FINAL REPORT

University Academic Center

Eastern USA

Alexander Altemose

Structural Option

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University Academic Center

Eastern USA

General Information

Function:	Mixed use (A-3, B, M, S-1)
Size:	192,000 sq. ft.
Height:	72 ft.
Constructed:	September 2005 – August 2007
Project Cost:	$55.7 million

Project Team

Owner:	Multiple Universities
General Contractor:	Skanska USA
Architect:	Cannon Design
Structural Engineer:	Columbia Engineering
Mechanical/Electrical Engineer:	Cannon Design

Architecture

- 3 wing building containing 45 classrooms, over 120 offices, full kitchen dining service, a bookstore, and library resource center
- Varying façade using glass curtain wall, metal panels, brick, stone, and glazed CMU
- Highly sustainable design incorporating solar shading, low-E coated glass and accessible roof gardens to achieve a LEED Gold rating

Structural

- Slab on grade foundation and spread footings
- Steel framing using mainly wide flange members
- Concentrically braced framing for lateral support
- Floor system is mostly composite decking using LWC

MEP

- Demand ventilation with occupancy sensors to minimize energy consumption
- VAV systems both with and without reheat used
- 277/480V 3 phase - 4 wire system
- Majority of lighting consists of fluorescent and compact fluorescent in interior with metal halide on exterior
- Building is protected by a fully automatic wet-pipe sprinkler system

Alexander Altemose **Structural Option**

http://www.engr.psu.edu/ae/thesis/portfolios/2013/aka5074/index.html

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Acknowledgements

I would like to thank the following groups and individuals for their support in completing this thesis report.

- For permission to use the University Academic Center for my thesis project:

  The Owner (who wished to remain anonymous)

- For supplying and help in interpreting the Construction Documents:

  Skanska USA Building Inc. (especially Karena Verkempinck and Paul White)

- For sharing in the joys and challenges associated with the AE program, the bouncing off of ideas, and continued support:

  The entire AE student body (especially fellow 5th years)

- And finally, for the past five years of continued education and guidance:

  The entire AE faculty

Executive Summary

The University Academic Center was designed as a composite steel structure with braced frames. It houses all elements of a typical education center including classrooms, staff offices, a library, dining facilities, and fitness center. The building has three main wings and multiple roof levels including a roof garden. This report will focus in on the south office wing and its redesign as a concrete structure separated from the main building.

In the beginning of this process of redesign, the office wing presented itself as the best choice for a concrete structure. It had relatively repeatable floor plans which could save on formwork costs. This also made reinforcing layouts more uniform throughout since each floor saw similar loading. When considering architecture, the floor plan of the office wing was also compatible with a concrete redesign where the new column locations did not interfere drastically with any of the spaces.

Overall this redesign consisted of a one-way pan joist floor system with an ordinary moment frame system to resist lateral forces. All concrete used on for this redesign was 5000psi except for the foundations which kept the 4500psi noted in the construction documents. Joists and beams were designed 20" thick cast integrally with the 5" slab, totaling a 25" overall depth. This floor system was repeated on all floors and roof for sake of time. Columns were also all designed the same with a 24"x24" section and (12)#8 vertical bars as reinforcement. Together these members resisted the calculated wind and seismic loading with seismic controlling most of the design.

The added weight of concrete versus steel created several issues, one of which was column line L-2 (referenced in both the ETABS and RAM models used in this report) located above the exterior walkway. This was corrected by a 36" deep beam spanning across the walkway that took the load from the columns above into the foundations. Another issue was the increased demand on the foundations requiring a redesign. This was done using RAM Foundation with spot checks to determine validity of results. Foundation sizes increased but were still reasonably sized so spread footing could still be used effectively.

In addition to the structural depth, two breadth topics were discussed. The construction breadth focused on the cost and scheduling concerns with the redesigned concrete structure. This resulted in the concrete system costing less but construction time being considerably longer than that of the original steel. For that reason the steel system was determined the more preferable design.

The other breadth, a lighting redesign of a computer lab located on the 2nd floor of the office wing, focused on changing the current recessed lighting to a pendant lighting design as an alternative. This redesign reduced the number of fixtures, which also reduced the power consumption, while maintaining a recommended illuminance value of 30 footcandles.

Introduction

Located in the eastern United States, the University Academic Center is a 192,000 square foot building designed to house a library resource center, dining area, 45 classrooms, and over 120 offices. Other key features include a 5-story atrium and multiple roof gardens.

The layout of the building consists of three main sections. The northern 3-story section contains mostly dining and classroom areas. In the center of the building, a 4 story section houses the library and the majority of classrooms, as well as acting as the main entrance. The southern end of the building consists almost entirely of office spaces. On either side of the center section are the vertical circulation cores which also provide access to the roof gardens.

There are 4 main types of building façade incorporated in this building. The 3 and 5 story sections of the building have a brick façade with cast stone bands running horizontally across the brick surface. Glass curtain walls are used in the vertical circulation located on either side of the 4-story section. The 4-story section's façade is mostly metal panels. There is also glazed CMU used to accent the other façade types at various places.

By implementing multiple energy saving techniques, University Academic Center holds a LEED gold rating. This includes energy efficient HVAC equipment and the use of natural daylighting, as well as shading devices, to help minimize energy consumption. All these features, along with the roof gardens, provide a "green" learning environment. LEED credits were also gained through site design to minimize storm water runoff, use of recyclable and local materials, and the addition of bike racks and on site showering facilities to promote alternative modes of transportation.

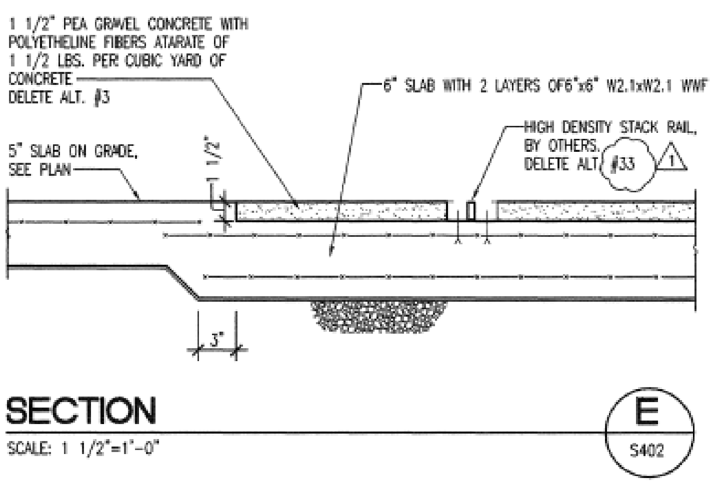
Structural Overview

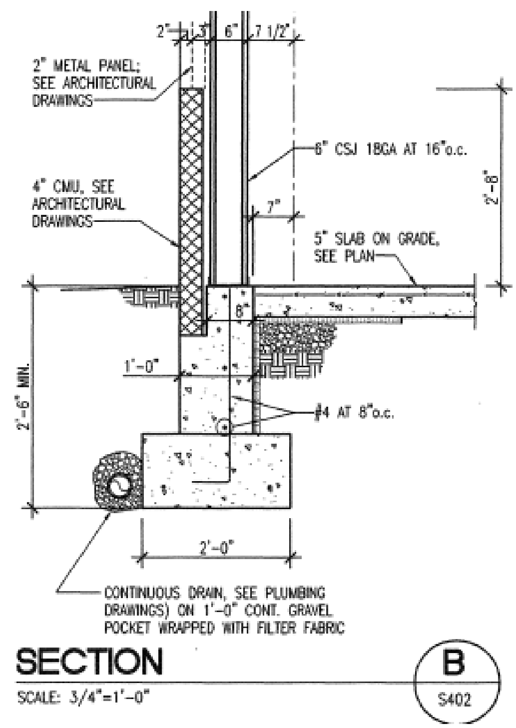
The University Academic Center is a steel framed building with composite metal decking supported by a foundation of spread footings and slab-on-grade. The building resists lateral forces by a combination of braced and moment frames.

Foundation

Based on the 2002 geotechnical report taken, footings for University Academic Center are designed for an allowable bearing capacity of 3,000 psf. Footings are placed on undisturbed soil or on structurally compacted fill. The bottoms of exterior footings are a minimum of 2'-6" below grade to protect against freeze-thaw affecting the foundations.

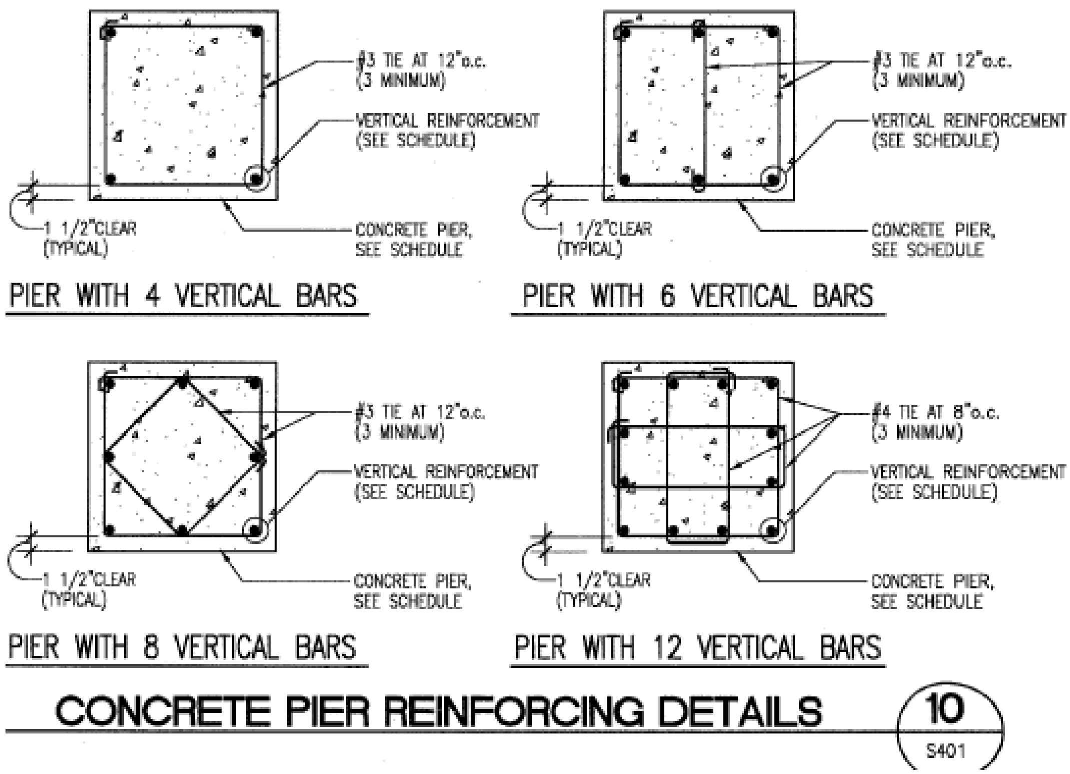
Slab-on-grade sits on a coarse granular fill material compacted to 95% of maximum density as defined by ASTM D1557 modified proctor test. The slab-on-grade is designed as 5" thick concrete reinforced with 6"x6", W1.4xW1.4 WWF. This is the reinforcement for all slab-on-grade except for the area located under the library stacks which is 6" thick concrete reinforced with 2 layers of 6"x6", W2.1xW2.1 WWF to account for the increased loading in this area.





Drawings provided by Skanska

The columns in the University Academic Center bear on piers ranging in size depending on loading and connection type. The piers come in 4 configurations: 4, 6, 8, and 12 vertical bar reinforced piers based on axial load taken from the columns above into the footings. Footings also range in size under the columns with a maximum 19’x19’, 34” deep footing under a single column. Foundations also include continuous footings around perimeter walls and combined footings.

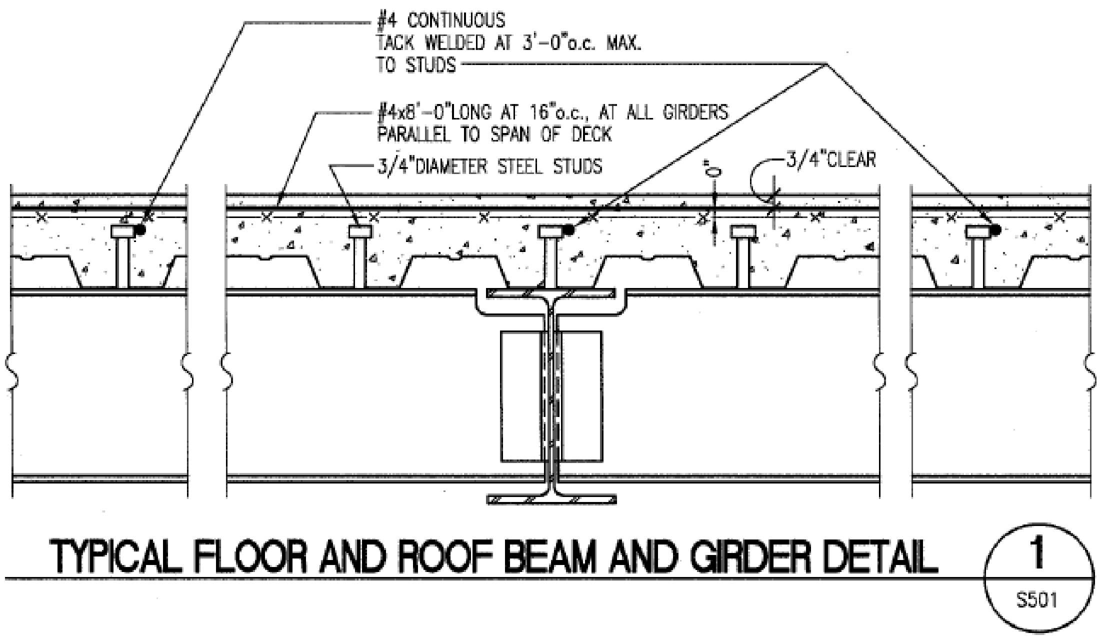


Drawings provided by Skanska

Floor and Roof Systems

The University Academic Center uses a composite metal deck flooring system. This includes 2” composite 20 gage deck with ribs 12” o.c. and 1.5” type B, wide rib 20 gage deck. All metal deck is designed to be continuous over 3 spans. The floor system also includes shear studs and lightweight concrete topping varying in thickness based on location and loading.

Roofing systems also vary due to some areas like the roof gardens and mechanical spaces of greater loading. Decking for roofs includes both 2” composite 18 gage deck with ribs 12” o.c. and 1.5” type B, wide rib 20 gage deck, covered by a built up roof and rigid insulation.



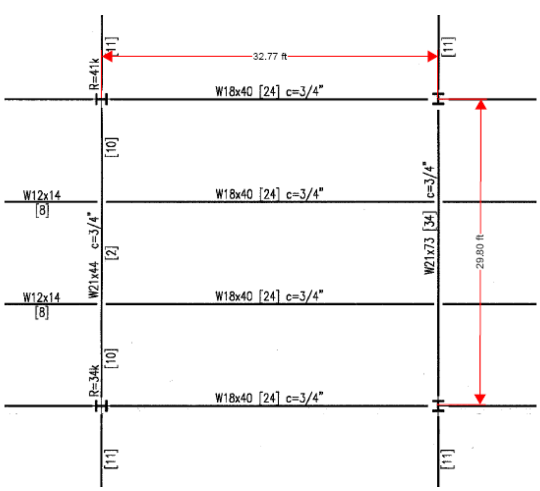
Drawings provided by Skanska

Framing System

The framing system for the University Academic Center includes C-shapes, HSS members, and Wide Flange members with the majority being W-shapes. Gridlines are set at multiple angles with bay sizes varying throughout the building. Areas with consistent framing between floors are located in the classroom wing in the central section of the building and the office spaces on the south side. The gravity system transfers vertical loads due to dead, live, and snow loading across a floor or roof deck, into beams and girders, and is taken as axial force in columns to the foundation.

Lateral System

The lateral system for this building includes braced frames of varying heights and types located throughout the building. Below is a plan view of University Academic Center with the 15 lateral braced frames shown in blue. These frames resist the forces on the building due to wind and seismic loading. The wind loads are taken into the floor diaphragm from the façade and distributed amongst the bracing based on relative stiffness. The frames in turn transfer these loads to the foundation. A braced framing system is logical with a steel building given the lightweight paired with relative stiffness. Where shear walls would limit the circulation throughout the building, using knee braces, as University Academic Center does in multiple locations, allows for more useable space. Braced frames are also stiffer than moment framing alternatives and cheaper to construct.



Drawings provided by Skanska

Proposed Structural Depth

The completion of technical reports 1, 2, and 3 showed the current structural systems used in University Academic Center are adequate in meeting both strength and serviceability requirements. This eliminates any need to redesign in order to fix issues or meet codes. Instead this next phase of thesis work will be dedicated to redesigning the building to expand knowledge of structural systems.

With the current building being composed entirely of steel systems, the option of redesigning the office wing with a concrete structural system will be done in order to further knowledge in concrete design. This option will include designing a new flooring system and designing the concrete moment frames to resist both gravity and lateral forces. The office wing is the most suited for a concrete system with its masonry enclosure already giving it a more massive feel, and its repeated floor layouts.

The research into alternate flooring systems done in technical report 2 suggested a two-way slab flooring system would offer advantages over the existing composite steel system such as price and floor-to-floor heights. However, because a goal of this report will be minimizing changes to the architect's vision for the building, floor-to-floor heights will remain unchanged. This opens options for deeper concrete flooring systems capable of maximizing spans and possibly eliminating columns. A one-way joist system will be studied as an alternative flooring system.

The lateral system will also be redesigned in the form of concrete moment frames in the office wing as opposed to the current braced frame system. The change to a concrete system and effects this will have on lateral design will be determined through lateral analysis, including calculations of displacements/drifts compared to code required values.

Cracking and settlement issues could become a problem when connecting two differing structural systems. For this reason the two buildings will be separated by an expansion joint to isolate the structures allowing safe displacements in either structural system without harming the other.

The foundation must also be investigated in the new concrete wing to ensure the added weight will still be supported by the foundation. If this is not the case the foundation will have to be redesigned. The redesigned foundation will then be determined feasible; if not an alternative type of foundation will be considered.

Proposed Construction Breadth

The building of a concrete office wing will place a big change on the building's construction; this change will be addressed along with a cost comparison of the concrete system versus the composite steel system currently employed in a construction breadth. Detailed take-offs of material costs using RSMeans will compare the two systems and determine which is cheaper. Schedules for both the concrete and steel office wing designs will be made to determine effects on construction times. These construction issues will help in determining the overall feasibility of such a change.

Proposed Lighting Breadth

The second floor of the office wing includes many computer labs. Lighting design says that spaces with computer screens benefit from indirect lighting to reduce glare on monitors. Current lighting in these spaces consists of recessed direct lighting. Because of this the lighting in one of these spaces will be redesigned with a new pendant lighting layout.

A computer lab will be chosen and analyzed with AGi32 software to determine current lighting levels and total power usage. Then new pendant lighting will be selected to replace the recessed lighting. The interior space will then be reanalyzed to determine if lighting levels or power consumption changed. Rearranging of pendant lighting will be done if new lighting levels are too high or low until levels are acceptable. This change could offer the owner a possible refit option in the future.

Design Codes and References

As Designed:

- 2000 ICC International Building Code
- 2000 ICC International Energy Conservation Code
- 2000 Americans with Disabilities Act – Accessibility Code
- 1999 National Electrical Code
- AIC 318 "Building Code Requirements for Structural Concrete"
- AIC 530 "Building Code Requirements for Masonry Structures"
- AISC Manual of Steel Construction (locally approved edition)
- ANSI "Structural Welding Code"

Thesis Calculations:

- 2009 International Building Code
- American Society of Civil Engineers ASCE 7-10
- AISC Steel Construction Manual, 14th Edition
- ACI 318-11 "Building Code Requirements for Structural Concrete"
- Vulcraft steel deck catalog
- Concrete Floor Systems: Guide to Estimating and Economizing, 2nd Edition
- IES Handbook, 10th Edition
- Pearson Construction Technology: Penn State-AE 311 Fundamentals of Building Electrical and Illumination Systems
- RS Means Building Construction Cost Data 2012

Load Combinations

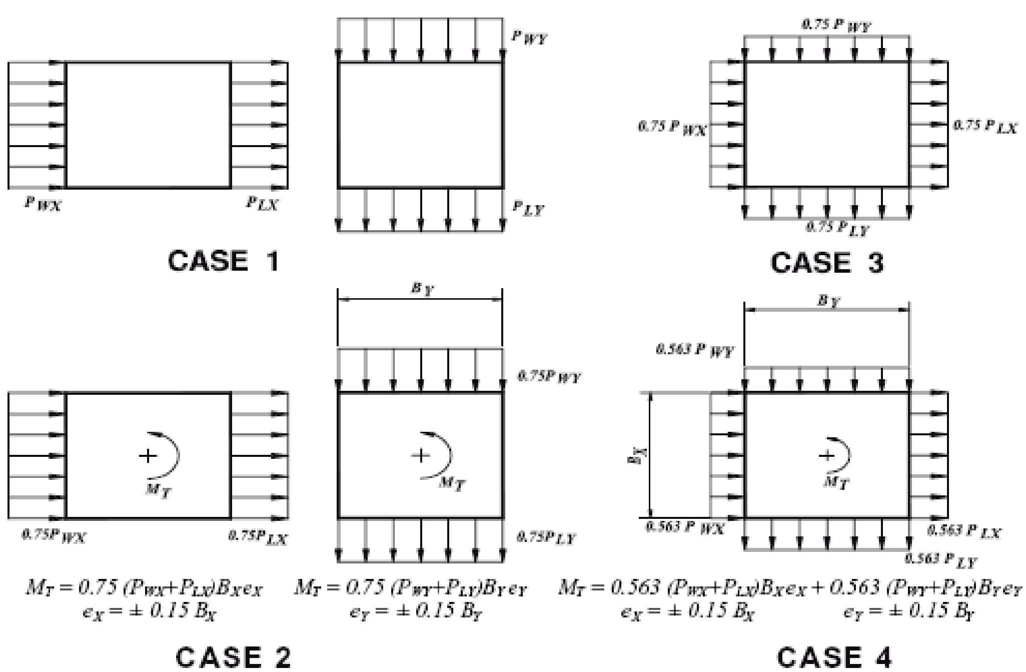
Load combinations taken from ASCE7-10 used in this report are shown below. It can be deduced that load combination 2 will control in members analyzed as gravity members. Whereas the design of lateral members will be done using combination 4 when wind loading controls and combination 5 when seismic loading controls.

