* = 50 psf is truck / bus load
where 40 psf is vehicular load

J.M.V. TECH 2 GRAVITY

Façade Load

Typical Wall section:



Precast Panels : 5" thick [on average] with LWC

Glazing : 15 psf [assumed]

Interior Framing / wall : 5 psf [assumed]

Line Load = 4'-1" (6/12)(110 + 5) + 7'-6"(15)

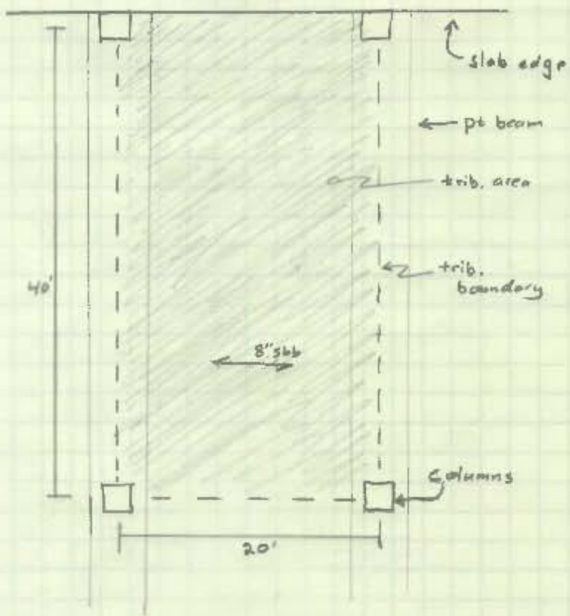
= 347.3 plf

Note: original design assumed 500 plf

J.M.V. | Tech 2 | Gravity

Typical Office Floor Loading (slab only)

Typ. Bay



Dead load:

$8''/12'' \times 150$ pcf

$= 100$ psf

SDL:

Flooring ≈ 2psf
ACT ≈ 1psf

∴ use design value of:

$= 5$ psf

Live: by IBC:

80 psf office

20 psf Partition

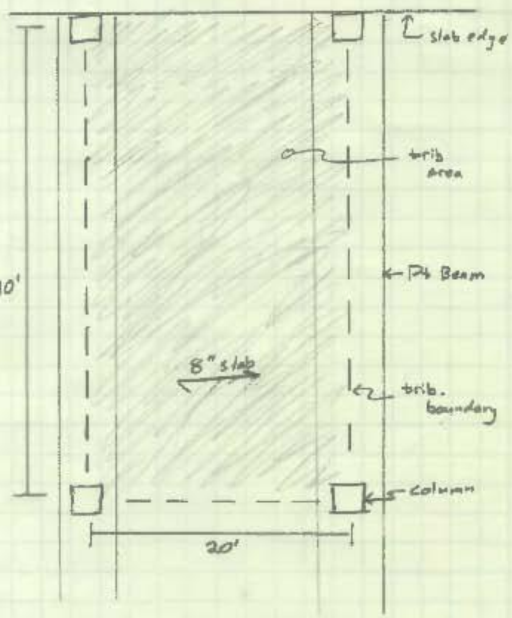
Note: see digital report for section view

Note: Live Load Reduction is possible here

J.M.V. | TECH 2 GRAVITY

Typical Roof Loading (slab only)

Typ Bay



Dead Load:

slab: 8"/12" x 150 pcf = 100 psf
vegitation: ≈ 1 psf
soil: 4"/12" x 87 pcf = 29 psf
filter/moisture mat: ≈ 2 psf
rigid insulation: ≈ 3 psf
Roof membrane: ≈ 5 psf

Σ = 140 psf

SDL:

10 psf as per CD's

RLL:

30 psf (ponding value)

SL:

17.5 psf (flat roof)

Note: see digital report for section view

J.M.V. TECH 2 SNOW

Exposure Category - B
Occupancy - II

Exposure factor $C_e = 1.0$ [Table 7-2]

Thermal factor $C_t = 1.0$ [Table 7-3]