2.3.2 Basic Combinations

Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations:

1. $1.4D$
2. $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
4. $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.0W$
7. $0.9D + 1.0E$

Further breakdown of the wind loading must be done to include all cases as described in Figure 6-9 of the ASCE7-10 shown below. The controlling case will act as the wind loading when using the load combinations above.



Design Loads

Previous technical reports had determined loading for the entire structure. Since the office wing was now being considered a separate concrete structure, new loading calculations would need to be done for each structure separately. Values for dead and live loads would remain the same but the forces obtained for wind and seismic loading calculations must be redone for both structures. This report focused solely on the loading and design of the office wing when designed as concrete.

Dead Loads

Dead loads were estimated based off material weights found in the AISC Steel Construction Manual since no values were given on drawings except for weights of rooftop units which range from 8,000-45,000 lbs. Deck weights were compared to similar weights in Vulcraft catalog based on topping thickness and deck type.

Dead Loads	
Description	Load (psf)
Steel Framing	10
Superimposed DL	10
MEP	10
Composite Deck	
3.25” LCW topping	42
4.75” LCW topping	50
5” NWC topping	70
Roof Garden	80
Façade	
Brick	40
Glass	10
Metal Panel	15
NW Concrete	150 (pcf)

Live loads

Live load values were given on the drawings. These values are shown, along with the values given in ASCE7-10, in the table below. Where values were not given in one source the value from the other source was used in calculations. Likewise, when differing values are present the larger of the two was used in thesis calculations.

When input into modeling software these loads were considered irreducible to minimize inconsistency with any hand calculations since live loads were kept unreduced in hand calculations to save time.

Live Loads		
Description	Designed Load (psf)	ASCE 7-10 Load (psf)
Slab on grade	100	100
Library slab on grade	150	150
Storage	125	125
Offices	50 + 20 (partitions)	50 + 15 (partitions)
Classrooms	40 + 20 (partitions)	40 + 15 (partitions)
Corridors (elevated floors)	80	80
Lobbies	100	100
Recreational areas	100	100
Mechanical/Electrical	125	N/A
Stairs	100	100
Chiller room	150 + equipment	N/A
Boiler room	200 + equipment	N/A
Roof	30	20
Roof Garden	N/A	100

Snow Loads

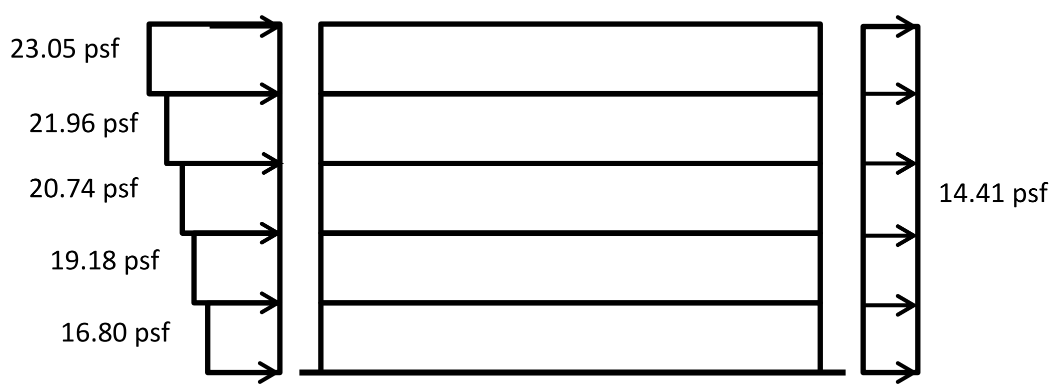
With the use of flat roofs, both uniform snow loading and drifting must be factored into design. Using ASCE7-10 to confirm the design loads used on the building were efficient, a flat roof snow load of 15.75 psf was calculated. According to the plans, the building was designed conservatively for a snow load of 20 psf. This 20 psf load was the value used in design of the new concrete office wing.

However, it was also important to consider how influential snow drifts around the parapet walls and mechanical penthouse would be on roof members. The hand calculations for this can be found on page 37 in Appendix A.

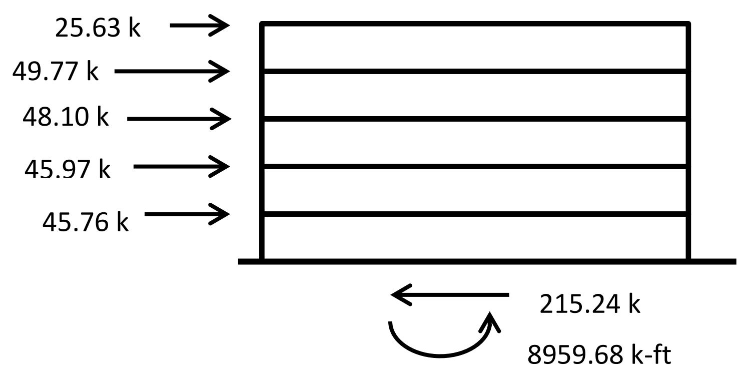
Wind Loads

Wind loads were calculated using the Directional Procedure found in ASCE7-10 Chapter 27. Preliminary values taken from the drawings along with detailed calculations in determining wind loads can be found on page 38 in Appendix A. The wind pressures were then taken and converted into story forces, as seen on the following page, for later use in ETABS and RAM lateral modeling software.

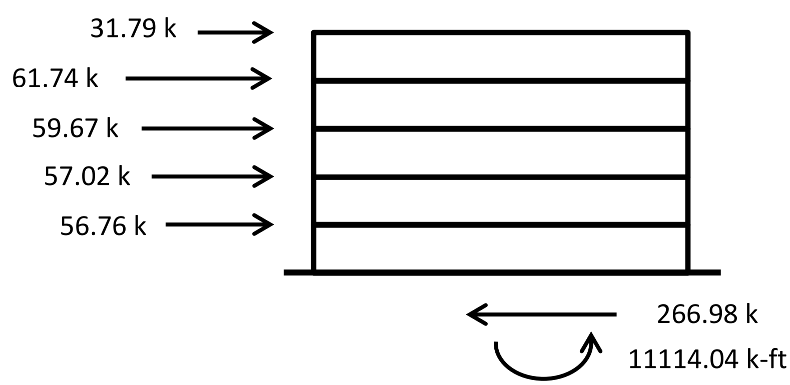
Wind Pressures					
Location	Height (ft)	q_z (psf)	C_p	Wind Pressure (psf)	Internal Pressure (psf)
Windward	0-16	33.9	0.8	23.05	+/- 6.1
	16-30	32.3	0.8	21.96	+/- 6.1
	30-44	30.5	0.8	20.74	+/- 6.1
	44-58	28.2	0.8	19.18	+/- 6.1
	58-72	24.7	0.8	16.80	+/- 6.1
Leeward	0-72	33.9	0.5	14.41	+/- 6.1



Wind Forces (E-W)							
Floor Level	Elevation (ft)	Façade Height (ft)	Façade Length (ft)	p (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (k-ft)
Roof	72	7	97.75	37.46	25.63	25.63	1845.36
5	58	14	97.75	36.37	49.77	75.41	2886.66
4	44	14	97.75	35.15	48.10	123.51	2116.4
3	30	14	97.75	33.59	45.97	169.48	1379.1
2	16	15	97.75	31.21	45.76	215.24	732.16
Total Base Shear =					215.24		
Total Overturning Moment =							8959.68



Wind Forces (N-S)							
Floor Level	Elevation (ft)	Façade Height (ft)	Façade Length (ft)	p (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (k-ft)
Roof	72	7	121.25	37.46	31.79	31.79	2288.88
5	58	14	121.25	36.37	61.74	93.53	3580.92
4	44	14	121.25	35.15	59.67	153.2	2625.48
3	30	14	121.25	33.59	57.02	210.22	1710.6
2	16	15	121.25	31.21	56.76	266.98	908.16
Total Base Shear =					266.98		
Total Overturning Moment =							11114.04

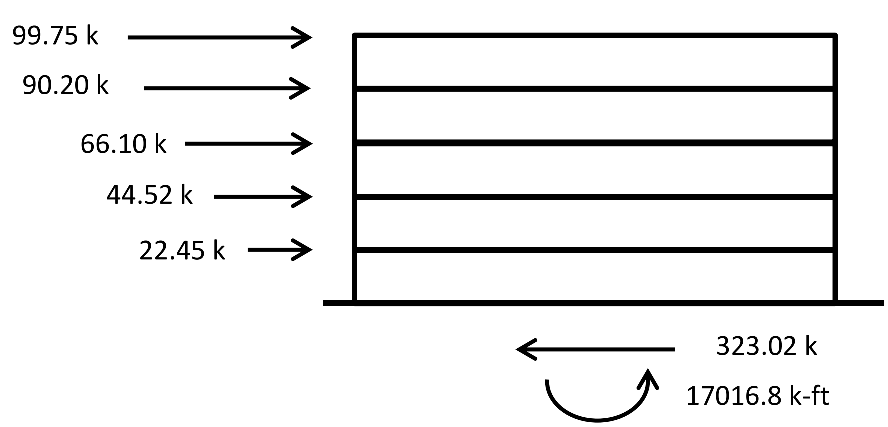


Seismic Loads

Seismic loading was designed using the Equivalent Lateral Force Procedure as outlined in ASCE7-10 to follow the process used on the University Academic Center as stated in the construction documents which gave a site class D, S_{DS} =0.21, and S_{D1} =0.11. However from previous technical reports, the values for the spectral response coefficients were already in question. So as an alternative, values were obtained using the building location in the USGS Seismic DesignMaps application, resulting in values of S_{DS} =0.167, and S_{D1} =0.081. These values place the office wing in Seismic Design Category B. Further calculations can be seen on page 39 of Appendix A.

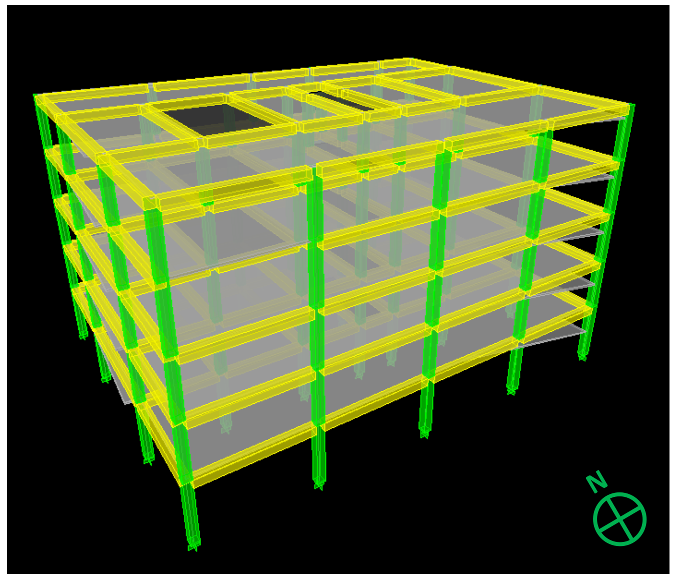
To determine seismic story forces, a base shear needs to be calculated using the value C_S obtained through the Equivalent Lateral Force Procedure mentioned above and the total weight of the building. A breakdown of building weights and the resulting calculations to find seismic story forces and ultimately the base shear and overturning moment due to seismic forces is shown in the table below.

Seismic Forces (N-S) & (E-W)							
Floor	Height h (ft)	Weight w_x (kips)	$w*h^k$	C_{vx}	Story Force F_x (kips)	Story Shear V_x (kips)	Overturning Moment (k-ft)
Roof	72	1567.64	192639.5	0.31	99.75	99.75	7182
5	58	1807.88	174191.2	0.28	90.20	189.95	5231.6
4	44	1807.88	127659.7	0.20	66.10	256.05	2908.4
3	30	1873.27	85972.9	0.14	44.52	300.57	1335.6
2	16	1916.15	43357.5	0.07	22.45	323.02	359.2
Totals	-	8972.82	623820.9	1	323.02	-	17016.8

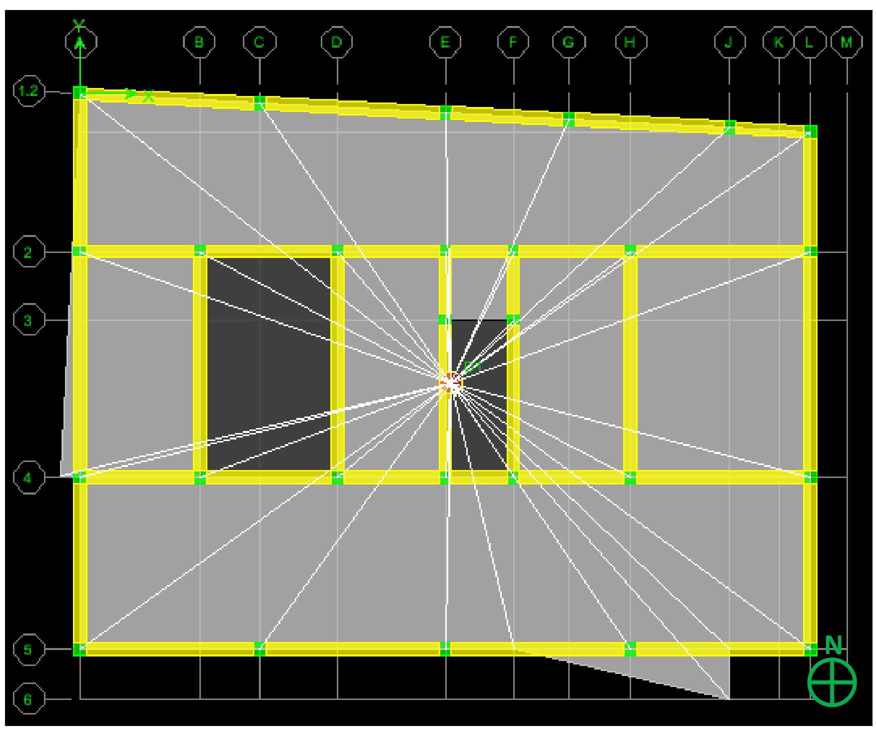


Computer Model

An ETABS model was used to determine story drifts for the new office wing and compared to allowable values from ASCE7-10 for seismic loading and the accepted value of h/400 for wind loading. This model was also used to calculate member forces due to wind and seismic loading in designing the moment frames. The new concrete office wing was modeled as shown below including only the moment frames used to resist the lateral forces due to wind and seismic.



3-D view of the office wing lateral system modeled in ETABS



Plan view of the office wing lateral system modeled in ETABS (Roof Level)

To improve the validity of ETABS output, data was modeled under the following parameters:

- Mass is lumped at the story levels using the Additional Area Mass option and setting all other self-weights equal to zero.
- f'c = 5000psi for all members
- Beams were all modeled with 25" depth and 24" width and $I_{cr} = 0.35*I_g$ in strong axis bending.
- Columns were all modeled as 24"x24" and $I_{cr} = 0.7*I_g$ in both axes for bending.
- Supports assumed fixed from rotation in all directions.
- Diaphragms modeled as rigid.

The following load combinations were analyzed to account for all scenarios of both wind and seismic loading described in ASCE7-10. Wind forces were applied at the building's center of pressure while seismic forces were applied at the centers of mass. ETABS also accounted for accidental torsion with an eccentricity of 0.05. From the results of these load cases it was determined the largest forces and displacements came when the model was loaded under load case 13; meaning seismic forces controlled the lateral design.

Load Cases for Wind		
1	WX	Wind Case 1
2	WY	
3	$.75WX+.75M_T$	Wind Case 2
4	$.75WX-.75M_T$	
5	$.75WY+.75M_T$	
6	$.75WY-.75M_T$	
7	.75WX+.75WY	Wind Case 3
8	$.563WX+.563WY+.563M_T$	Wind Case 4
9	$.563WX+.563WY-.563M_T$	
Load Cases for Seismic		
10	EX+Accidental Eccentricity	
11	EY+Accidental Eccentricity	
12	EX-Accidental Eccentricity	
13	EY-Accidental Eccentricity	

Story Drifts

Using the ETABS model results, story drifts were found for the new office wing and compared to the allowable limits for both wind and seismic loading. Values for story drifts are shown in the tables below.

ASCE7-10 defines the allowable story drift for seismic design in Table 12.12-1 based on occupancy category, structure type, and story height. This value is compared to the amplified displacement found from ETABS output multiplied by an amplification factor, C_d, based on the type of lateral system, and divided by the Importance factor, I_e. The controlling load case for seismic drift was load case 13.

Story drifts for wind were compared to the accepted value of h/400 for serviceability purposes. The controlling case varied over the height of the building with load case 2 controlling on the top 2 floors of the office wing, while load case 6 controlled in the bottom floors.

Office Wing Story Drifts (Wind)						
Floor	Story Height (ft)	Drift X (in.)	Drift Y (in.)	Controlling Load Case	Allowable Drift (in.)	Pass?
Roof	14	0.002	0.098	2	0.42	YES
5	14	0.003	0.171	2	0.42	YES
4	14	0.055	0.250	6	0.42	YES
3	14	0.070	0.301	6	0.42	YES
2	16	0.052	0.237	6	0.48	YES
Total	72	0.24	1.06	6	2.16	YES

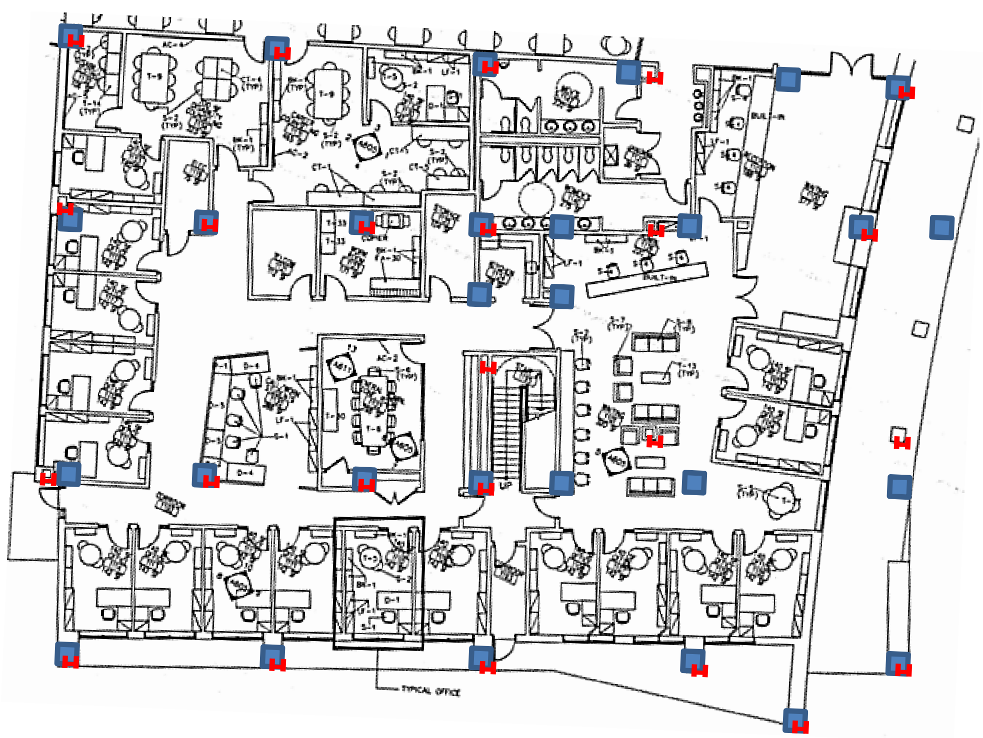
Office Wing Story Drifts (Seismic)						
Floor	Story Height (ft)	Amplified Drift X (in.)	Amplified Drift Y (in.)	Controlling Load Case	Allowable Drift (in.)	Pass?
Roof	14	0.051	0.598	13	2.52	YES
5	14	0.084	0.900	13	2.52	YES
4	14	0.110	1.144	13	2.52	YES
3	14	0.126	1.211	13	2.52	YES
2	16	0.088	0.860	13	2.88	YES
Total	72	0.475	4.725	13	12.96	YES

Structural Depth: Office Wing Redesign

The main goal of this structural depth was to further knowledge in design through the creation of a concrete structural system as opposed to the as-built steel system studied in the technical reports in the previous semester. The office wing was chosen for this redesign due to its relatively uniform layout which will allow for the reuse of formwork as well as its brick façade giving it a more massive feel. The office wing also served a separate function than the rest of the building. Where the building was overall a public space the office wing became more of a private area. With the space already filling a differing role from the rest of the building, imagining this wing as a completely separate structure became easier.

To account for the two structural systems differing reactions to loading, an isolation joint was proposed. This would allow both structures to shift and settle under loading independently of one another. Based on the maximum drift from ETABS output previously discussed, the isolation joint should be at least 5" to ensure separation of the structures.

Another goal of this concrete redesign was to minimize the impact on the architecture, including overall appearance and interior spaces. This meant keeping overall height of the building unchanged and minimizing loss of floor space to columns. The floor system ultimately allowed for floor to floor heights to remain unchanged with a floor depth of 25" including slab and beams. Column layout also kept impact on the interior spaces to a minimum. Column layouts of the office wing, both original and new, can be seen in the ground floor plan below with the original steel columns shown in red and the new concrete columns shown in blue.

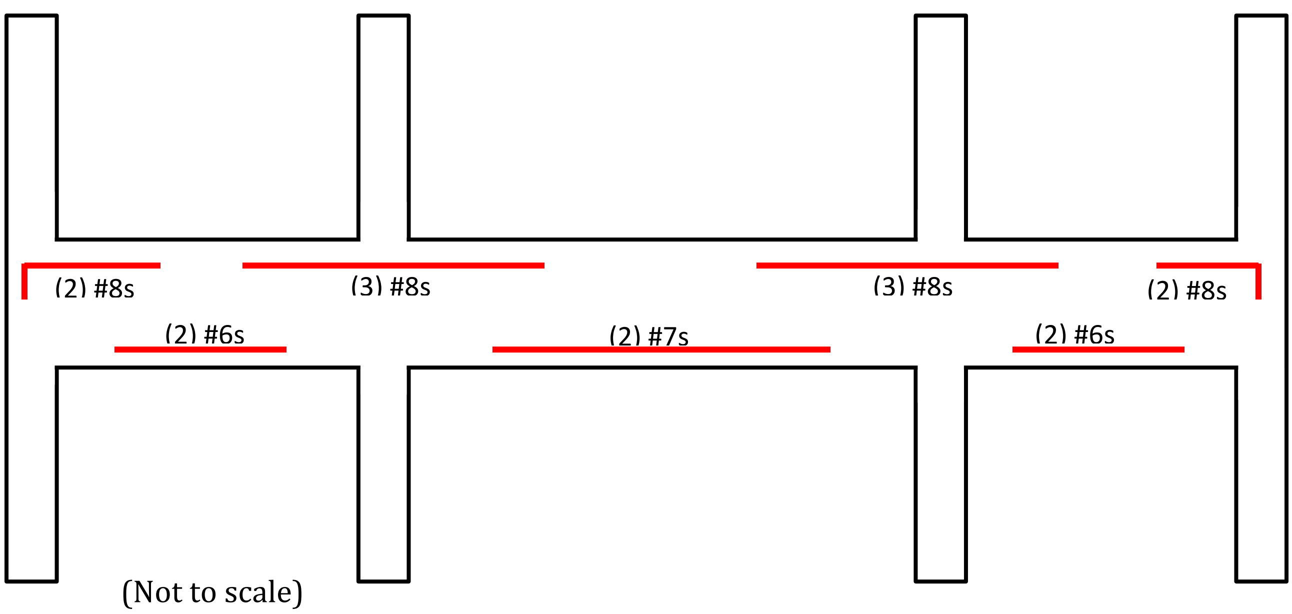


Gravity Redesign: One-way Joist Floor System

The gravity system for the new office wing was designed as a one-way pan joist system with 5" floor slab constructed integrally. With this floor system being along the same depth as the steel system floor-to-floor heights remained unchanged. The spanning capabilities of the pan joist system also allowed column layout to remain for the most part unchanged. The maximum span seen by the pan joists is 36' center-to-center column distance in the middle bay. Pan joists have pan depth of 20", pan width of 66", and rib width of 10". This sizing was based off of design guides found in *Concrete Floor Systems: Guide to Estimating and Economizing.*

The slab was designed for a fire rating of 2 hours as was the current floor system. This controlled the design thickness surpassing that needed to meet deflection requirements. Reinforcement for the slab resulted in #4s @8" o.c. for flexure and #4s @ 18" o.c. for shrinkage and temperature, both at midspan of the slab. The hand calculations can be found beginning on page 40 in Appendix B.

Design of the pan joists was only done for the 26' span and the 36' span. With joist layout being repetitive throughout the office wing only these two spans needed to be considered with all smaller or less loaded spans designed to match one of these spans. A layout for the reinforcement of these joists in a typical frame line can be seen below. All hand calculations of reinforcement design can be found starting on page 44 in Appendix B.



Lateral System: Ordinary Concrete Moment Frames

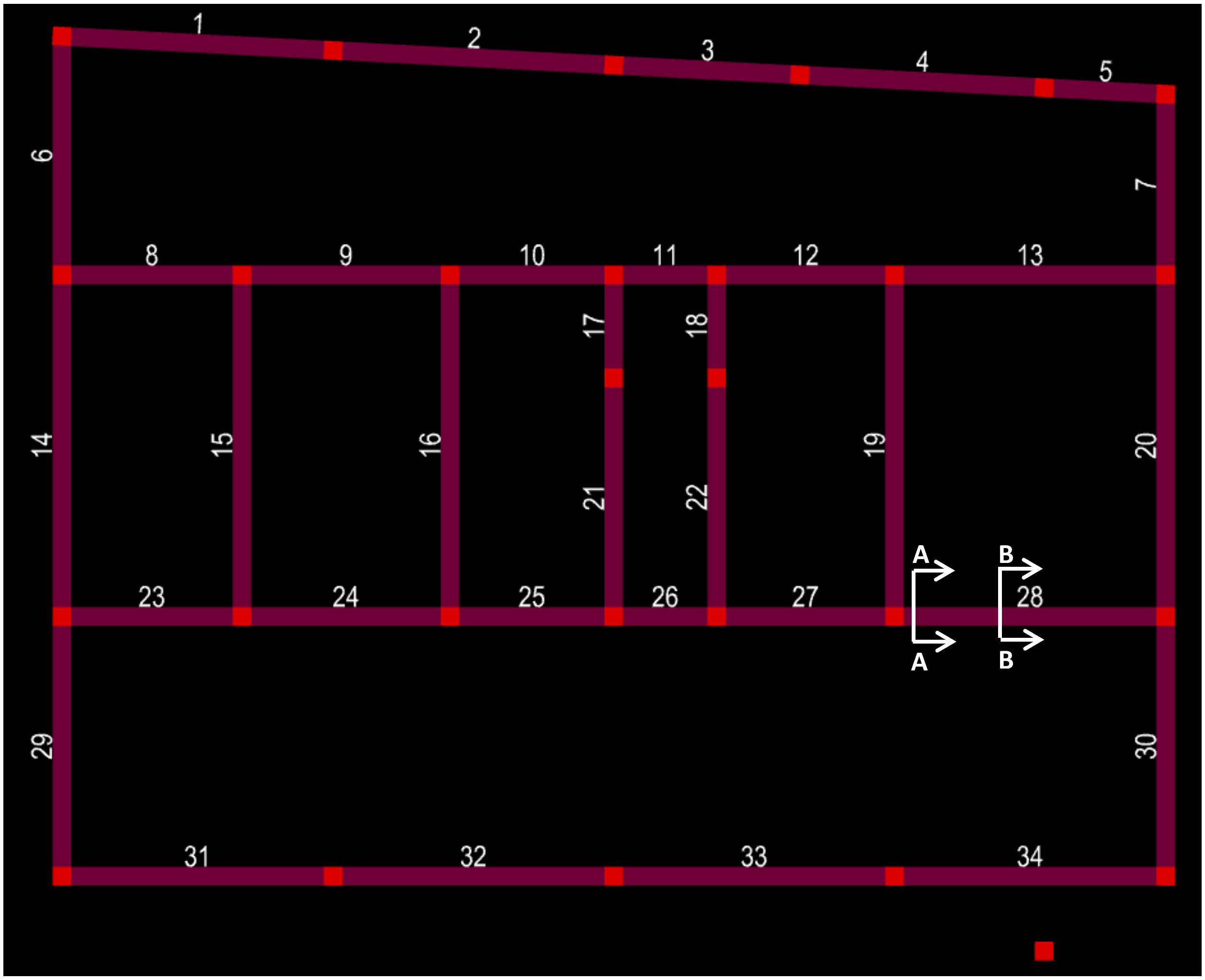
The lateral force resisting system in the redesigned office wing consisted of moment frames with beams and columns in both directions of analysis. ACI318-11 outlines requirements for seismic design that must be met based on the Seismic Design Category. Since the office wing fell into category B its lateral system was to be designed as Ordinary Moment Frames. This described the severity of the design, which according to the code was that outlined in chapters 1 through 19 with the addition of having 2 bars of reinforcing steel continuous along both the top and bottom of each frame, as described in section 21.2.2.

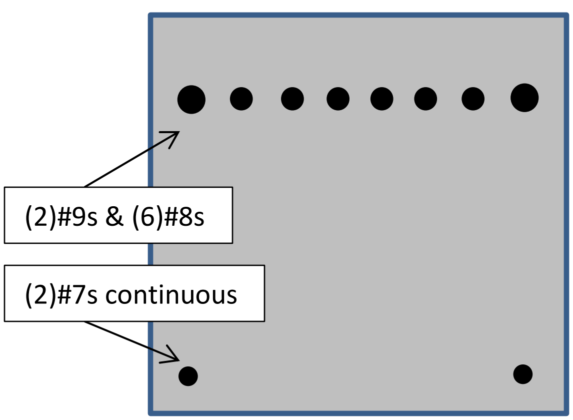
Since the loading on each beam varied, designing each member could result in different reinforcement and reduce the overall amount of steel in the redesign but would take a good amount of time. To save time, loading for each member was analyzed and a design of the member experiencing the largest forces on each floor was done. The floor layout for the second floor was analyzed for live and dead loads since it has more floor area than floors 3-5. These gravity loads could then be conservatively assumed the gravity loading for all floors. The resulting moments were then factored into load combination 5 along with the moments obtained from the ETABS lateral model for the worst case scenario, seismic case 13 for loading in the N-S direction and seismic case 10 for loading in the E-W direction. These moments were the ultimate moments Mu used in determining flexural reinforcement. A list of design moments for each member can be seen on page 51 of Appendix C.

The reinforcement was calculated for the beams numbered in the image on the following page. Based on these results it was found that all but 2 beams' reinforcement (beams 13 & 28) were controlled by $A_{s,\,min}$ at midspan whereas design moments controlled the amount of reinforcement at column faces.

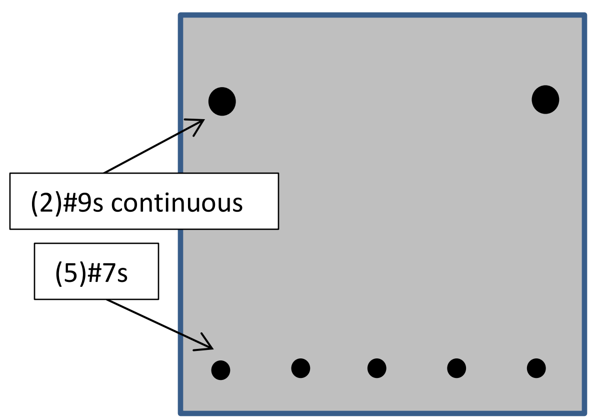
Beams 13 & 28 were the most severely loaded mainly due to their span and the fact that they supported pan joists on both sides. Reinforcement for these members is detailed on the bottom of the next page.

A complete list of the flexural reinforcing for beams 1 through 34 can be found on page 52 of Appendix C. This reinforcement was designed for the worst case loading and therefore in the interest of saving time, can be duplicated on all floors while insuring strength requirements. In each member the outermost bars in both the top and bottom layer of reinforcement were required to be continuous the entire span as noted in section 21.2.2 of ACI318-11.





A-A: Column face reinforcement



B-B: Midspan reinforcement

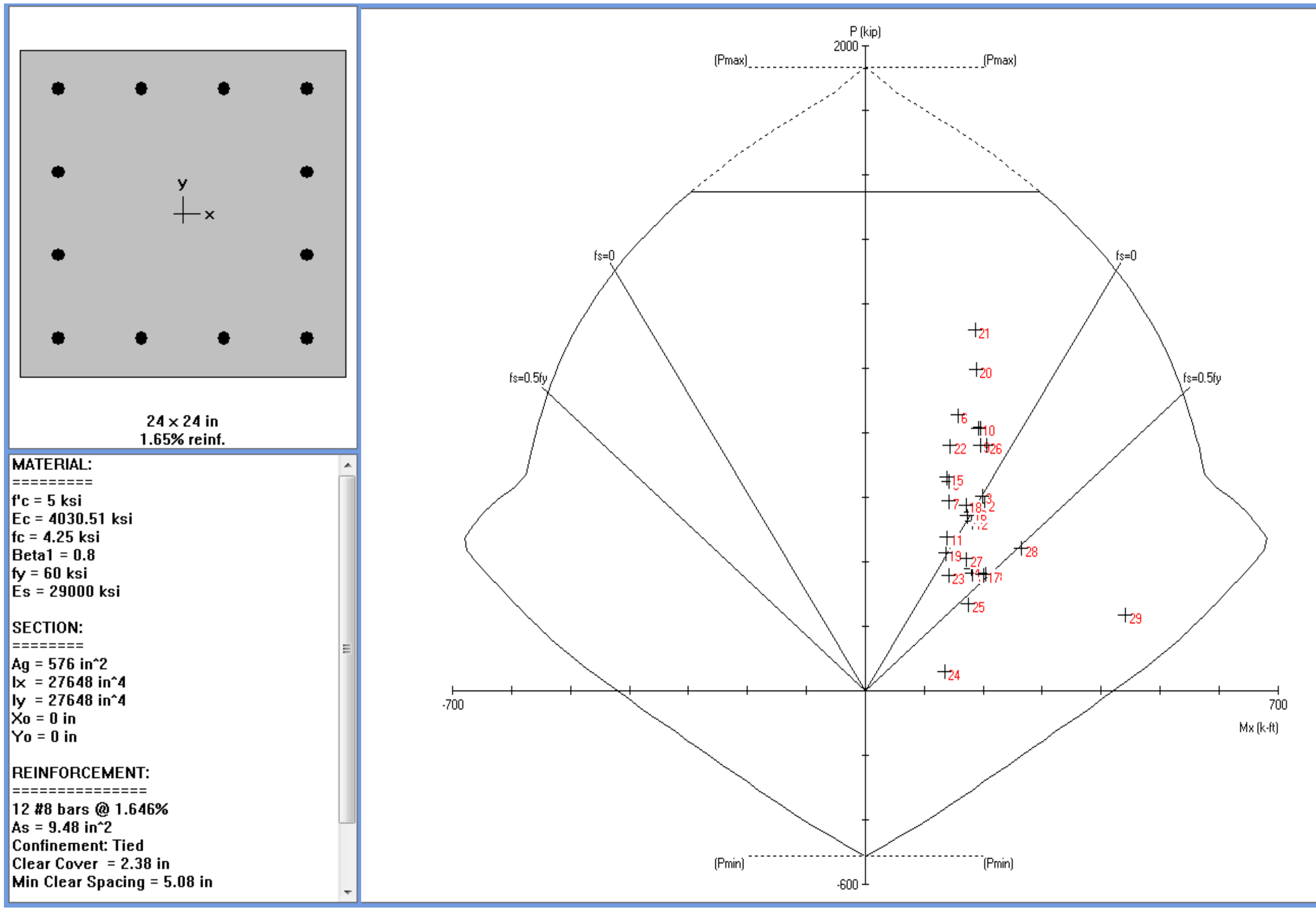
Column Design

The design of the columns required slightly more consideration when designing then beams and joists. Columns needed to resist both axial and flexural forces and were loaded along both axes of analysis. This is typically taken into account by designing the reinforcement symmetrically for the worst case scenario.

A RAM model of the office wing was also made in addition to the ETABS model previously mentioned. This was done mainly for educational purposes to further knowledge in creating models using another software program. This model also proved helpful in gathering load combination results for axial and bending forces in the columns as well as designing the foundations later in this report.

To save time, all columns were designed the same to resist the worst case loading combinations using spColumn. The column dimensions and reinforcing layout were based off the largest pier dimension of 28"x28" with (12)#6 bars. However design already indicated 24"x24" columns so this size was chosen instead. ACI318-11 also indicates A_S must fall between $0.01A_g$ and $0.08A_g$, because of this the reinforcement was increased to (12)#8bars, with A_S = 1.65%. The interaction diagram on the following page shows the capacity of this column's design with the ground floor columns plotted. These forces fall within the range of the column's capacity and the 24"x24" column with (12)#8s can therefore be used throughout the entire building since loading will only decrease on higher floors. It should be noted that small columns with less reinforcing steel will most likely be possible on higher levels but because of time this more efficient option was not pursued.

Ground Floor Column Design Forces			
Reference #	Column	Pu (kips)	Mu (k-ft)
1	A1	365	182
2	A2	582	203
3	A4	605	198
4	A5	380	179
5	B2	817	196
6	B4	855	157
7	C1.2	588	142
8	C5	650	142
9	D2	763	195
10	D4	816	192
11	E1.2	477	139
12	E2	526	181
13	E3	364	204
14	E4	543	173
15	E5	662	139
16	F2	554	179
17	F3	361	201
18	F4	577	171
19	G1.2	428	136
20	H2	997	188
21	H4	1120	186
22	H5	761	144
23	J1.2	359	141
24	J6	61	134
25	L1.2	269	175
26	L4	762	205
27	L5	409	172
28	K2	443	265
29	M2	236	441



Special Case Beam

One area of particular concern was the distribution of loads to the foundation at column line L-2. The added weight of concrete versus steel needed to be resolved in this redesign. The original steel structure had a cantilevered beam distributing the load to the column at K-2. This was redesigned so the column loads transferred into a beam spanning between K-2 and M-2. The walkway below this column line did not allow for a ground floor column to direct the load from upper levels into the foundations, so a beam was added with supports moved to the exterior wall at K-2 and the architectural column at M-2 was made into a structural column. This beam spanned 11 ft and has a concentrated load from the columns 5 floors above of around 600kips. This produces both moments and shears far greater than the loads experienced by any other beam in the office wing. For this reason this beam needed to be designed separately. The simplest option for increasing the beams capacity was to increase the depth so the beam was designed with a new depth of 36". Design calculations for this beam can be found starting on page 53 of Appendix D.

Foundation Impact

With the change to a heavier concrete structure along with now being isolated from the rest of University Academic Center, the new office wing's foundations saw a new combination of loads. Because of this, a new foundation was designed in RAM Foundation. All footings were modeled as spread footings to compare to the current foundation. As assumed, the footings increased in size to resist the higher loads of the concrete structure. A plan view of the new foundation and an overall footing summary can be seen on the following page.

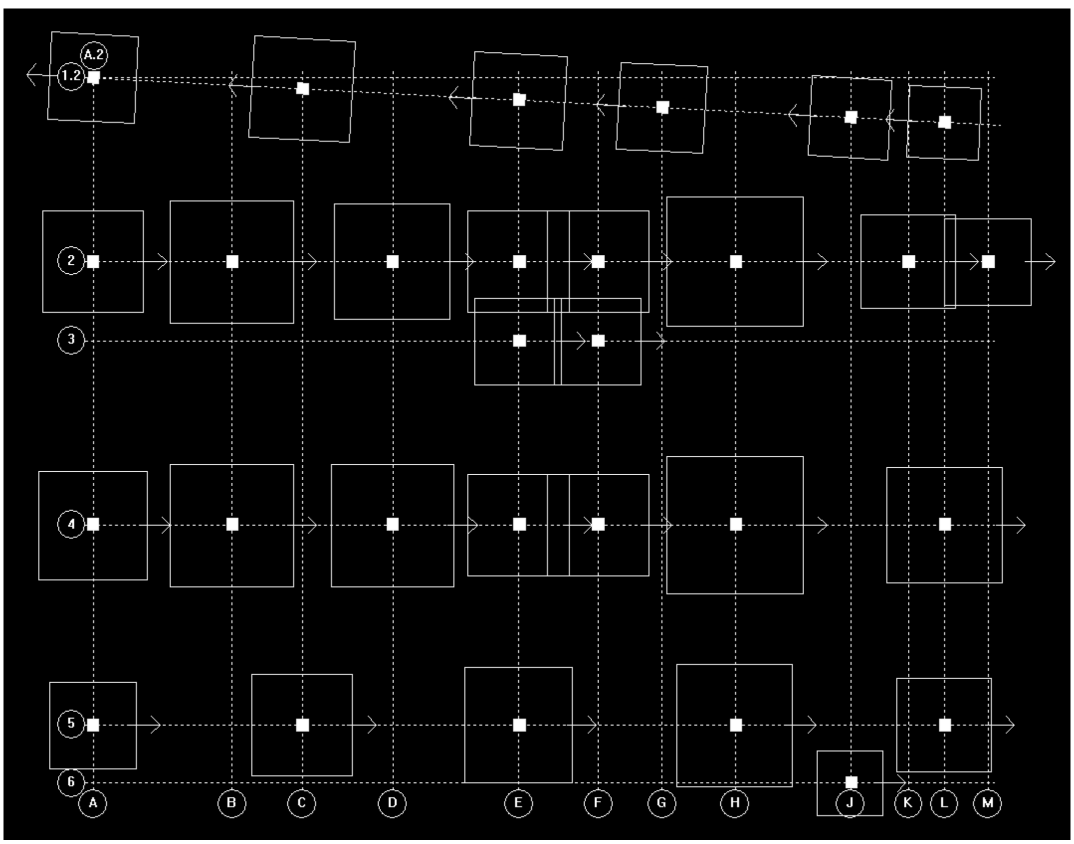
With the exception of a few locations spread footings were sufficient to resist loading without overlapping, and all footings maintained a maximum depth of 3 ft. Areas of overlap included all footings under the stairwell, and footings located at (K-2),(M-2). These areas would have to be redesigned as combined footings; however these calculations were not done due to time constraints. Spot checks were done for shear and reinforcement to validate the results of RAM Foundation for column (H-4), the highest loaded column. These hand calculations can be found on beginning on page 56 in Appendix E along with RAM output for footing (H-4).