Importance factor $I_s = 1.0$ [Table 7-4]

Flat Roof Snow Loads, p_f

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

p_g = ground snow load = 25 psf

$$p_f = 0.7(1.0)(1.0)(1.0)(25)$$

p_f = 17.5 psf

J.M.V. TECH 2 SNOW

Exposure Category - B

Occupancy - II

Exposure factor $C_e = 1.0$ [Table 7-2]

Thermal factor $C_t = 1.0$ [Table 7-3]

Importance factor $I_s = 1.0$ [Table 7-4]

Flat Roof Snow Loads, p_f

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

p_g = ground snow load = 25 psf

$$p_f = 0.7(1.0)(1.0)(1.0)(25)$$

$$p_f = 17.5 \text{ psf}$$

J.M.V. | Tech 2 | Snow

Snow Drift

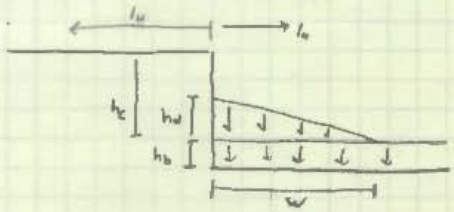
Snow density

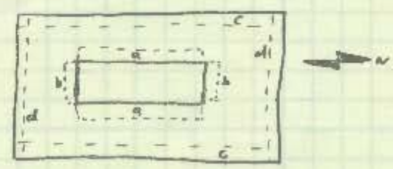
$$\gamma = 0.13 p_g + 14 \leq 30 \text{ pcf} \qquad [\text{Eq } 7\text{-}3]$$

Height of snow drift, h_d

Leeward: $h_d = (3/4)[0.43 (l_u)^{1/3} (p_g + 10)^{1/4} - 1.5]$ [Fig 7-9]

Windward: $h_d = (3/4)[0.43 (l_u)^{1/3} (p_g + 10)^{1/4} - 1.5]$ [Sect 7.7.1]





Value of "w"

- if $h_d \leq h_c$ ∴ $w = 4h_d$
- if $h_d > h_c$ ∴ $w = 4h_d^2/h_c$

Area	h_d	w
a	2.4'	9.6'
b	4.0'	16'
c	[2.7']	15.7'
d	[3.5']	26.1'