Spread Footing Design Summary

RAM Foundation v14.05.01.00
DataBase: office wing
Building Code: IBC
Date: 04/02/13 11:05:19
Design Code: ACI318-08

Grid	Orientation Col/Foot	Dimensions (ft) Length	Width	Thick	f'c/fy ksi	Bottom Reinforcement Parallel to Length	Parallel to Width	Top Reinforcement Parallel to Length	Parallel to Width
(A - 5)	0.00/ 0.00	12.00	12.00	1.50	4.50/60.00	12-#7	13-#7	None	None
(A - 4)	0.00/ 0.00	15.00	15.00	2.00	4.50/60.00	17-#7	17-#7	None	None
(A - 2)	0.00/ 0.00	14.00	14.00	2.00	4.50/60.00	15-#7	15-#7	None	None
(0.00 - 0.00)	177.00/177.00	12.00	12.00	1.50	4.50/60.00	11-#7	12-#7	None	None
(B - 4)	0.00/ 0.00	17.00	17.00	2.50	4.50/60.00	21-#7	22-#7	None	None
(B - 2)	0.00/ 0.00	17.00	17.00	2.50	4.50/60.00	15-#8	16-#8	None	None
(C - 5)	0.00/ 0.00	14.00	14.00	2.00	4.50/60.00	12-#8	13-#8	None	None
(29.00 - -1.52)	177.00/177.00	14.00	14.00	2.00	4.50/60.00	14-#7	15-#7	None	None
(D - 4)	0.00/ 0.00	17.00	17.00	2.50	4.50/60.00	15-#8	16-#8	None	None
(D - 2)	0.00/ 0.00	16.00	16.00	2.50	4.50/60.00	18-#7	18-#7	None	None
(E - 5)	0.00/ 0.00	15.00	16.00	2.00	4.50/60.00	19-#7(17)	19-#7	None	None
(E - 4)	0.00/ 0.00	14.00	14.00	2.00	4.50/60.00	10-#8	11-#8	None	None
(E - 3)	0.00/ 0.00	12.00	12.00	1.50	4.50/60.00	10-#7	13-#7	9-#3	9-#3
(E - 2)	0.00/ 0.00	14.00	14.00	2.00	4.50/60.00	13-#7	14-#7	None	None
(59.00 - -3.09)	177.00/177.00	13.00	13.00	2.00	4.50/60.00	9-#8	9-#8	None	None
(F - 4)	0.00/ 0.00	14.00	14.00	2.00	4.50/60.00	14-#7	15-#7	None	None
(F - 3)	0.00/ 0.00	12.00	12.00	1.50	4.50/60.00	10-#7	13-#7	9-#3	9-#3
(F - 2)	0.00/ 0.00	14.00	14.00	2.00	4.50/60.00	10-#8	11-#8	None	None
(78.89 - -4.13)	177.00/177.00	12.00	12.00	1.50	4.50/60.00	12-#7	13-#7	None	None
(H - 5)	0.00/ 0.00	16.00	17.00	2.50	4.50/60.00	14-#8(12)	14-#8	None	None
(H - 4)	0.00/ 0.00	19.00	19.00	3.00	4.50/60.00	26-#7	26-#7	None	None
(H - 2)	0.00/ 0.00	19.00	18.00	2.50	4.50/60.00	21-#8	21-#8(19)	None	None
(J - 6)	0.00/ 0.00	9.00	9.00	1.50	4.50/60.00	9-#5	9-#5	7-#3	7-#3
(105.00 - -5.50)	177.00/177.00	11.00	11.00	1.50	4.50/60.00	10-#7	10-#7	None	None
(K - 2)	0.00/ 0.00	13.00	13.00	2.00	4.50/60.00	9-#8	9-#8	None	None
(L - 5)	0.00/ 0.00	13.00	13.00	1.50	4.50/60.00	14-#7	14-#7	None	None
(L - 4)	0.00/ 0.00	16.00	16.00	2.50	4.50/60.00	18-#7	18-#7	None	None
(118.00 - -6.18)	177.00/177.00	10.00	10.00	1.50	4.50/60.00	9-#6	10-#6	8-#3	8-#3
(M - 2)	0.00/ 0.00	12.00	12.00	1.50	4.50/60.00	10-#7	9-#7	None	None

* - Number between () in reinforcement is quantity of bars in center strip of rectangular footing



Construction Breadth

With the design of the office wing in concrete complete, now the question to be asked was whether or not it would be practical to build. That was where an investigation into the construction management and estimating aspects of the design became important.

A cost breakdown of all structural items that changed between designs was estimated to determine feasibility around total cost. The values for this were taken from *RSMeans Building Construction Cost Data 2012*. A table of the specific data used in the estimates exactly as they appeared in the book is shown on page 59 of Appendix F. RSMeans allowed for the determining of cost broken down by cost of materials, labor, equipment, and factored in overhead and profit as well. Of course estimating is never exact with many variables to consider but RSMeans provided a nationally gathered source of knowledge on the construction process and allowed for a reasonable comparison in the cost of each building system.

Since no cost data was given on the original structure pricing had to be created for both systems. This was also necessary in order to accurately compare the two costs. The new concrete design was estimated at a cost of $1,552,739 including overhead and profit, while the original system came to a total of $1,914,708. It appeared that changing the office wing to a concrete structure would reduce the cost. However before recommending a change, the scheduling impact must be considered as well. Shown below is the cost breakdown for each structural system.

New Office Wing Design Costs					
	Material	Labor	Equipment	Total	Total with O&P
Formwork	$172,235.55	$407,588.51	$0.00	$579,824.06	$815,942.64
Rebar	$153,558.67	$108,194.27	$0.00	$261,752.94	$342,390.60
Concrete	$252,822.92	$53,140.34	$15,985.59	$321,948.85	$376,821.90
Finishing	$0.00	$11,722.32	$0.00	$11,722.32	$17,583.48
Total	$578,617.14	$580,645.43	$15,985.59	$1,175,248.16	$1,552,738.62

Original Office Wing Design Costs					
	Material	Labor	Equipment	Total	Total with O&P
Formwork	$1,670.70	$8,703.78	$0.00	$10,374.48	$15,224.89
Reinforcing	$24,621.93	$19,828.88	$0.00	$44,450.81	$58,945.93
Concrete	$146,751.02	$18,422.33	$5,011.77	$170,185.12	$194,658.14
Finishing	$0.00	$11,722.32	$0.00	$11,722.32	$17,583.48
Shear Studs	$4,189.50	$6,247.50	$3,013.50	$13,450.50	$19,110.00
Steel Framing	$1,010,429.31	$173,036.06	$49,631.94	$1,233,097.31	$1,467,798.24
Metal Deck	$1,511.39	$21,970.86	$1,608.43	$114,473.37	$141,387.47
Total	$1,189,173.85	$259,931.72	$59,265.64	$1,597,753.90	$1,914,708.14

Once again by using RSMeans, durations for construction were estimated. These durations were then input into Microsoft Project to help in building a basic schedule for each structure. The schedule tasks were created by grouping similar processes in the construction sequence and adding up their durations. These tasks were then arranged so tasks relying on the completion of other tasks would not precede them. The start date was estimated for early 2006 based on the known project duration in order to minimize the time working in freezing temperatures.

Based on the durations calculated from RSMeans and the schedules constructed in Microsoft Project, the concrete system will take approximately 337 days, while the steel system will take 107 days. The steel structure holds the advantage as far as time management is concerned. The full schedules can be seen on page 60 of Appendix F.

The cost advantage to concrete then seemed less believable due to the extra time of construction. Being almost a year to complete, construction of the concrete structure would potentially add extra costs to those calculated through RSMeans like space heating, snow removal, and concrete curing techniques. Because of the large difference in construction times and the potential added costs of year round construction, the decision to design a steel structure seems like the correct choice.

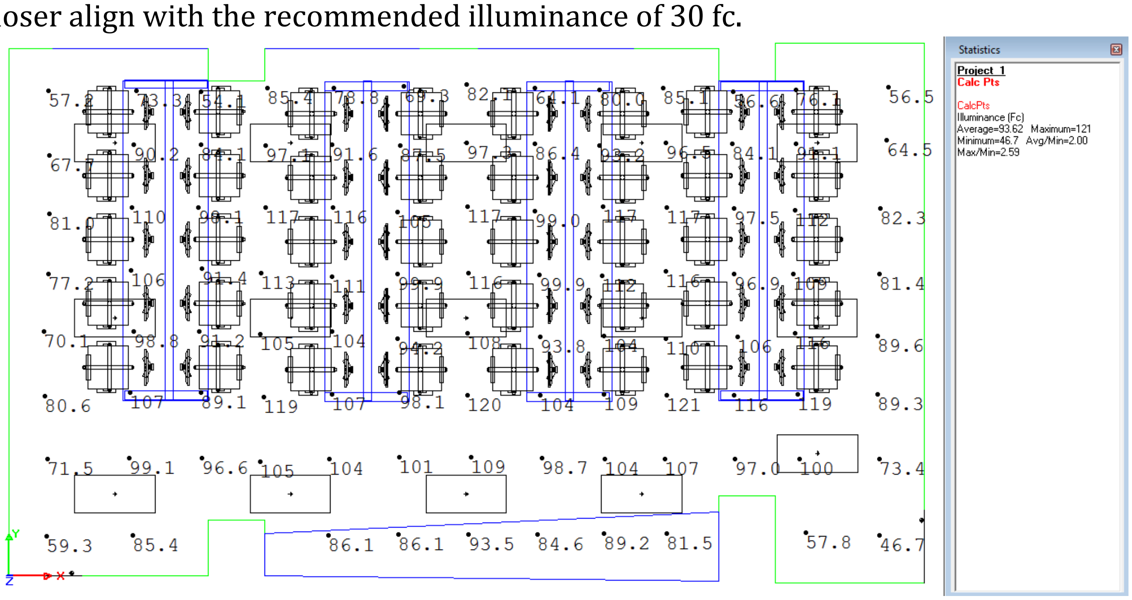
Lighting Breadth

As and added area of study for this thesis a lighting breadth was chosen to investigate the option of changing the luminaires in several computer lab spaces on the 2nd floor of the office wing from the original recessed direct lighting to a pendant direct-indirect lighting option. The space used in calculations for this redesign is labeled Room 2139 on the building plan shown on page 61 of Appendix G.

In order to gain a better understanding into the lighting needs of this type of space the first step in this process involved research into lighting design of interior spaces. This was done by consulting *IES Handbook, 10th Edition.* Table 24.2 out of *IES Handbook, 10th Edition* was used to assign the space a task based on its use; a task of READING AND WRITING – CSA/ISO Type I and II, positive polarity was chosen. This then gave the recommended target illuminance for the basis of design of 300 lux or 30 footcandles at a workplane of 2.5 ft, the typical height of a desk.

The IES Handbook provided additional reasons in support of a direct-indirect lighting scheme. The concept of ambient versus task lighting would better be accomplished by indirect lighting illuminating the space while direct lighting would still illuminate the task. The current system lacked the softer more evenly distributed ambient light given off by indirect lighting. Another reason to shy away from direct lighting in computer lab spaces was the potential for glare, with indirect lighting glare was less of an issue.

After the basis of design of 30 footcandles was determined an analysis of the space as designed was done in AGi32 to calculate lighting capabilities of the current system. The space was modeled as shown below with the luminaire data taken from Columbia Lighting as listed in the electrical plans and reflectances of 80/70/20 chosen for ceiling, walls, and floor. This design resulted in an average illuminance of 93.62 fc and minimum value of 46.7 fc. This seems high given the needs of the space. The space should therefore be redesigned to closer align with the recommended illuminance of 30 fc.

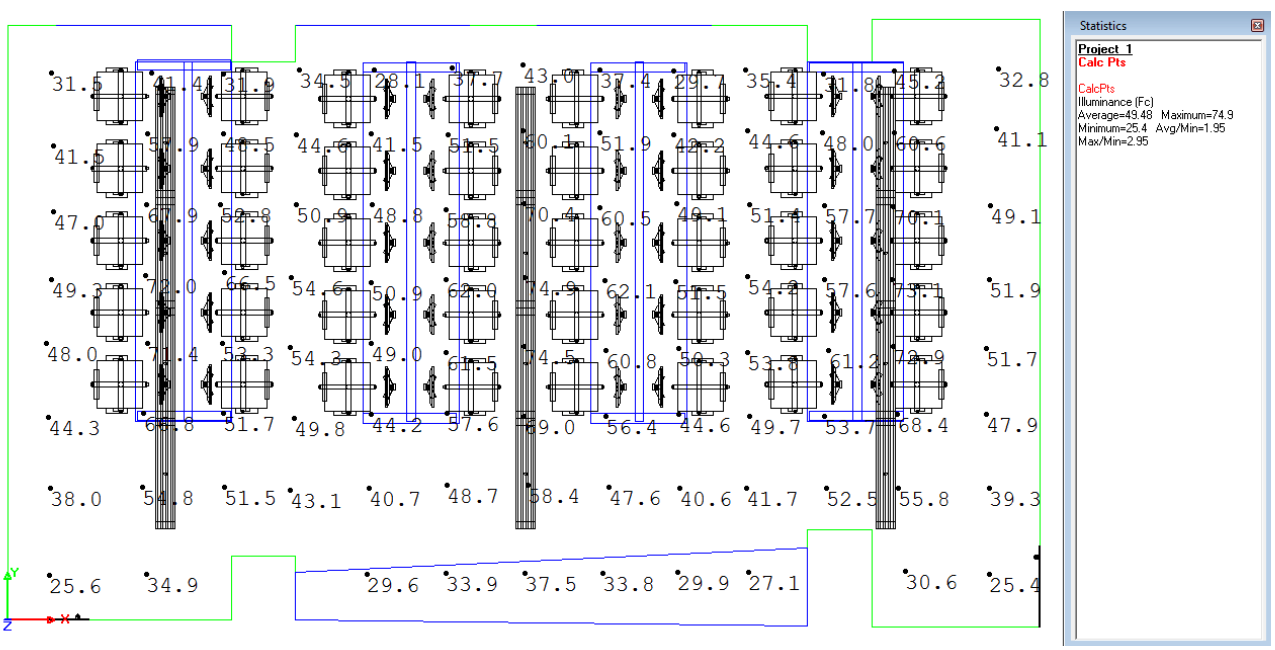


Knowing from the analysis of the current space there is room for improvement, a new luminaire layout was created using a direct-indirect pendant style luminaire. This new luminaire, FINELITE Series 12-ID, supplied both direct (26%) and indirect (74%) lighting. This provided ambient lighting as well as task specific lighting.

A preliminary hand calculation was done using the Zonal Cavity Method as described in *Pearson Construction Technology: Penn State-AE 311 Fundamentals of Building Electrical and Illumination Systems* to determine the number of fixtures required to reach the target illuminance. This process ultimately resulted in a 3x4 layout with 12 fixtures providing an estimated 28.8 fc. The calculations are outlined on page 64 of Appendix G.

This layout was then input into the AGi32 model and recalculated for the new average illuminance; results for this are shown below. It should be noted the pendant fixtures are mounted at a height of 9 ft and the recessed ceiling was raised to 11 ft as opposed to the original 10 ft ceiling height. This layout resulted in an average illuminance of 49.5 fc and a minimum value of 25.4 fc. This option was still somewhat overdesigned but much closer than the original design to the target 30 fc.

The new layout reduced the number of fixtures which, even though the new fixtures used 91W versus the original 90W, meant a decrease in the power consumption. The original design used 1.29 watts/ft^2 whereas the new design used 1.04 watts/ft^2. The decrease in power consumption as well as the added visual benefits of indirect lighting make this new design much more suitable for computer lab spaces and would be recommended as a possible refit in the future.



Conclusion

After a semester's work with this building much was learned about structural analysis and different structural systems. While the previous semester offered the ability to analyze a steel structure and the limitations building code places on it, this semester offered experience in the design of a concrete structure and code limitations mostly focused around ACI318-11.

In the beginning of this process of redesign, the office wing presented itself as the best choice for a concrete structure. Concerning constructability, it had relatively repeatable floor plans which could save on formwork costs. This also made reinforcing layouts more uniform throughout since each floor saw similar loading. When considering architecture, the floor plan of the office wing was also compatible with a concrete redesign where the new column locations did not interfere drastically with any of the spaces.

Overall this redesign consisted of a one-way pan joist floor system with an ordinary moment frame system to resist lateral forces. Joists and beams were designed 20" thick cast integrally with the 5" slab, totaling a 25" overall depth. This floor system was repeated on all floors and roof for sake of time. Columns were also all designed the same with a 24"x24" section and (12)#8 vertical bars as reinforcement. Together these members resisted the calculated wind and seismic loading with seismic controlling most of the design. All concrete used on for this redesign was 5000psi except for the foundations which kept the 4500psi noted in the construction documents.

The added weight of concrete versus steel created several issues, one of which was column line L-2 located above the exterior walkway. This was corrected by a 36" deep beam spanning across the walkway that took the load from the columns above into the foundations. Another issue was the foundations themselves. The added weight of concrete increased the demand on the foundations requiring a redesign. This was done using RAM Foundation with spot checks to determine validity of results. Foundation sizes increased but were still reasonably sized so spread footing could still be used effectively.

In addition to the structural depth, two breadth topics were discussed. The construction breadth focused on the cost and scheduling concerns with the redesigned concrete structure. This resulted in the concrete system costing less but construction time being considerably longer than that of the original steel. For that reason the steel system was determined the more preferable design.

The other breadth composed of a lighting redesign of a computer lab located on the 2nd floor of the office wing. Currently using recessed lighting, the option of a pendant indirect lighting design was created as an alternative. This redesign reduced the number of fixtures, which also reduced the power consumption, while maintaining a recommended illuminance value of 30 footcandles.

Appendices

Appendix A: Loading Hand Calculations

Thesis | Final Report | Snow Loading

Flat Roof Snow load (ASCE 7-10)

$P_f = 0.7\, C_e C_t I\, P_g$ (7.3-1)

$P_g = 25$ psf (Figure 7-1)

$C_e = 0.9$ (Table 7-2)

$C_t = 1.0$ (Table 7-3)

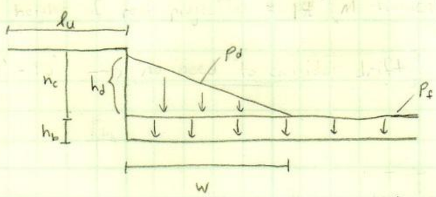
$I_s = 1.0$ (Table 1.5-2)

$P_f = 0.7(0.9)(1)(1)(25)$

$\boxed{P_f = 15.75 \text{ psf}}$

$\rightarrow$ $\boxed{\text{USE } 20 \text{ psf}}$

Snow Drift Considerations (ASCE 7-10)



(Figure 7-8)

$l_u \cong 50$ ft

(Figure 7-9)

$h_d = 2.33 \text{ ft}\left(\frac{3}{4}\right) = 1.75 \text{ ft}$

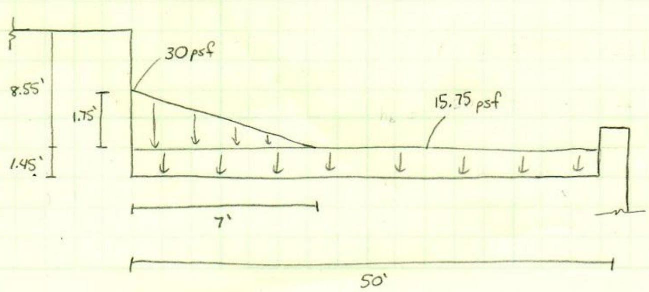
$\gamma = 0.13(25) + 14 = 17.25$ pcf (7.7-1)

$h_b = \frac{P_f}{\gamma} = \frac{25}{17.25} = 1.45$ ft

$h_c = 10 \text{ ft} - 1.45 \text{ ft} = 8.55$ ft

$w = 4 h_d = 4(1.75) = 7$ ft

$P_d = h_d \gamma = 30$ psf



Thesis | Final Report | Wind Loading

ASCE 7-10 Directional Procedure

1. Risk Category II (Construction Documents)

2. Basic Wind Speed $(V) = 115$ mph

3. Wind directionality factor $(K_d) = 0.85$ (Table 26.6.1)

Exposure Category C (Construction Documents)

Topographic factor $(K_{zt}) = 1$ (Assumed)

Gust Effect factor $(G) = 0.85$

Enclosed Building

Internal pressure coefficient $(GC_{pi}) = \pm 0.18$ (Table 26.11-1)

4. Velocity pressure exposure coefficients $(K_h) = 1.178$

(K_z) @ $\begin{cases} 72' = 1.178 \\ 58' = 1.122 \\ 44' = 1.06 \\ 30' = 0.98 \\ 16' = 0.86 \end{cases}$

5. Velocity pressure $(q_z) = 0.00256\, K_z K_{zt} K_d V^2$ (27.3-1)

q_z @ $\begin{cases} 72' = 33.9 \text{ psf} \\ 58' = 32.3 \text{ psf} \\ 44' = 30.5 \text{ psf} \\ 30' = 28.2 \text{ psf} \\ 16' = 24.7 \text{ psf} \end{cases}$

$q_h = 33.9$ psf

6. External pressure coefficient $(C_p) =$ 0.8 (Windward)
0.5 (Leeward)

10. Design wind pressures $(p) = qGC_p - q_i(GC_{pi})$ (27.4-1)

$p_{windward}$ @ $\begin{cases} 72' = 23.05 \text{ psf} \pm 6.1 \text{ psf} \\ 58' = 21.96 \text{ psf} \pm 6.1 \text{ psf} \\ 44' = 20.74 \text{ psf} \pm 6.1 \text{ psf} \\ 30' = 19.18 \text{ psf} \pm 6.1 \text{ psf} \\ 16' = 16.80 \text{ psf} \pm 6.1 \text{ psf} \end{cases}$

$p_{leeward} = 14.41$ psf ± 6.1 psf

Thesis | Final Report | Seismic Loading (Office Wing)

ASCE 7-10 Equivalent Lateral Force Procedure

Seismic Force Resisting System

Ordinary reinforced concrete moment frames $\longrightarrow$ $R = 3$, $\Omega_0 = 3$, $C_d = 2\frac{1}{2}$

Occupancy Category II $\rightarrow$ $I_e = 1.0$ Site Class : D

$S_s = 0.156$
$S_1 = 0.051$

$S_{MS} = 0.250$
$S_{M1} = 0.121$

$S_{DS} = 0.167$
$S_{D1} = 0.081$

[Values taken from usgs.gov]

SDC = B

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.167}{3} = 0.0557$$

$$T = T_a = C_t h_n^x = 0.016(72)^{0.9} = 0.75\ s \qquad [12.8.2.1]$$

$T_L = 8s$ Figure [22-12]

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} \quad \text{for } T \le T_L \qquad C_s = \frac{0.081}{0.75(3)} = 0.036$$

$$C_s = \min \begin{vmatrix} 0.075 \\ 0.036 \end{vmatrix} \longleftarrow \quad C_s = 0.036 > 0.01 \quad \text{OK}$$

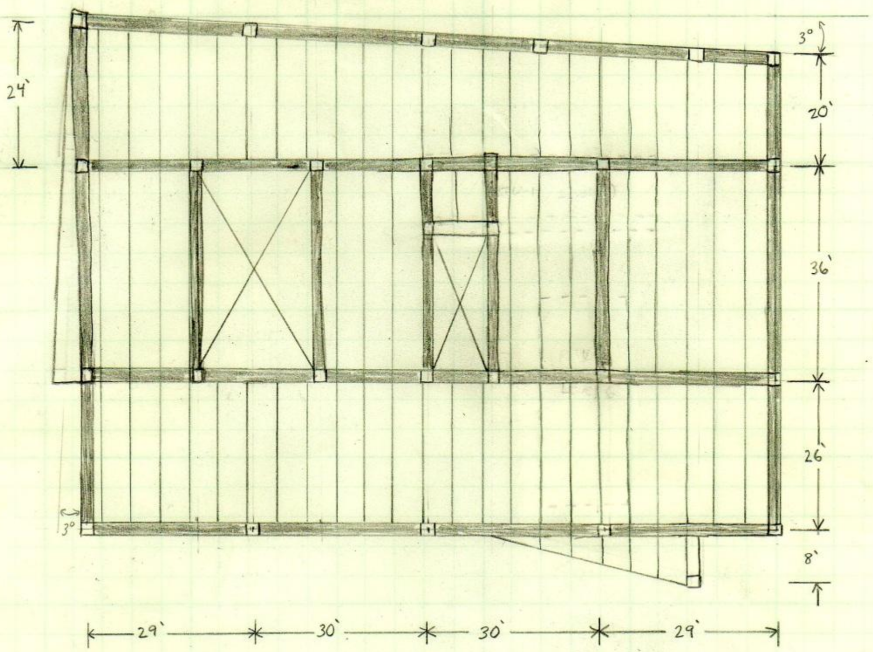
$$V = C_s W$$

$$V = 0.036\ (8973\ \text{kip}) = 323\ \text{kip}$$

$$F_x = C_{vx} V \qquad C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}, \qquad k = 1.125$$

Appendix B: Gravity System Calculations

Thesis | Final Report | Gravity System ; One-way Pan Joists



Levels 4-6

Preliminary Sizes

[approximated from "Concrete Floor Systems - Guide to Estimating and Economizing" – David A. Fanella]

Bay Size = 30' × 36'

66" pan , 20" Pan Depth , 8" Rib Width

42" Beam Width

24" × 24" Column

5" thick slab

Thesis | Final Report | Gravity System: One-way Pan Joists

2hr Fire rating Requirements (IBC 2009)

min Slab thickness : 5 in

min Reinforcing cover in slab : 3/4 in

min Reinforcing cover in beams, columns : 1 1/2 in

Min thickness of one-way Slab (ACI 318-11) Table 9.5(a)

slab: $h = \frac{66''}{28} = 2.4$ in $< 5''$ OK

joists: $h = \left| \begin{array}{l} \frac{36'}{28} = 1.3 \text{ ft} < 20'' \text{ OK} \\ \text{max } \frac{8'}{8} = 1' \text{ft} \end{array} \right.$

Trial Sizes

Slab : 5"

Joists : 20" pan depth, 10" Rib width

Beams: 20" depth, 24" width

Columns : 24" × 24"

Material Properties

$f'_c = 5{,}000$ psi NWC

$f_y = 60$ ksi

Thesis | Final Report | Gravity System : Slab Design

Loading

Superimposed Dead = 20 psf

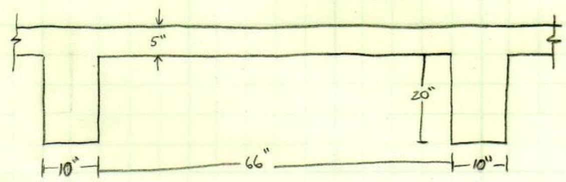
Live : Office = 50 psf + 20 partitions
Corridor = 80 psf ← USE TO BE CONSERVATIVE

Roof = 30 psf

Snow = 20 psf

RTUs = 25,000 lb each

Slab Design



$w_D = \frac{5''}{12''}(150\ pcf) + 20\ psf \times 1\ ft\ strip \cong 82.5\ plf$

$w_L = 80\ psf \times 1\ ft\ strip = 80\ plf$

$w_u = 1.2(82.5) + 1.6(80) = 227\ plf$

Shear Check

$V_u = 1.15\frac{(227)(5.5')}{2} = 718\ lb/ft$ width of slab

$\phi V_c = 0.75(2\lambda\sqrt{f'c}\ b_w d) = 0.75(2)(1)\sqrt{5000}(12'')(2.5'') = 3{,}182\ lb/ft$ width of slab

$d = 2.5''$ (center of slab)

$\phi V_c > V_u$ OK

Thesis | Final Report | Gravity System: Slab Design

Reinforcement

Sample Calc → Exterior Midspan

$M_{coeff} = \frac{1}{14}$

$M_u = \frac{227(5.5)^2}{14} = 490$ lb-ft

$A_s \approxeq \frac{M_u}{4d} = \frac{0.490}{4(2.5)} = 0.049 \text{ in}^2/\text{ft}$ $\qquad a = \frac{0.049(60)}{0.85(5)(12'')} = 0.058$ in

$A_{s,req} \geq \frac{0.490(12)}{0.9(60)(2.5 - \frac{0.058}{2})} = 0.0441 \text{ in}^2$

$A_{s,min} = 0.0018(12'')(5'') = 0.108 \text{ in}^2/\text{ft} \longrightarrow$ use #4 @ 12 in o.c. $\quad A_s = 0.2 \text{ in}^2/\text{ft}$

	Ext Support	Ext Midspan	1st Int Support	Int Midspan	Other Supports
$M_{coeff.}$	$-1/24$	$1/14$	$-1/10$	$1/16$	$-1/11$
M_u (k-ft)	−0.265	0.490	−0.635	0.397	−0.578
$A_{s,req}$ (in²/ft)	0.0237	0.0441	0.0573	0.0356	0.0521
$A_{s,min}$ (in²/ft)	0.108 →				
Reinf.	#4 @ 12" o.c. →				
A_s (in²/ft)	0.2 →				

Transverse steel

$A_{min\ S+T} = 0.0018(12'')(5'') = 0.108 \text{ in}^2/\text{ft}$

Max spacing = min $\begin{cases} 5h = 25'' \\ 18'' \leftarrow \end{cases}$ → use #4 @ 18" o.c.

Spacing

$s \leq 15\left(\frac{40{,}000}{40{,}000}\right) - 2.5(2.5) \qquad f_s = \frac{2}{3}f_y = 40{,}000$

$s \leq 8.75'' \longrightarrow$ use #4 @ 8" o.c.

Slab Details:

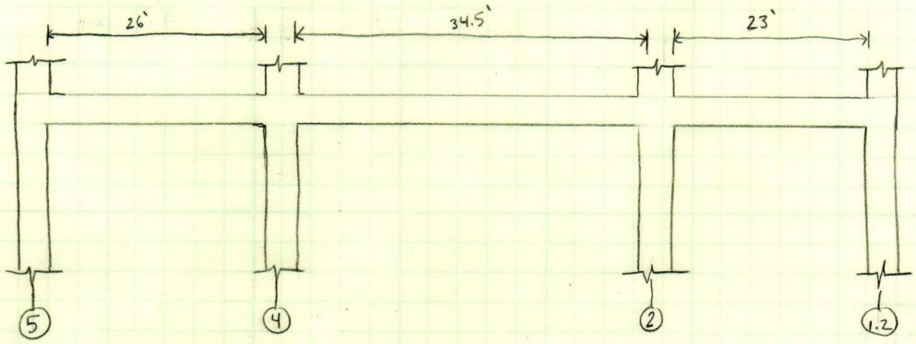
Thickness: 5"

Reinforcement:

#4s @ 8" o.c. (flexural)

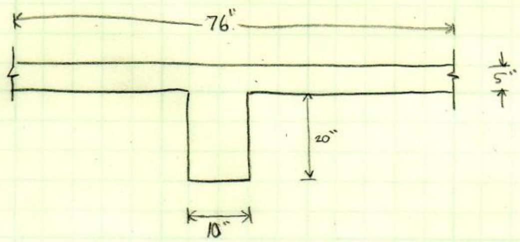
#4s @ 18" o.c. (transverse)

Thesis | Final Report | Gravity System : Joist Design



Pan Joist dimensions :

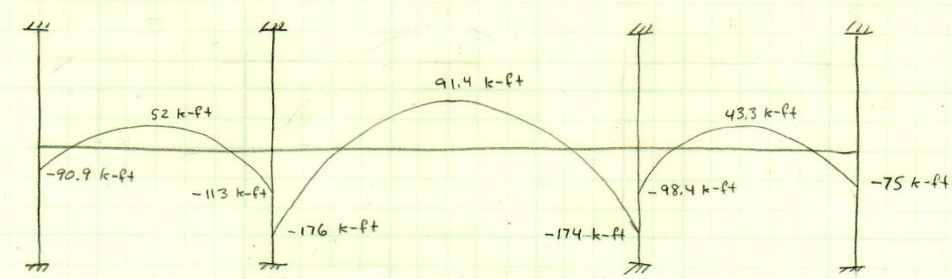
- 20" pan depth
- 10" rib width
- 66" spacing



$$w_D = \frac{150\text{ pcf}(8")(20")}{144"} + \frac{150\text{ pcf}(5")(74")}{144"} + 20\text{ psf}\left(\frac{74"}{12"}\right) = 731\ \text{lb/ft}$$

$$w_L = 80\text{ psf}\left(\frac{74"}{12"}\right) = 507\ \text{lb/ft}$$

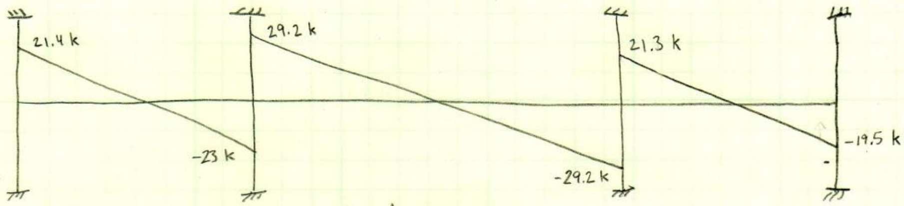
$$w_u = 1.2\,(675) + 1.6\,(507) = 1690\ \text{lb/ft} = 1.69\ \text{k/ft}$$



Design Moments

[Values taken from STAAD.Pro output]

Thesis | Final Report | Gravity System : Joist Design



Design Shears

[Values taken from STAAD.Pro output]

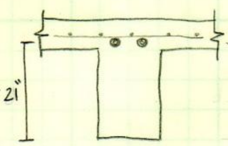
Flexural Reinforcement

- Simplify by designing only a 26' span and 34.5' span
- 26' span design will be used at 23.5' span as well

26' span

$M_u^- = 113$ k-ft

$d = 25'' - 2.5'' - 0.5'' - 1'' = 21''$



$A_s \cong \frac{M_u}{4d} = \frac{113}{4(21)} = 1.35 \text{ in}^2$

$a \cong \frac{1.35(60)}{0.85(5)(10)} = 1.9''$

$A_{s,req} = \frac{113(12)}{0.85(60)(21 - \frac{1.9}{2})} = 1.33 \text{ in}^2$

$a = 1.9''$

$\beta_1 = 0.85$

$c = 1.615''$

$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{1.615}(21 - 1.615)$

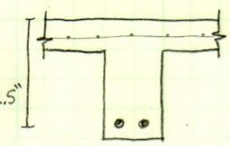
$= 0.028 > 0.005 \rightarrow \phi = 0.9$

Try (2) #8s $\rightarrow A_s = 1.58 \text{ in}^2$

$\phi M_n = \frac{0.9(1.58)(60)(21 - \frac{1.9}{2})}{12} = \boxed{140 \text{ k-ft} > 113 \text{ k-ft}}$ OK

$M_u^+ = 52$ k-ft

$d = 25'' - 1.5'' - 1'' = 22.5''$



$A_s \cong \frac{M_u}{4d} = \frac{52}{4(22.5)} = 0.58 \text{ in}^2$

$a = \frac{0.58(60)}{0.85(5)(10)} = 0.82''$

$A_{s,req} = \frac{52(12)}{0.85(60)(22.5 - \frac{0.82}{2})} = 0.56 \text{ in}^2$

$a = 0.82''$

$\beta_1 = 0.85$

$c = 0.70''$

$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{0.7}(22.5 - 0.70)$

$= 0.075 > 0.005 \rightarrow \phi = 0.9$

$A_{s,min} = \frac{3\sqrt{5000}(8'')(22.5'')}{60000} = 0.64 \text{ in}^2$

Try (2) #6s $\rightarrow A_s = 0.88 \text{ in}^2$

$\phi M_n = \frac{0.9(0.88)(60)(22.5 - \frac{0.82}{2})}{12} = \boxed{87 \text{ k-ft} > 52 \text{ k-ft}}$

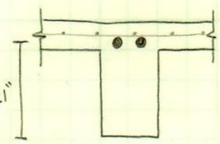
OK

Thesis | Final Report | Gravity System : Joist Design

36' span

$M_u^- = 176$ k-ft

$d = 25" - 2.5" - 0.5" - 1" = 21"$



$A_s \approx \frac{M_u}{4d} = \frac{176}{4(21)} = 2.10 \text{ in}^2$

$a = \frac{2.10(60)}{0.85(5)(10)} = 2.97"$

$A_{s,req} = \frac{176(12)}{0.85(60)(21 - \frac{2.97}{2})} = 2.12 \text{ in}^2$

$a = 2.97"$
$\beta_1 = 0.85$
$c = 2.52"$

$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{2.52}(21 - 2.52)$

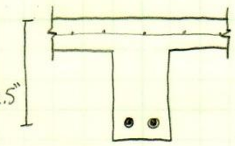
$= 0.017 > 0.005 \rightarrow \phi = 0.9$

Try (3) #8s $\rightarrow A_s = 2.37 \text{ in}^2$

$\phi M_n = \frac{0.9(2.37)(60)(21 - \frac{2.97}{2})}{12} = \boxed{207 \text{ k-ft} > 176 \text{ k-ft OK}}$

$M_u^+ = 91.4$ k-ft

$d = 25" - 1.5" - 1" = 22.5"$



$A_s \approx \frac{M_u}{4d} = \frac{91.4}{4(22.5)} = 1.02 \text{ in}^2$

$a = \frac{1.02(60)}{0.85(5)(10)} = 1.8"$

$A_{s,req} = \frac{91.4(12)}{0.85(60)(22.5 - \frac{1.8}{2})} = 1.00 \text{ in}^2$

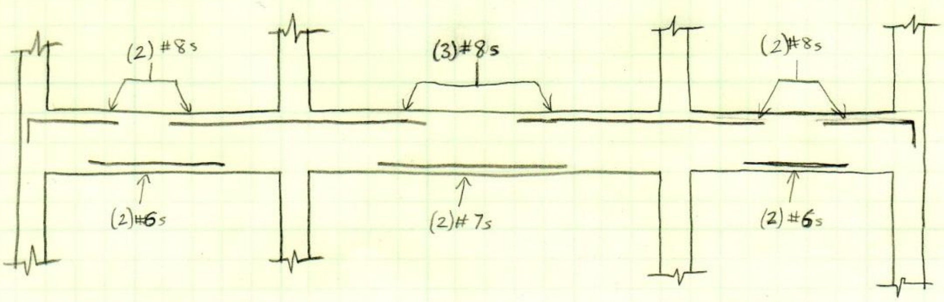
$a = 1.8"$
$\beta_1 = 0.85$
$c = 1.53"$

$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{1.53}(22.5 - 1.53)$

$= 0.041 > 0.005 \rightarrow \phi = 0.9$

Try (2) #7s $\rightarrow A_s = 1.2 \text{ in}^2$

$\phi M_n = \frac{0.9(1.2)(60)(22.5 - \frac{1.8}{2})}{12} = \boxed{116 \text{ k-ft} > 91.4 \text{ k-ft OK}}$



Thesis | Final Report | Gravity System : Joist Design

Shear Reinforcement

26' span

V_u @ $\frac{d}{2}$ from support = 21.4 k

$0.5\ \phi V_c = 0.75(2)\sqrt{5000}\,(10)(21)(0.5) = 11.1\text{ k} < 21.4\text{k} \longrightarrow$ Needs shear reinforcement

$S_{max} = \min \left| \begin{array}{l} \frac{d}{2} = \frac{21}{2} = 10.5'' \leftarrow \text{controls} \\ 24'' \end{array} \right.$

$\frac{14.37\text{ ft}}{23\text{ k}} = \frac{x}{11.1\text{ k}} \longrightarrow x = 6.94\text{ ft}$ (point where shear reinforcement is no longer required)

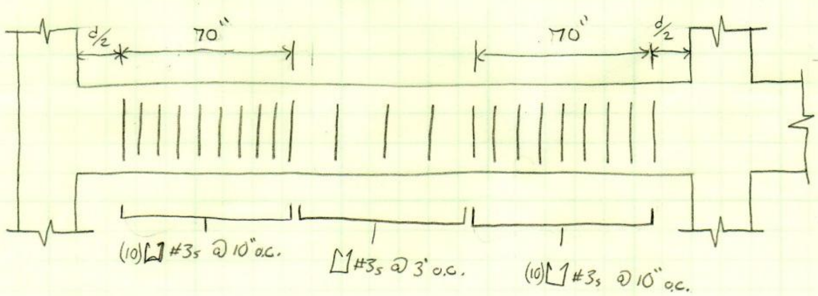
$\frac{A_v}{s} = \frac{(V_u - \phi V_c)}{\phi f_{yt} d} \longrightarrow A_v = \frac{(21.4 - 17.8)(10.5)}{0.75(60)(21)} = 0.04\text{ in}^2$

$A_{v,min} = 0.75\sqrt{f'c}\ \frac{b_w s}{f_{yt}} \longrightarrow A_{v,min} = 0.75\sqrt{5000}\ \frac{(8)(10.5)}{60{,}000} = 0.074\text{ in}^2 \leftarrow$ controls

USE ⊔ #3s $\rightarrow A_v = 0.22\text{in}^2$

$V_s = \frac{A_v f_{yt} d}{s} \longrightarrow V_s = \frac{0.22(60)(21)}{(10.5)} = 26.4\text{ k}$

$\phi V_n = 0.75(V_c + V_s) = 0.75(23.7 + 26.4) = \boxed{37.6\text{ k} > 21.4\text{ k}} = V_u$ OK



USE (10) #3 ⊔s @ 10" o.c. in shear region

USE #3 ⊔s @ 3' o.c. elsewhere

36' span

V_u @ $\frac{d}{2}$ from support $= 27.7$ k

$0.5\,\phi V_c = 0.5(0.75)(2)\sqrt{5000}\,(10)(21) = 11.1\text{ k} < 27.7\text{ k} \longrightarrow$ Needs shear reinforcement

$s_{max} = 10.5''$

$\frac{18.23\text{ ft}}{29.24\text{ k}} = \frac{x}{11.1\text{ k}} \longrightarrow x = 6.92$ ft (point where shear reinforcement is no longer required)

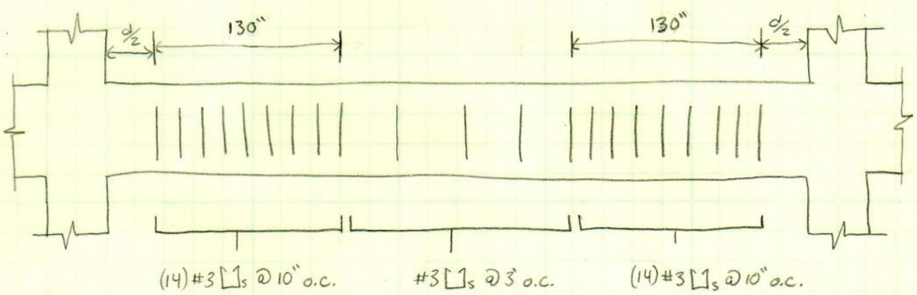
$A_v = \frac{(27.7 - 17.8)(10.5)}{0.75(60)(21)} = 0.11\text{ in}^2 \longleftarrow$ controls

$A_{v,min} = 0.074\text{ in}^2$

USE #3 ⊔ $\longrightarrow A_v = 0.22\text{ in}^2 > 0.11\text{ in}^2$

$V_s = \frac{0.22(60)(21)}{(10.5)} = 26.4$ k

$\phi V_n = \boxed{37.6\text{ k} > 27.7\text{ k}}$ OK



USE (14) #3 ⊔s @ 10" o.c. in shear region

USE #3 ⊔s @ 3' o.c. elsewhere

Appendix C: Lateral System Calculations

Thesis | Final Report | Lateral System: Beam Design

Loading

Dead: beam self-weight = $\frac{150(24")(20")}{144"} = 500$ plf

slab self-weight = $\frac{150(5")}{12"} = 62.5$ psf

pan joist self-weight = $\frac{150(8")(20")}{144"} \frac{12}{74"} = 27$ psf

Superimposed = 20 psf

facade = 40 psf (14') = 560 plf

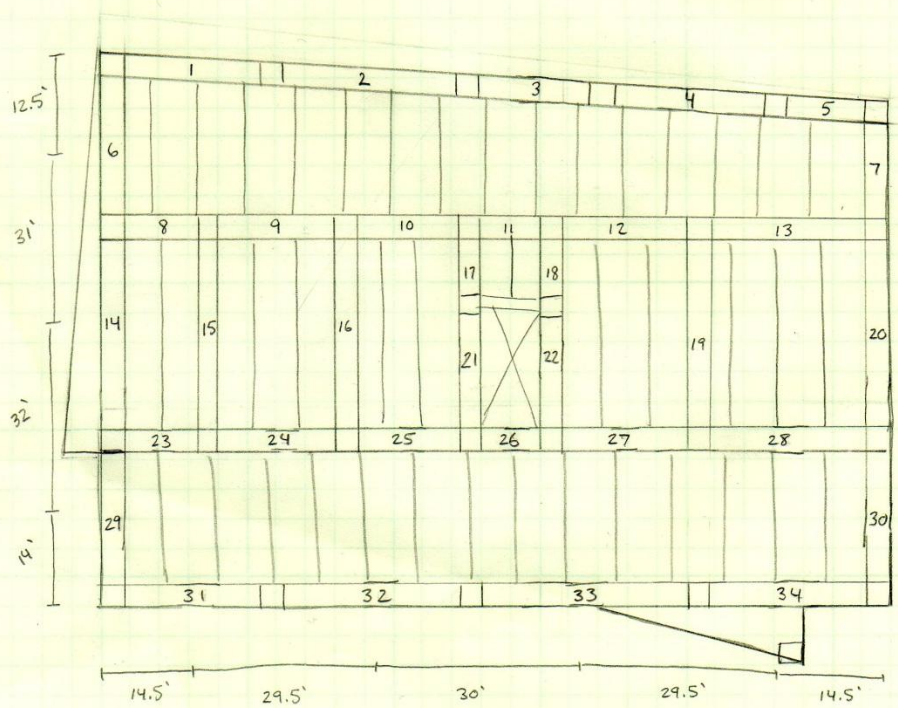
Live: 80 psf

Roof: 30 psf

Snow: 20 psf

Wind: Variable (ETABS Output)