Appendix B

Building Drawings

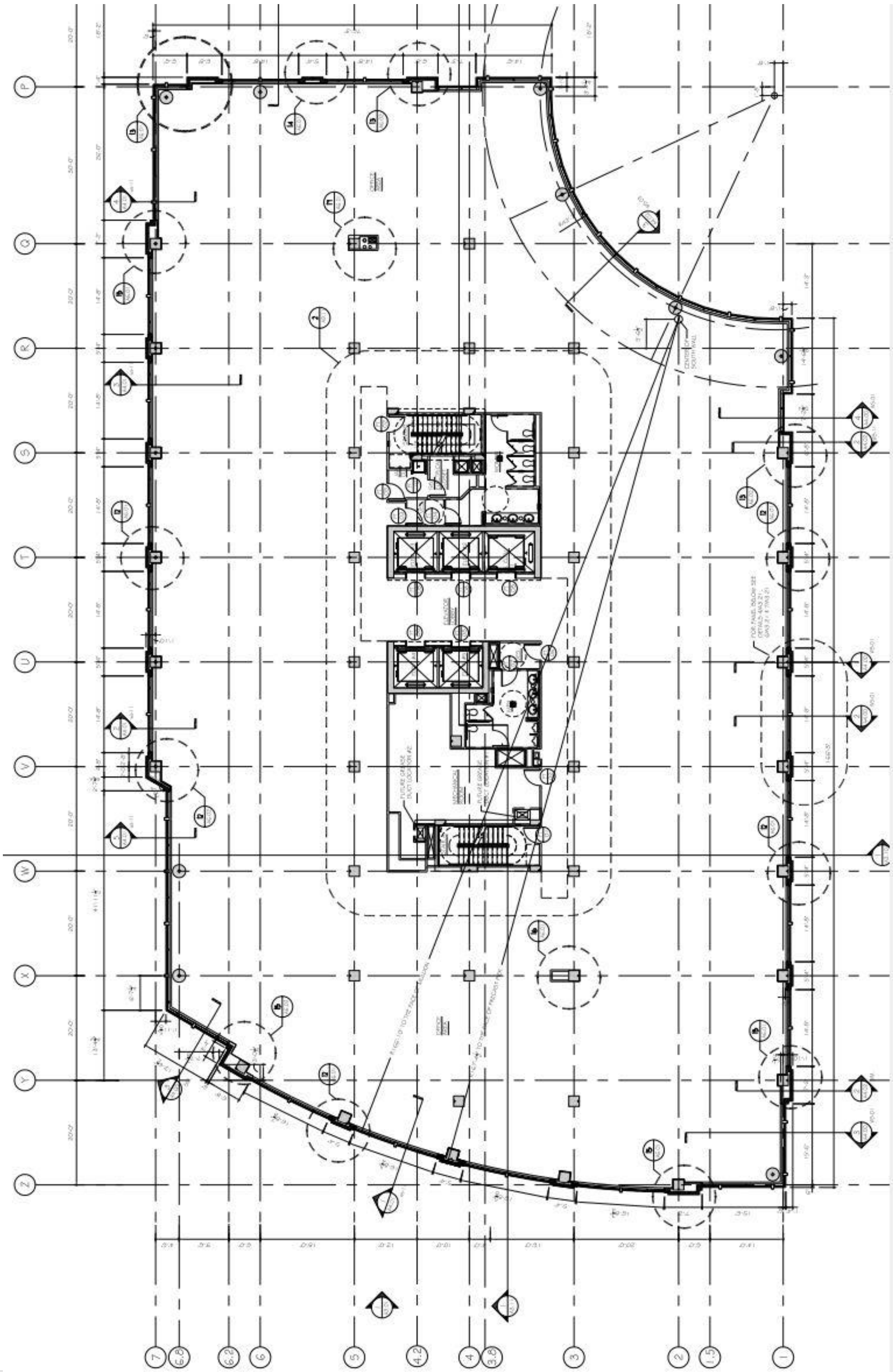


Figure 17: Typical Office Floor Plan – A2.19 of Construction Documents

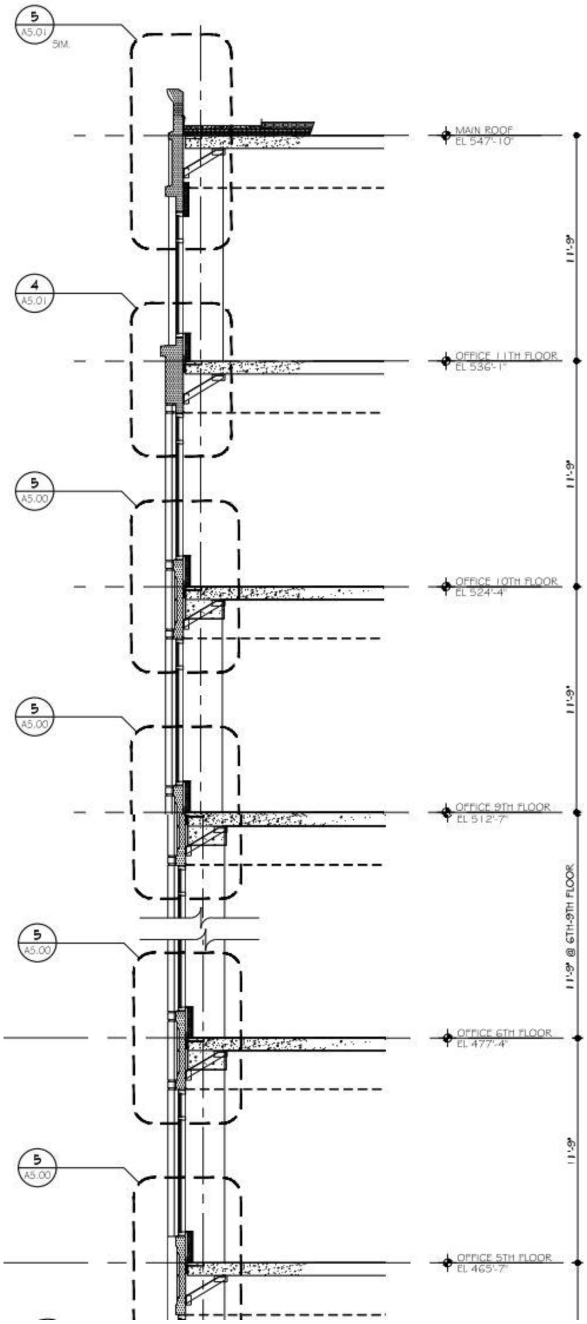


Figure 18: Wall Section – A4.05 of Construction Documents

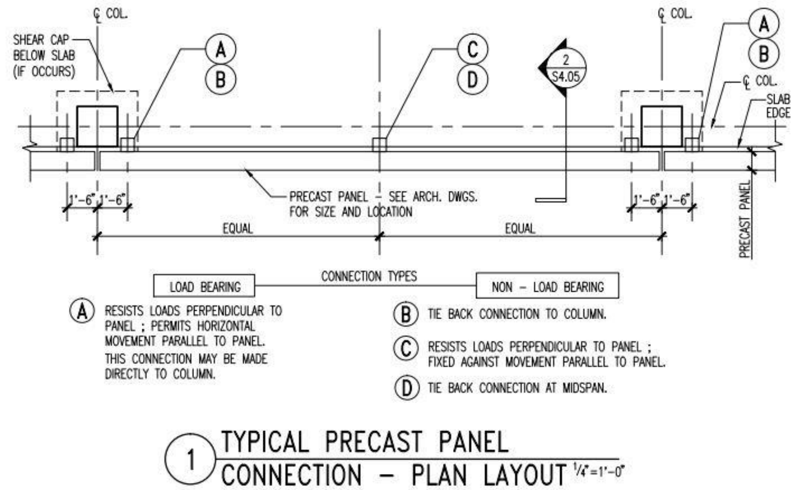


Figure 19: Precast Connection Plan – S4.01 of CD's

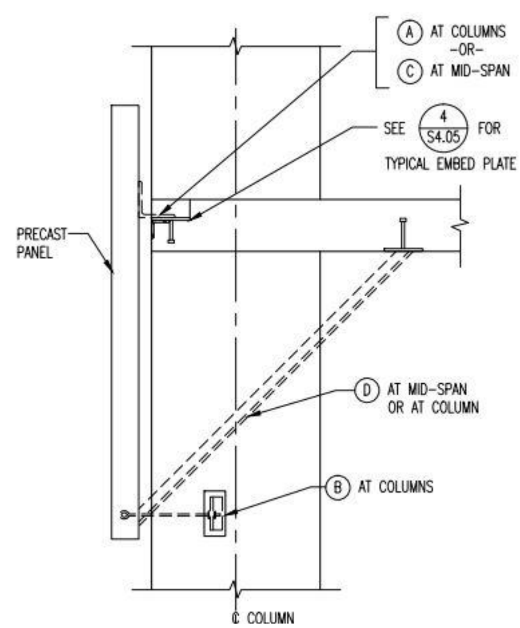


Figure 20: Precast Connection Detail – S4.01 of CD's

Appendix C

Photos

Figure 21: Decorative Precast Panel – by JMV

Figure 23: Unfinished Retail Space – by JMV

Figure 24: South West Corner – by JMV

Figure 22: North East Curtain Wall – by JMV

Figure 25: Projection of Post Tension Beam – by JMV