Seismic: Variable (ETABS Output)



Thesis | Final Report | Lateral System: Beam Design

Loading [Numbers reference members on previous page floor plan]

1. $w_D = (20 + 27 + 62.5)\,12.5' + 500 + 560 = 2430$ lb/ft
   $w_L = 80(12.5) = 1000$ lb/ft

2, 3, 4, 5 assumed same as 1

6. $w_D = 3.5'(20 + 62.5) + 500 + 560 = 1350$ lb/ft
   $w_L = 3.5'(80) = 280$ lb/ft

7, 14, 20, 29, 30 assumed same as 6

8. $w_D = 31'(20 + 27 + 62.5) + 500 = 3900$ lb/ft
   $w_L = 80(31') = 2480$ lb/ft

9, 10, 12, 13 assumed same as 8

11. $w_D = 19'(20 + 27 + 62.5) + 500 = 2080$ lb/ft
    w_L $19'(80) = 1520$ lb/ft

15. $w_D = 5'(20 + 62.5) + 500 = 920$ lb/ft
    $w_L = 5'(80) = 640$ lb/ft

16, 17, 18, 19 assumed same as 15

23. $w_D = 32'(20 + 27 + 62.5) + 500 = 4000$ lb/ft
    $w_L = 32'(80) = 2560$ lb/ft

24, 25, 27, 28 assumed same as 23

26. $w_D = 14'(20 + 27 + 62.5) + 500 = 2040$ lb/ft
    $w_L = 14'(80) = 1120$ lb/ft

31. $w_D = 14'(20 + 27 + 62.5) + 500 + 560 = 2600$ lb/ft
    $w_L = 14'(80') = 1120$ lb/ft

32, 33, 34 assumed same as 31

21. $w_D = 3.5'(20 + 62.5) + 500 = 800$ lb/ft
    $w_L = 3.5'(80) = 400$ lb/ft

22 assumed same as 21

Design Moments for Lateral System Beams								
Beam #	M_D^+ (k-ft)	M_D^- (k-ft)	M_L^+ (k-ft)	M_L^- (k-ft)	M_E^- (k-ft)	M_U^+ (1.2D+1.6L+0.5Lr) (k-ft)	M_U^- (1.2D+1.6L+0.5Lr) (k-ft)	M_U^- (1.2D+E+L+0.2S) (k-ft)
1	73.8	147.6	30.4	60.8	65.8	137.2	274.3	303.7
2	79.4	158.8	32.7	65.3	58.9	147.5	295.0	314.7
3	32.8	65.6	13.5	27.0	79.9	61.0	121.9	185.6
4	58.3	116.6	24.0	48.0	63.9	108.4	216.8	251.9
5	12.3	24.5	5.0	10.1	109.1	22.8	45.5	148.6
6	31.1	62.1	9.2	18.4	140.7	52.0	104.0	233.7
7	16.8	33.7	5.0	10.0	127.9	28.2	56.4	178.3
8	48.4	96.7	30.7	61.5	88.7	107.2	214.4	266.3
9	66.6	133.3	42.4	84.7	72.4	147.8	295.5	317.1
10	39.0	78.1	24.8	49.7	86.8	86.6	173.1	230.1
11	7.0	14.0	5.1	10.3	107.2	16.6	33.3	134.3
12	47.0	93.9	29.9	59.7	83.1	104.1	208.3	255.5
13	118.5	236.9	75.3	150.7	71.2	262.7	525.4	506.2
14	67.0	133.9	19.8	39.7	97.1	112.1	224.2	297.4
15	45.6	91.3	31.7	63.5	104.0	105.5	211.1	277.0
16	45.6	91.3	31.7	63.5	99.6	105.5	211.1	272.6
17	3.1	6.2	2.2	4.3	191.2	7.2	14.4	202.9
18	3.1	6.2	2.2	4.3	185.7	7.2	14.4	197.5
19	45.6	91.3	31.7	63.5	89.3	105.5	211.1	262.3
20	67.0	133.9	19.8	39.7	76.3	112.1	224.2	276.7
21	18.4	36.8	9.2	18.4	122.8	36.8	73.6	185.3
22	18.4	36.8	9.2	18.4	119.7	36.8	73.6	182.3
23	49.6	99.2	31.7	63.5	89.7	110.3	220.6	272.2
24	68.3	136.7	43.7	87.5	73.2	152.0	304.0	324.7
25	40.0	80.1	25.6	51.3	87.5	89.1	178.1	234.8
26	6.9	13.8	3.8	7.6	105.8	14.3	28.6	129.8
27	48.2	96.3	30.8	61.7	84.1	107.1	214.2	261.3
28	121.5	243.0	77.8	155.5	67.3	270.2	540.4	514.4
29	37.3	74.6	7.7	15.5	132.1	57.1	114.3	237.1
30	37.3	74.6	7.7	15.5	102.4	57.1	114.3	207.4
31	79.0	158.0	34.0	68.0	68.3	149.2	298.4	325.9
32	84.9	169.9	36.6	73.2	61.4	160.5	320.9	338.4
33	84.9	169.9	36.6	73.2	61.4	160.5	320.9	338.4
34	79.0	158.0	34.0	68.0	68.0	149.2	298.4	325.6

Controlling design moment

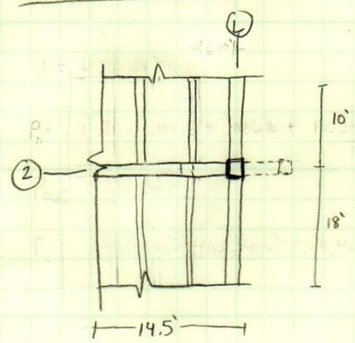
Reinforcing for Lateral System Beams								
Beam #	As,req^{+} (in^{2})	Bars	As,provided^{+} (in^{2})	øMn^{+} (k-ft)	As,req^{-} (in^{2})	Bars	As, provided^{-} (in^{2})	øMn^{-} (k-ft)
1	1.46	*	*	*	3.58	6#7s	3.60	347.3
2	1.58	*	*	*	3.72	5#8s	3.95	380.4
3	0.64	*	*	*	2.15	5#6s	2.20	216.3
4	1.15	*	*	*	2.95	5#7s	3.00	291.8
5	0.24	*	*	*	1.71	*	*	*
6	0.55	*	*	*	2.72	9#5s	2.79	272.2
7	0.30	*	*	*	2.06	5#6s	2.20	216.6
8	1.14	*	*	*	3.12	4#8s	3.16	306.7
9	1.58	*	*	*	3.75	5#8s	3.95	380.2
10	0.92	*	*	*	2.68	9#5s	2.79	272.4
11	0.17	*	*	*	1.54	*	*	*
12	1.11	*	*	*	2.99	5#7s	3.00	291.7
13	2.86	5#7s	3.00	292.2	6.45	6#8s & 2#9s	6.74	626.6
14	1.19	*	*	*	3.51	8#6s	3.52	339.9
15	1.12	*	*	*	3.25	8#6s	3.52	341.0
16	1.12	*	*	*	3.20	8#6s	3.52	341.3
17	0.08	*	*	*	2.35	8#5s	2.48	243.2
18	0.08	*	*	*	2.29	8#5s	2.48	243.4
19	1.12	*	*	*	3.07	7#6s	3.08	299.1
20	1.19	*	*	*	3.25	8#6s	3.52	341.1
21	0.39	*	*	*	2.14	5#6s	2.20	216.3
22	0.39	*	*	*	2.11	5#6s	2.20	216.4
23	1.17	*	*	*	3.20	8#6s	3.52	341.3
24	1.63	*	*	*	3.85	5#8s	3.95	379.7
25	0.94	*	*	*	2.74	9#5s	2.79	272.2
26	0.15	*	*	*	1.49	*	*	*
27	1.14	*	*	*	3.06	7#6s	3.08	299.2
28	2.94	5#7s	3.00	291.8	6.65	6#8s & 2#9s	6.74	625.0
29	0.60	*	*	*	2.77	9#5s	2.79	272.1
30	0.60	*	*	*	2.41	8#5s	2.48	243.0
31	1.59	*	*	*	3.86	5#8s	3.95	379.7
32	1.72	*	*	*	4.02	7#7s	4.20	402.9
33	1.72	*	*	*	4.02	7#7s	4.20	402.9
34	1.59	*	*	*	3.86	5#8s	3.95	379.7

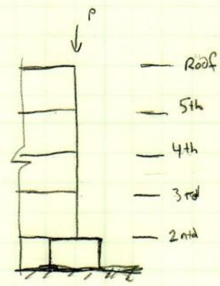
	As,min (in^{2})	Bars	As, provided (in^{2})	øMn (k-ft)
*	1.91	5#6s	2.2	211.02

Appendix D: Special Case Beam Design

Thesis | Final Report | Beam supporting L-2 Columns

Beam supporting Column load





Area = 406 ft^2

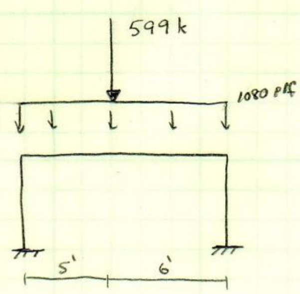
$$P_D = 150\frac{(20")(24")}{144"}(13.5'+9'+17') + \frac{150(8")(20")}{144"}\,2(9'+17') + \frac{5"}{12"}(150)(406) + 20(406)$$

$$P_D = (61.9\text{ k})\ 5 \text{ floors} = 309.5\text{ k}$$

$$P_L = 30(406) + 80(406)\ 4\text{ floors} = 142.1\text{ k}$$

$$P_u = 1.2(309.5) + 1.6(142.1) = 599\text{ k}$$

$$w_u = (1.2)150\frac{(36")(24")}{144"} = 1080\text{ lb/ft}$$



$M_u^+ = 1202$ k-ft midspan

$M_u^- = 165$ k-ft face of support

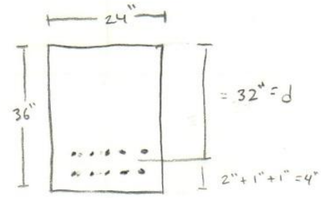
$V_u = 334$ k

→ TRY INCREASING DEPTH TO INCREASE d

36" DEPTH

Thesis | Final Report | Beam Supporting L-2 Columns

Reinforcement



$M_u^+ = 1202$ k-ft

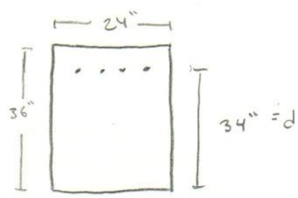
$A_s \cong \frac{1202}{4(32)} = 9.39\ in^2$

$a \cong 5.52"$

$A_{s,req} = \frac{1202(12)}{0.85(60)(32 - \frac{5.52}{2})} = 9.67\ in^2 \longrightarrow$ TRY (10) #9s = 10 in^2

$\phi M_n = \frac{0.9(10)60(32 - \frac{5.52}{2})}{12} = \boxed{1316 \text{ k-ft} > 1202 \text{ k-ft}}$ OK

Reinforcement



$M_u^- = 165$ k-ft

$A_s \cong \frac{165}{4(34)} = 1.22\ in^2$

$a = 0.72"$

$A_{s,req} = 1.15\ in^2 \longrightarrow$ TRY (2) #7s = 1.2 in^2

$\phi M_n = \frac{0.9(1.2)(60)(34 - \frac{0.72}{2})}{12} = \boxed{182 \text{ k-ft} > 165 \text{ k-ft}}$ OK

$A_{s,min} = \frac{3\sqrt{5000}(24)(34)}{60{,}000} = 2.88\ in^2$

USE (5) #7s $\rightarrow A_s = 3\ in^2$

Reinforcement

Shear : $V_u = 334$ k

$\phi V_c = 0.75(2)\sqrt{5000}(24)(32) = 81.4$ k

$V_u > \phi V_c \longrightarrow$ needs shear reinforcement

Thesis | Final Report | Beam Supporting L-2 Columns

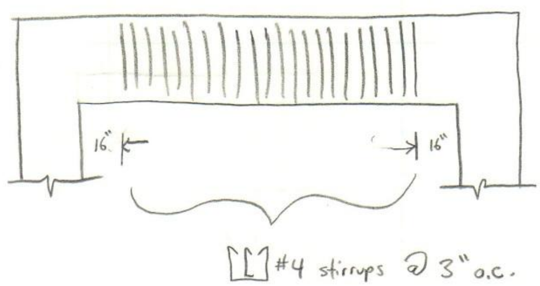
$V_u = \phi V_c + \phi V_s \longrightarrow \phi V_s = 252.6\ k$

use #4 stirrups $A_v = 0.4\ in^2$

$\phi V_s = 0.75(0.4)(60)\frac{32}{s} \geq 252.6k \longrightarrow s \leq 2.28"$

use #4 stirrups $A_v = 0.6\ in^2$

$\phi V_s = 0.75(0.6)(60)\frac{32}{s} \geq 252.6k \longrightarrow s \leq 3.42"$



Appendix E: Foundation Design Checks

Spread Footing Design

RAM Foundation v14.05.01.00
DataBase: office wing
Building Code: IBC
Date: 03/29/13 15:27:22
Design Code: ACI318-08

FOOTING DESIGN

Footing # 84

Footing Column Location: ----- (H - 4)

Footing Orientation (deg): ------	0.00	Column Orientation (deg): -----	0.00
Length (ft): ------------------	19.00		
Width (ft):-------------------	19.00		
Thickness (ft):---------------	3.00		

Bottom Reinf. Parallel to Length: 20 - #8 Width: 20 - #8
Concrete f'c (ksi): 4.50 fct (ksi): CODE Density (pcf): 150.00 Ec (ksi): 4066.84
Reinf. fy (ksi): 60.00
Safety Factor Overturning: Major---- 28.7 (84) Minor---- 26.2 (65)

INPUT DATA

Column Size: *24 x 24
Base Plate Dimensions (in) 0.00 x 0.00 Percent of overhang to assume Rigid: 0.00

LOADS

Surcharge (ksf)	Dead Load:	0.000	Live Load:	0.000
Axial (kip)	Dead Load:	595.00		
	Pos. Live:	249.65	Neg. Live:	N/A
	Pos. Roof:	14.31	Neg. Roof:	N/A

CONCRETE CAPACITY

	Major	Ld Co/Code Ref.	Minor	Ld Co/Code Ref.
Required Shear (kip)	347.96	2	350.54	2
Provided Shear: (kip)	745.62	Sec. 11.5.6.1 a) b) c)	722.67	Sec. 11.5.6.1 a) b) c)
Required Moment: (kip-ft)	2170.76	2	2155.81	2
Provided Moment: (kip-ft)	2272.11		2201.01	
Required Punching Shear: (kip)	1076.51	2		
Provided Punching Shear: (kip)	1442.53			

REINFORCEMENT

	Bottom Bars Parallel to		Top Bars Parallel to	
	Length	Width	Length	Width
Bar Quantity/Bar Size:	20-#8	20-#8	None	None
Required Steel/Provided Steel (in²)	15.08/15.80	15.47/15.80	None	None
Required Steel Code Ref.	Sec. 7.12	Sec. 7.12	None	None
Bar Spacing (in)	11.63	11.63	None	None
Bar Depth (in)	32.50	31.50	None	None

Cover (in) Top N/A Bottom: 3.00 Side: 3.00

SOIL CAPACITY

		Ld Co
Allowable Soil Bearing Capacity (ksf) ------------------	3.00	
Max Unfactored Soil Bearing (ksf) ---------------------	2.95	73
Max Average Unfactored Soil Bearing (ksf) --------------	2.79	43
Max Soil Bearing for Factored Design (ksf) --------------	3.24	2
Max Average Soil Bearing for Factored Design (ksf) -------	3.10	2

Thesis | Final Report | Foundation Design: Shear

RAM Foundation Shear Check

Column H-4 (largest load) square column & footing → 2-way shear controls

- $P_u = 1113$ k
- $24" \times 24"$ column

$$V_c = \min \begin{cases} \left(2 + \frac{4}{\beta_c}\right)\sqrt{f'c}\,[b_o d], & \beta_c = \frac{24}{24} = 1, \quad 6\sqrt{f'c}\,[b_o d] \\ \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'c}\,[b_o d], & \alpha_s = 40 \quad 5.56\sqrt{f'c}\,[b_o d] \\ 4\sqrt{f'c}\,[b_o d] & \leftarrow \text{controls} \end{cases}$$

$\phi V_c = 0.75(4)\sqrt{3000}$ ← [Drawings Spec all foundation concrete is 3000 psi = $f'c$]
$= 164.3$ psi

$q_A = 3000$ psf

$$\left.\begin{aligned} P_D &= 590.79 \text{ k} \\ P_L &= 247.87 \text{ k} \\ P_{roof} &= 14.3 \text{ k} \end{aligned}\right\} P_u = 1.2(P_D) + 1.6(P_L) + 0.5(P_{roof}) = 1113 \text{ k}$$

$$q = \frac{P_u}{b^2} = \frac{1113\text{k}(1000)}{(19)^2(144)} = 21.4 \text{ psi}$$

$$d^2\left[\phi V_c + \frac{q}{4}\right] + d\left[\phi V_c + \frac{q}{2}\right]w = \frac{q}{4}\left[b^2 - w^2\right]$$

$$d^2\left[164.3 + \frac{21.4}{4}\right] + d\left[164.3 + \frac{21.4}{2}\right]24" = \frac{21.4}{4}\left(204^2 - 24^2\right)$$

$$d^2(169.65) + d(4{,}200) - 219564 = 0$$

$$d \geq 25.7"$$

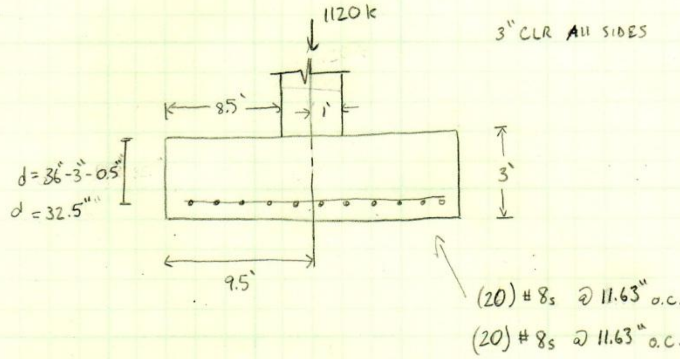
$$25.7" + \frac{0.875"}{2} + 3" = \boxed{29.1" < 36"} \rightarrow 3' \text{ deep OK}$$

Thesis | Final Report | Foundation Design : Reinforcement

RAM Foundation Reinforcement Check

Column H-4

- $P_u = 1120\text{ k}$
- 24"×24" column
- 19'×19' footing
- 3' thick



$$M_u = \frac{(3.08\text{ ksf})(8.5)^2}{2} = 111.4\ \frac{\text{k-ft}}{\text{ft}}$$

$$a = \frac{A_s(60)}{0.85(3)(12)} = 1.96\ A_s$$

$$M_u \le \phi M_n = 0.9\ A_s(60)\left(32.5 - \frac{1.96 A_s}{2}\right)$$

$$52.92\ A_s^2 - 1755\ A_s + 1336.8\ \frac{\text{k-in}}{\text{ft}} = 0 \longrightarrow A_s = 0.78\ \frac{\text{in}^2}{\text{ft}}$$

$$\#8\ @\ 11.63"\text{ o.c.} \longrightarrow A_s = 0.79\frac{(12)}{11.63} = \boxed{0.81\ \frac{\text{in}^2}{\text{ft}} > 0.77\ \frac{\text{in}^2}{\text{ft}}}\quad \text{OK}$$

$$111.4 = 0.9\ A_s(60)\left(32 - \frac{1.96 A_s}{2}\right)$$

$$52.92\ A_s^2 - 1728\ A_s + 1336.8 = 0 \longrightarrow A_s = 0.79\ \frac{\text{in}^2}{\text{ft}}$$

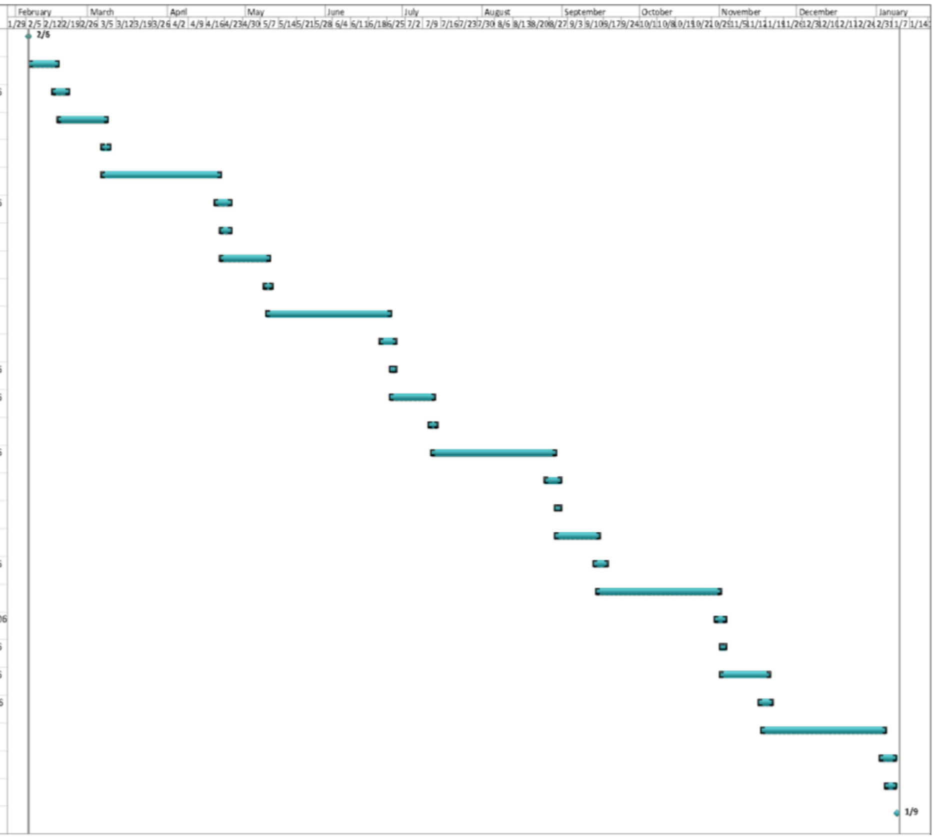
$$\#8\ @\ 11.63"\text{ o.c.} \longrightarrow A_s = 0.79\frac{(12)}{11.63} = \boxed{0.81\ \frac{\text{in}^2}{\text{ft}} > 0.79\ \frac{\text{in}^2}{\text{ft}}}\quad \text{OK}$$

Appendix F: Construction Breadth Data

Building Construction Cost Data 2012										
	FORMING					Costs				
03 11 13	Structural cast-in-place concrete forming	Crew	Daily output	Labor-hours	Unit	Material	Labor	Equipment	Total	Total with O&P
0650	Exterior spandrel,job-built plywood, 24" wide, 4 use	C-2	325	0.148	SFCA	$0.65	$6.35		$7.00	$10.45
1650	Interior beam, job-built plywood, 24" wide, 4 use	C-2	377	0.127	SFCA	$0.99	$5.45		$6.44	$9.50
6650	24"x24" columns, 4 use	C-1	238	0.134	SFCA	$0.83	$5.65		$6.48	$9.55
3550	Floor slab, with 1-way joist pans, 4 use	C-2	500	0.096	SF	$2.92	$4.12		$7.04	$9.55
5150	Spread footings, job-built lumber, 4 use	C-1	414	0.077	SFCA	$0.62	$3.23		$3.85	$5.65
	REINFORCING					Costs				
03 21 10	Uncoated reinforcing steel	Crew	Daily output	Labor-hours	Unit	Material	Labor	Equipment	Total	Total with O&P
0100	Beams & girders, #3 to #7	4 Rodm	1.6	20	Ton	$980.00	$980.00		$1,960.00	$2,650.00
0150	#8 to #18	4 Rodm	2.7	11.852	Ton	$980.00	$580.00		$1,560.00	$2,000.00
0250	Columns, #8 to #18	4 Rodm	2.3	13.913	Ton	$980.00	$685.00		$1,665.00	$2,175.00
0400	Elevated slabs, #4 to #7	4 Rodm	2.9	11.034	Ton	$1,050.00	$540.00		$1,590.00	$2,025.00
0500	Footings, #4 to #7	4 Rodm	2.1	15.238	Ton	$930.00	$750.00		$1,680.00	$2,225.00
0550	#8 to #18	4 Rodm	3.6	8.889	Ton	$880.00	$435.00		$1,315.00	$1,675.00
03 23 05	Uncoated welded wire fabric									
	6x6-W1.4xW1.4	2 Rodm	35	0.457	CSF	$13.80	$22.50		$36.30	$51.00
	CONCRETE					Costs				
03 31 05.35	Normal weight structural concrete	Crew	Daily output	Labor-hours	Unit	Material	Labor	Equipment	Total	Total with O&P
0150	NWC, ready mix, delivered, 3000psi				CY	$102.00			$102.00	$112.00
0350	NWC, ready mix, delivered, 4500psi				CY	$106.00			$106.00	$116.00
400	NWC, ready mix, delivered, 5000psi				CY	$109.00			$109.00	$120.00
2000	For all lightweight aggregate, add				CY	45%				
03 31 05.70	Placing concrete									
0050	Beams, elevated, small beams, pumped	C-20	60	1.067	CY		$40.00	$12.85	$52.85	$75.50
100	Beams, elevated, large beams, pumped	C-20	90	0.711	CY		$27.00	$8.55	$35.55	$50.50
0800	Columns, square 24" thick, pumped	C-20	92	0.696	CY		$26.00	$8.40	$34.40	$49.00
1400	Elevated slab, less than 6" thick, pumped	C-20	140	0.457	CY		$17.25	$5.50	$22.75	$32.50
2600	Footings, spread, over 5 CY, direct chute	C-6	120	0.4	CY		$14.65	$0.46	$15.11	$23.00
2650	Footings, spread, over 5 CY, pumped	C-20	150	0.427	CY		$16.10	$5.15	$21.25	$30.00
	FINISHING					Costs				
03 35 29	Tooled concrete finishing	Crew	Daily output	Labor-hours	Unit	Material	Labor	Equipment	Total	Total with O&P
0100	Bull float only	C-10	4000	0.006	SF		$0.24		$0.24	$0.36
	METAL FASTENINGS					Costs				
05 05 23.85	Weld shear connectors	Crew	Daily output	Labor-hours	Unit	Material	Labor	Equipment	Total	Total with O&P
0200	3/4" diameter, 3-7/8" long	E-10	945	0.017	Each	$0.57	$0.85	$0.41	$1.83	$2.60
	STRUCTURAL STEEL FOR BUILDINGS					Costs				
05 12 23.77	Structural steel projects	Crew	Daily output	Labor-hours	Unit	Material	Labor	Equipment	Total	Total with O&P
0800	Offices, hospitals, etc. steel bearing, 3 to 6 stories	E-6	14.4	8.889	Ton	$2,550.00	$435.00	$124.00	$3,109.00	$3,700.00
4300	Column base plates, light, up to 150 lb.	2Sswk	2000	0.008	lb.	$1.38	$0.39		$1.77	$2.22
4400	Column base plates, heavy, over 150 lb.	E-2	7500	0.007	lb.	$1.44	$0.36	$0.20	$2.00	$2.42
	DECKING					Costs				
05 31 13	Steel floor decking	Crew	Daily output	Labor-hours	Unit	Material	Labor	Equipment	Total	Total with O&P
5300	Non-cellular composite decking, galanized, 2" deep, 20 gauge	E-4	3600	0.009	SF	$1.83	$0.44	$0.03	$2.30	$2.84
05 31 33	Steel form decking									
7100	Sheet metal edge closure form, 12" wide with 2 bends, galvanized, 18 gauge	E-14	360	0.022	LF	$3.59	$1.14	$0.34	$5.07	$6.35

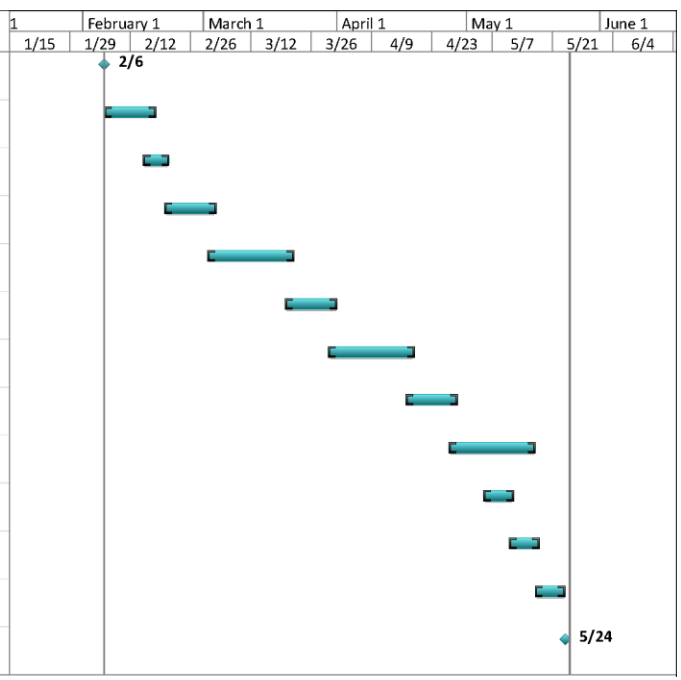
Concrete Structure Schedule

ID	Task Mode	Task Name	Duration	Start
1		Start	0 days	Mon 2/6/06
2		Foundation: Formwork & Rebar Placement	10 days	Mon 2/6/06
3		Foundation: Pour	5 days	Wed 2/15/06
4		1st flr columns: Formwork & Rebar Placement	14 days	Fri 2/17/06
5		1st flr columns: Pour	4 days	Mon 3/6/06
6		2nd floor: Formwork & Rebar Placement	35 days	Mon 3/6/06
7		2nd floor: Pour	5 days	Wed 4/19/06
8		2nd floor: Finish	3 days	Fri 4/21/06
9		2nd flr columns: Formwork & Rebar Placement	14 days	Fri 4/21/06
10		2nd flr columns: Pour	4 days	Mon 5/8/06
11		3rd floor: Formwork & Rebar Placement	35 days	Tue 5/9/06
12		3rd floor: Pour	5 days	Thu 6/22/06
13		3rd floor: Finish	3 days	Mon 6/26/06
14		3rd flr columns: Formwork & Rebar Placement	14 days	Mon 6/26/06
15		3rd flr columns: Pour	4 days	Tue 7/11/06
16		4th floor: Formwork & Rebar Placement	35 days	Wed 7/12/06
17		4th floor: Pour	5 days	Fri 8/25/06
18		4th floor: Finish	3 days	Tue 8/29/06
19		4th flr columns: Formwork & Rebar Placement	14 days	Tue 8/29/06
20		4th flr columns: Pour	4 days	Wed 9/13/06
21		5th floor: Formwork & Rebar Placement	35 days	Thu 9/14/06
22		5th floor: Pour	5 days	Mon 10/30/06
23		5th floor: Finish	3 days	Wed 11/1/06
24		5th flr columns: Formwork & Rebar Placement	14 days	Wed 11/1/06
25		5th flr columns: Pour	4 days	Thu 11/16/06
26		Roof: Formwork & Rebar Placement	35 days	Fri 11/17/06
27		Roof: Pour	5 days	Tue 1/2/07
28		Roof: Finish	3 days	Thu 1/4/07
29		Structure Complete	0 days	Tue 1/9/07

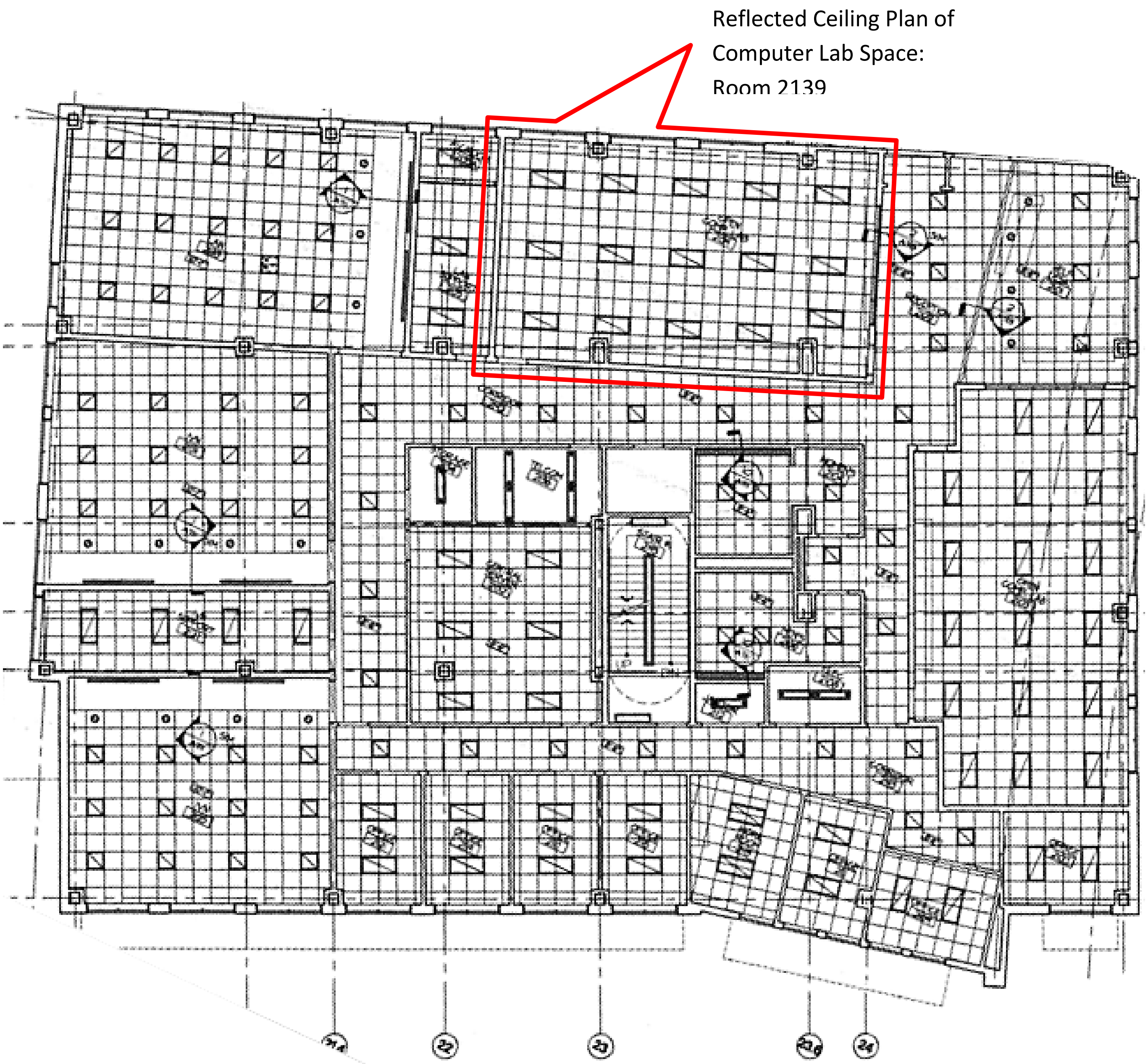


Steel Structure Schedule

ID	Task Mode	Task Name	Duration	Start
1		Start	0 days	Mon 2/6/06
2		Foundation: Formwork & Rebar Placement	10 days	Mon 2/6/06
3		Foundation: Pour	4 days	Wed 2/15/06
4		Erect Steel: Floors 1 &2	10 days	Mon 2/20/06
5		Detail & Decking: Floors 1 & 2	14 days	Thu 3/2/06
6		Erect Steel: Floors 3 & 4	10 days	Mon 3/20/06
7		Detail & Decking: Floors 3 & 4	14 days	Thu 3/30/06
8		Erect Steel: Floor 5 & Roof	10 days	Mon 4/17/06
9		Detail & Decking: Floor 5 & Roof	14 days	Thu 4/27/06
10		Form, Pour, & Finish Deck: Floors 1 & 2	5 days	Fri 5/5/06
11		Form, Pour, & Finish Deck: Floors 3 & 4	5 days	Thu 5/11/06
12		Form, Pour, & Finish Deck: Floor 5 & Roof	5 days	Wed 5/17/06
13		Structure Complete	0 days	Wed 5/24/06



Appendix G: Lighting Breadth Data



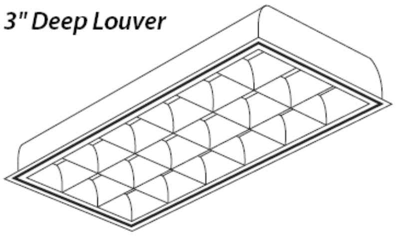
Original Recessed Lighting Data

Columbia
LIGHTING

P424

2' × 4' Recessed Air Handling Parabolic / 2, 3, or 4-Lamp T5, T5HO, T8

3" Deep Louver



FEATURES

- 3" deep cell parabolic louver for optimum control
- IES standard RP-1 glare control models available
- Wide light distribution provides uniform illumination
- Black reveal with full air handling capabilities
- Anodized low iridescent aluminum louver
- Treated with a five stage phosphate bonding process and finished with a baked white enamel
- Mechanical light trap prevents light leaks
- Surface mount available

PROJECT INFORMATION

Project Name

Catalog No.

Type

Date

CONSTRUCTION
Luminaire housing and end caps are die formed code gauge cold rolled steel. Louver is formed from low iridescent anodized aluminum and is secured by die-formed spring steel hinges. Louver hinges from either side. Mechanical light trap prevents light leaks. Latches are finger-tip actuated, positive feed type and completely concealed in the black reveal.

FINISH
Painted parts are treated with a five stage phosphate bonding process and finished with a high temperature baked white enamel. For post painted housing, suffix catalog number with PAF. Regressed slots are flat black.

AIR HANDLING
All supply/return and air extract functions are available as a specified option. Directional control vanes and/or extract damper features available. Air extract slots are located out of sight in end caps. See air removal data on reverse.

INSTALLATION
An access plate is furnished with each luminaire for fast wiring access from the plenum. No need to open fixture. Product ships standard with mylar dust cover to eliminate job site contamination.

BALLASTS
Energy efficient, thermally protected, automatic resetting, Class P, high power factor, sound rated A, magnetic or electronic ballasts. CEE NEMA Premium compliant.

ELECTRICAL
Standard class "P", thermally protected, autoresetting HPF ballast, sound rated A. CEE NEMA Premium compliant. All ballast leads extend a minimum of 6" through access location. NEC/CEC-compliant ballast disconnect is standard.

CEILING COMPATIBILITY
Luminaire is available to fit most standard ceiling types. NEC-compliant tee-bar clips supplied with all grid trim fixtures. See ceiling details on reverse. Contact your Columbia representative for compatibility information for specific ceiling types.

CERTIFICATION
All luminaires are built to UL 1598 standards and bear appropriate UL and cUL or CSA labels. Damp location labeling is standard. Emergency-equipped fixtures labeled UL 924.

PHOTOMETRIC DATA

Test **11939** Test Date **6/11/03**

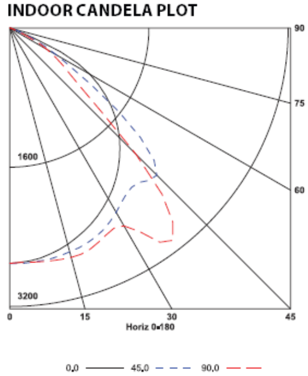
LUMINAIRE DATA

Luminaire	**P424-332G-LD36-3E-PAF** **P4 Parabolic** **2 × 4 3-Lamp with 3 × 6 Cell Semi-Specular Louver**
Ballast	**E332PI120G01**
Ballast Factor	**0.87**
Lamp	**F32T8**
Lumens per Lamp	**2900**
Watts	**90**
Shielding Angle	**N/A**
Spacing Criterion	**Along = 1.26 Across = 1.66**

ZONAL LUMEN SUMMARY

Zone	Lumens	% Lamp	% Fixt.
0-30	2175	25.0	32.3
0-40	3713	42.7	55.1
0-60	6309	72.5	93.6
0-90	6740	77.5	100.0
0-180	6740	77.5	100.0

INDOOR CANDELA PLOT



ENERGY DATA

Total Luminaire Efficiency	**77.5%**
Luminaire Efficacy Rating (LER)	**65**
IESNA RP-1-1993 Compliance	**Noncompliant**
Comparative Yearly Lighting Energy Cost per 1000 Lumens	**$3.69 based on 3000 hrs. and $0.08 per KWH**

AVG. LUMINANCE (Candela/Sq. M.)

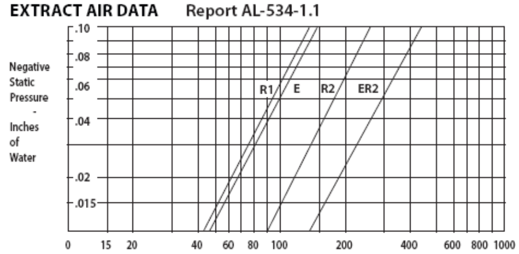
Average Luminance Angle	0.0	22.5	45.0	67.5	90.0
0	4362	4362	4362	4362	4362
30	4213	4366	4554	4703	4888
40	4127	4361	4873	6054	6140
45	4065	4323	5450	5178	4672
50	3960	4266	4921	3067	2710
55	3774	4151	3271	2126	2081
60	3420	3646	2000	1616	1607
65	2443	2107	1237	924	775
70	424	491	448	283	259
75	156	181	162	168	156
80	102	111	130	130	121
85	56	56	93	93	19

COEFFICIENTS OF UTILIZATION (%)

RC	80				70				50			0
RW	70	50	30	10	70	50	30	10	50	30	10	0
RCR 1	86	83	81	78	84	82	79	77	78	76	75	69
2	80	74	70	66	78	73	69	66	70	67	64	60
3	73	66	61	57	72	65	60	56	63	59	55	52
4	68	60	54	49	66	59	53	49	57	52	48	45
5	63	54	47	43	61	53	47	42	51	46	42	40
6	58	48	42	38	56	48	42	37	46	41	37	35
7	54	44	38	33	52	43	37	33	42	37	33	31
8	50	40	34	30	49	40	34	30	39	33	29	28
9	47	37	31	27	46	36	31	27	36	30	27	25
10	44	34	28	24	43	34	28	24	33	28	24	23

RCR = Room Cavity Ratio
RC = Effective Ceiling Cavity Reflectance **RW** = Wall Reflectance

EXTRACT AIR DATA Report AL-534-1.1



R1 - Air Return thru One Side Passage
R2 - Air Return thru Two Side Passages
E - Heat Extract thru Lamp Compartment
ER2 - Thru Both Sides and Lamp Compartment

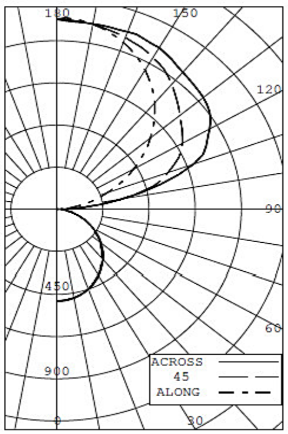
New Pendant Lighting Data

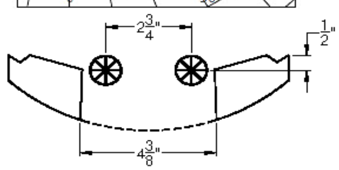
Lighting Sciences
www.lightingsciences.com

Lighting Sciences Inc.
7826 E. Evans Road
Scottsdale, Arizona 85260 USA
Tel: 480-991-9260 • Fax: 480-991-0375

INDEPENDENT TEST LABORATORY REPORT No. 29704

FINELITE - 4 FT FLUORESCENT LUMINAIRE, CAT# S12-ID-DCO-2T8-91W-OPEN
WITH WHITE INTERIOR AND WHITE CURVED PLASTIC LENS
TWO SYLVANIA 32 WATT T8 LAMPS, CAT# FO32/835/ECO. LUMEN RATING = 3100 LMS.
ONE SYLVANIA QHE2X32T8/UNV ISN SC BALLAST OPERATING AT 120 VAC AND 56.8 WATTS





INTENSITY (CANDLEPOWER) SUMMARY

ANGLE	ALONG	22.5	45	67.5	ACROSS	OUTPUT LUMENS
0	486	486	486	486	486	
5	485	483	482	485	483	47
15	466	464	463	467	466	131
25	427	427	427	431	431	197
35	373	374	375	379	379	235
45	308	309	311	316	316	240
55	235	238	239	243	243	214
65	162	163	162	164	165	161
75	94	89	88	90	91	95
85	22	23	29	34	36	33
90	0	3	11	17	19	
95	32	113	126	130	130	133
105	179	341	488	521	523	440
115	350	468	656	762	803	608
125	518	609	747	869	909	656
135	673	731	829	908	944	633
145	805	832	912	960	978	562
155	910	915	960	997	1006	441
165	981	977	991	1001	1007	280
175	1020	1009	1012	1007	1008	98
180	1015	1015	1015	1015	1015	

ZONAL LUMENS AND PERCENTAGES

ZONE	LUMENS	% LAMP	%LUMINAIRE
0-30	375	6.04	7.20
0-40	609	9.83	11.71
0-60	1063	17.15	20.44
0-90	1353	21.82	26.01
40-90	744	12.00	14.29
60-90	290	4.67	5.57
90-180	3850	62.09	73.99
0-180	5203	83.92	100.00

** EFFICIENCY: 83.9% **

LUMINOUS LENGTH: 40.000 INS
WIDTH: 4.380 INS

S/MH: 1.2
SC: 1.2

LUMINANCE SUMMARY CD./SQ.M.

ANGLE	ALONG	45	ACROSS
45	3848	3911	3973
55	3622	3703	3767
65	3398	3410	3462
75	3204	3016	3106
85	2184	2933	3652

CERTIFIED BY: Ryder Tiernan DATE: OCT 19, 2011
PREPARED FOR:
FINELITE
UNION CITY, CA

TESTED IN ACCORDANCE WITH IES PROCEDURES.

LIGHTING SCIENCES, INC.
7826 E. EVANS RD.
SCOTTSDALE, AZ, USA 85260

INDEPENDENT TEST LABORATORY REPORT No. 29704

FINELITE - 4 FT FLUORESCENT LUMINAIRE, CAT# S12-ID-DCO-2T8-91W-OPEN
WITH WHITE INTERIOR AND WHITE CURVED PLASTIC LENS
TWO SYLVANIA 32 WATT T8 LAMPS, CAT# FO32/835/ECO. LUMEN RATING = 3100 LMS.
ONE SYLVANIA QHE2X32T8/UNV ISN SC BALLAST OPERATING AT 120 VAC AND 56.8 WATTS

COEFFICIENTS OF UTILIZATION

ZONAL CAVITY METHOD

EFFECTIVE FLOOR CAVITY REFLECTANCE = .20

CC	90				80				70				50			30			10			0
WALL	70	50	30	10	70	50	30	10	70	50	30	10	50	30	10	50	30	10	50	30	10	0
RCR																						
0	0.94	0.94	0.94	0.94	0.85	0.85	0.85	0.85	0.76	0.76	0.76	0.76	0.59	0.59	0.59	0.43	0.43	0.43	0.29	0.29	0.29	0.22
1	0.86	0.82	0.78	0.76	0.78	0.74	0.71	0.68	0.69	0.66	0.64	0.61	0.51	0.50	0.48	0.38	0.37	0.36	0.25	0.25	0.24	0.19
2	0.78	0.72	0.66	0.61	0.71	0.65	0.60	0.56	0.63	0.58	0.54	0.50	0.45	0.42	0.40	0.33	0.32	0.30	0.22	0.21	0.21	0.16
3	0.72	0.63	0.56	0.51	0.65	0.57	0.51	0.47	0.57	0.51	0.46	0.42	0.40	0.37	0.34	0.30	0.28	0.26	0.20	0.19	0.18	0.13
4	0.66	0.55	0.48	0.43	0.59	0.50	0.44	0.39	0.52	0.45	0.40	0.36	0.35	0.32	0.29	0.26	0.24	0.22	0.18	0.16	0.15	0.12
5	0.60	0.48	0.42	0.36	0.54	0.44	0.38	0.33	0.48	0.40	0.34	0.30	0.31	0.27	0.24	0.23	0.21	0.19	0.16	0.14	0.13	0.10
6	0.55	0.44	0.36	0.31	0.49	0.40	0.33	0.29	0.44	0.36	0.30	0.26	0.28	0.24	0.21	0.21	0.18	0.16	0.14	0.13	0.11	0.09
7	0.51	0.39	0.32	0.27	0.46	0.36	0.29	0.25	0.40	0.32	0.26	0.23	0.25	0.21	0.18	0.19	0.16	0.14	0.13	0.11	0.10	0.07
8	0.47	0.35	0.28	0.23	0.42	0.32	0.26	0.21	0.37	0.29	0.23	0.20	0.23	0.19	0.16	0.17	0.14	0.12	0.12	0.10	0.09	0.06
9	0.43	0.32	0.25	0.20	0.39	0.29	0.23	0.19	0.34	0.26	0.21	0.17	0.20	0.17	0.14	0.15	0.13	0.11	0.11	0.09	0.07	0.06
10	0.40	0.29	0.22	0.18	0.36	0.26	0.20	0.16	0.32	0.24	0.18	0.15	0.19	0.15	0.12	0.14	0.11	0.09	0.10	0.08	0.07	0.05

LUMINAIRE INPUT WATTS 56.8

LABORATORY RESULTS MAY NOT BE REPRESENTATIVE OF FIELD PERFORMANCE.
BALLAST AND FIELD FACTORS HAVE NOT BEEN APPLIED.

TEST DISTANCE EXCEEDS FIVE TIMES THE GREATEST
LUMINOUS OPENING OF LUMINAIRE.

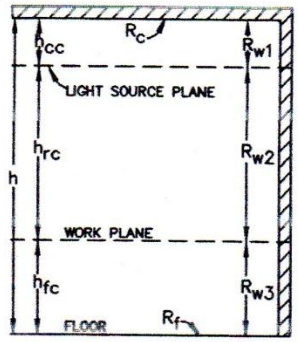
CALCULATIONS OF ILLUMINATION

AVERAGE ILLUMINANCE CALCULATION FORM

FOR ROOM: 2139 COMPUTER LAB

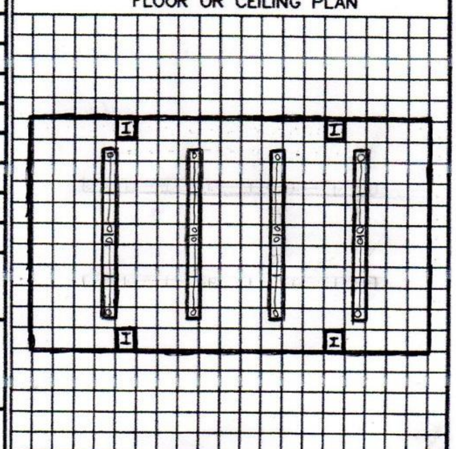
PROJECT: UNIVERSITY ACADEMIC CENTER
PROJECT NO: LIGHTING BREADTH
CALCULATION BY: ALEXANDER ALTEMOSE
DATE: ______ PAGE: ______

ILLUMINANCE CRITERIA	IES ILLUMINANCE CATEGORY					
	MAINTAINED ILLUMINANCE, FC, (LUX)					30
FIXTURE DATA	MFR/MODEL	FINELITE / S12 - ID - DCO				
	TYPE DISTRIBUTION	GENERAL DIFFUSE				
	NO. OF LAMPS PER FIXTURE					2
	RATED LAMP LUMEN & WATTS/LAMP					3100/32
	LUMENS PER FIXTURE (LPF)					5203
ROOM DIMENSIONS	h	11	W, width	25	L, length	42
ROOM CHARACTERS	h_{cc}	1	R_c	0.8	R_{w1}	0.7
	h_{rc}	7.5	R_w	0.7	R_{w2}	0.7
	h_{fc}	2.5	R_f	0.2	R_{w3}	0.7



P	PERIMETER, FT(M):		156
A	AREA, SF(SM):		1044
PAR	PERIMETER/AREA RATIO (P ÷ A)		0.15
CCR	2.5 x PAR x h_{cc}		0.375
RCR	2.5 x PAR x h_{rc}		2.81
FCR	2.5 x PAR x h_{fc}		0.94
ρ_{cc}	FROM R_c & R_{w1} & CCR		0.8
ρ_w	SAME AS R_w OR R_{w2}		0.7
ρ_{fc}	FROM R_f & R_{w3} & FCR		0.2
CU	FROM CU TABLE OF FIXTURE MFGR. INTERPOLATING BETWEEN RCR AND ρ_{cc}, ρ_w, ρ_{fc}		0.66
LOF	BF – BALLAST FACTOR	0.88	0.88
	VF – VOLTAGE FACTOR	–	
	OTHER	–	
LLF	LLD–LAMP LUMEN DEPREC.	0.9	0.837
	LDD–LUMINAIRE DIRT DEPREC.	0.93	
	OTHER	–	

FLOOR OR CEILING PLAN



(USE SEPARATE DRAWINGS FOR ADDITIONAL LAYOUTS)

CALCULATIONS

MAINTAINED ILLUMINANCE

$$E = \frac{N \times (LPF \times LOF) \times CU \times LLF}{A}$$

INITIAL ILLUMINANCE

$$E_i = E \div LLF$$

CALCULATION & REMARKS:

$$30 = \frac{N \times (5203 \times 0.88) \times 0.66 \times 0.837}{1044} \rightarrow N = 12.38$$

TRY 12 in 4 rows

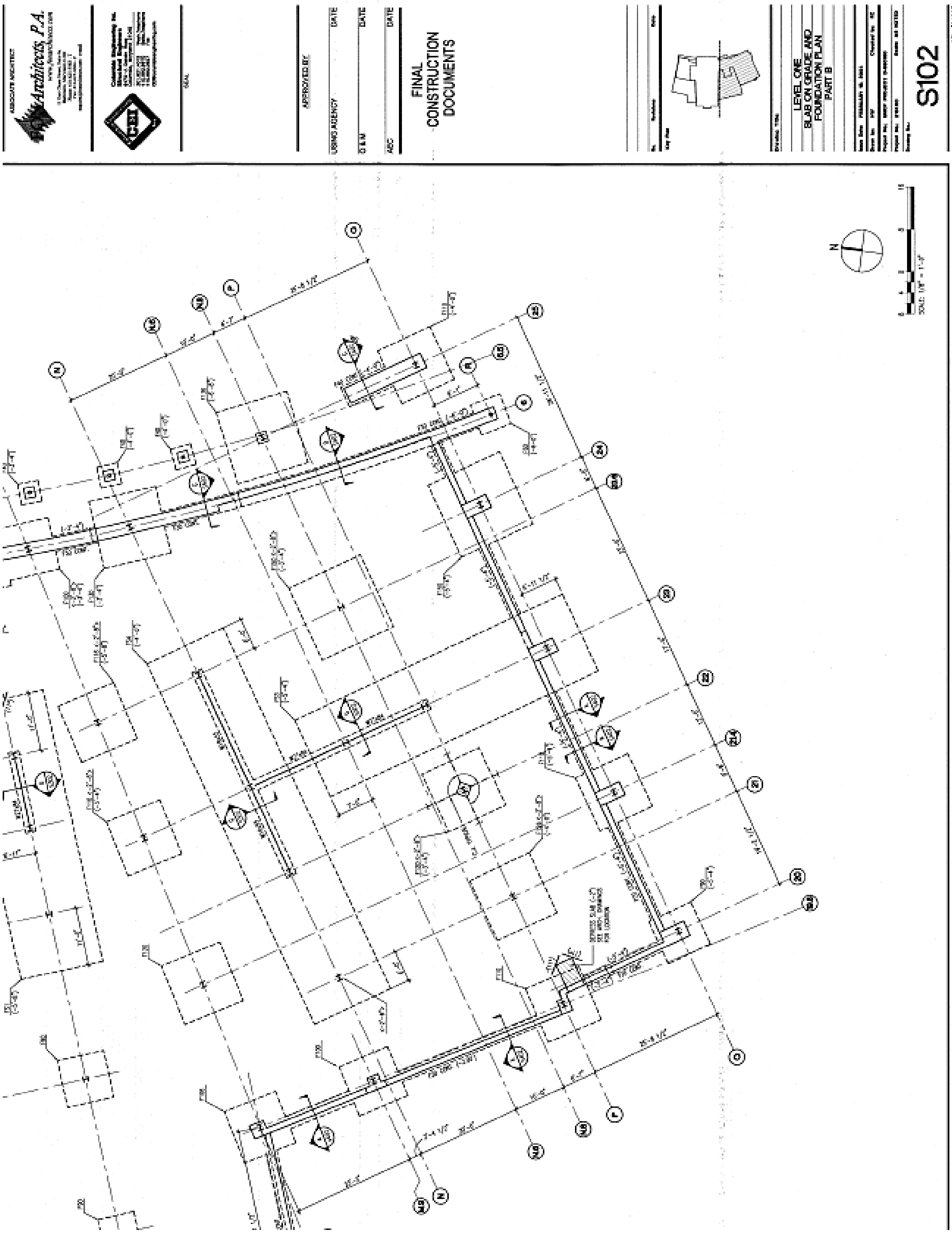
$$E = \frac{12 \times 5203 \times 0.88 \times 0.66 \times 0.83}{1044} = 28.8\ fc$$

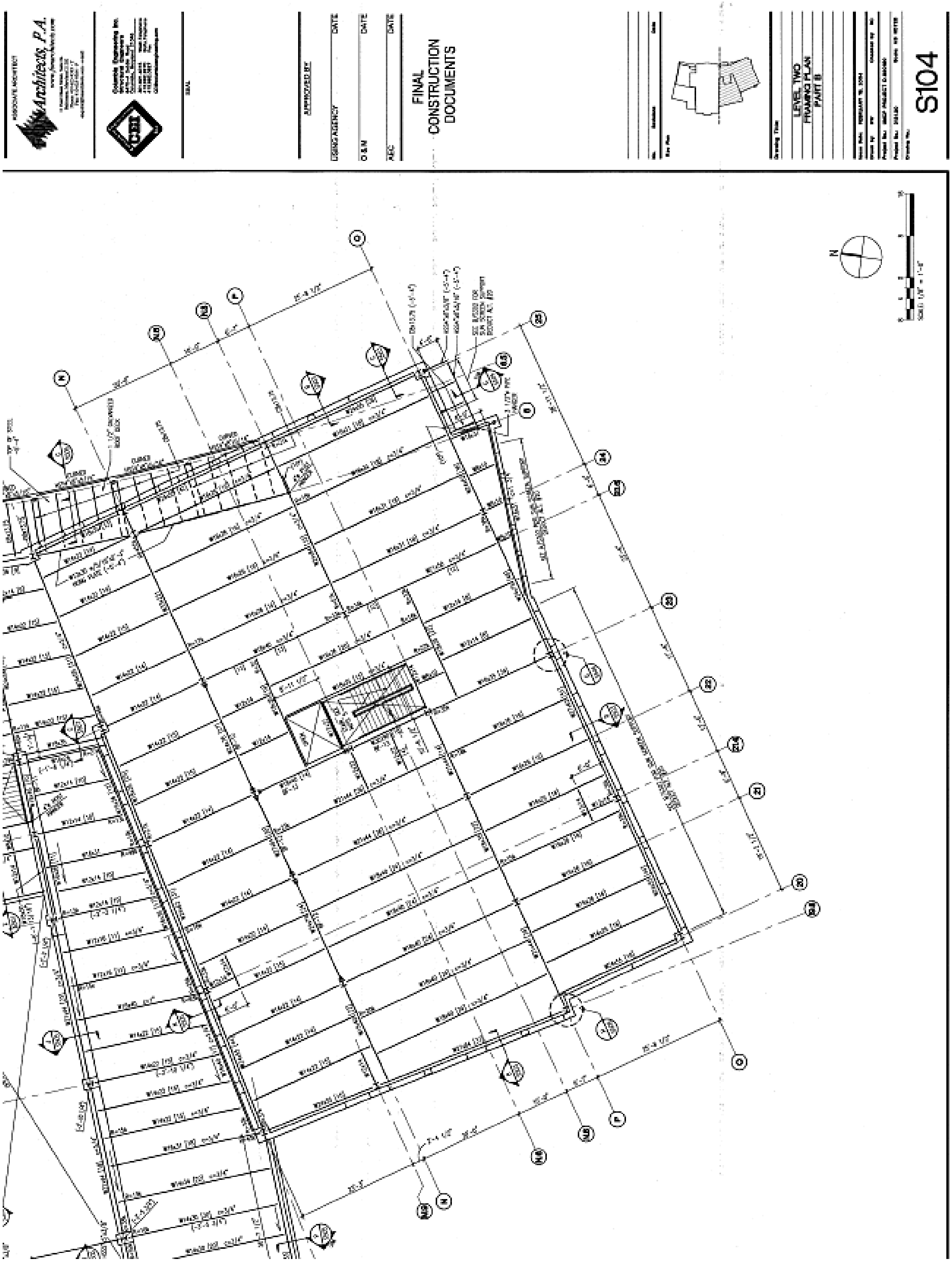
* N – NUMBER OF FIXTURES

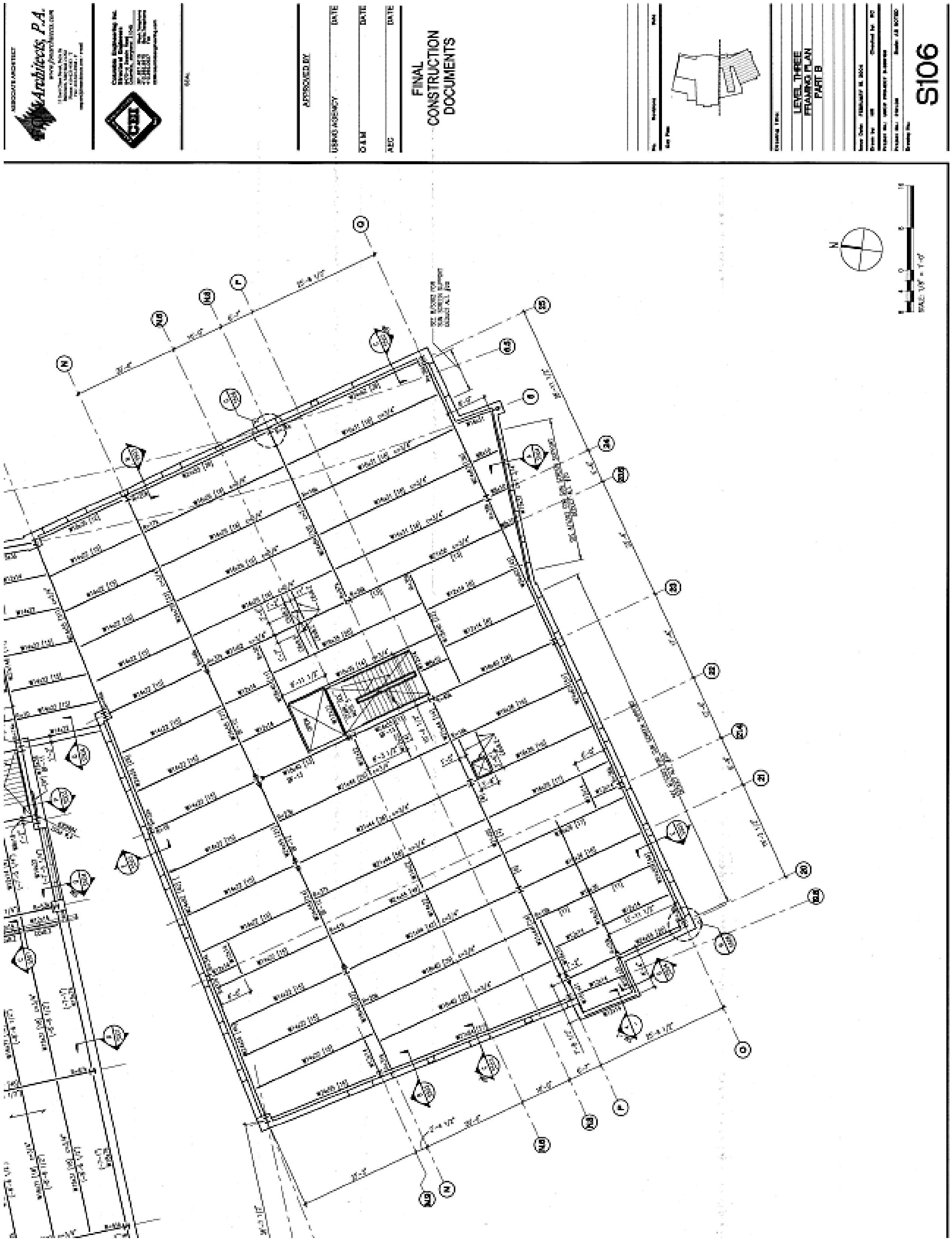
■ FIGURE 7
Typical average illuminance calculation form using the zonal cavity method. (Courtesy: William Tao & Associates, St. Louis, MO.)

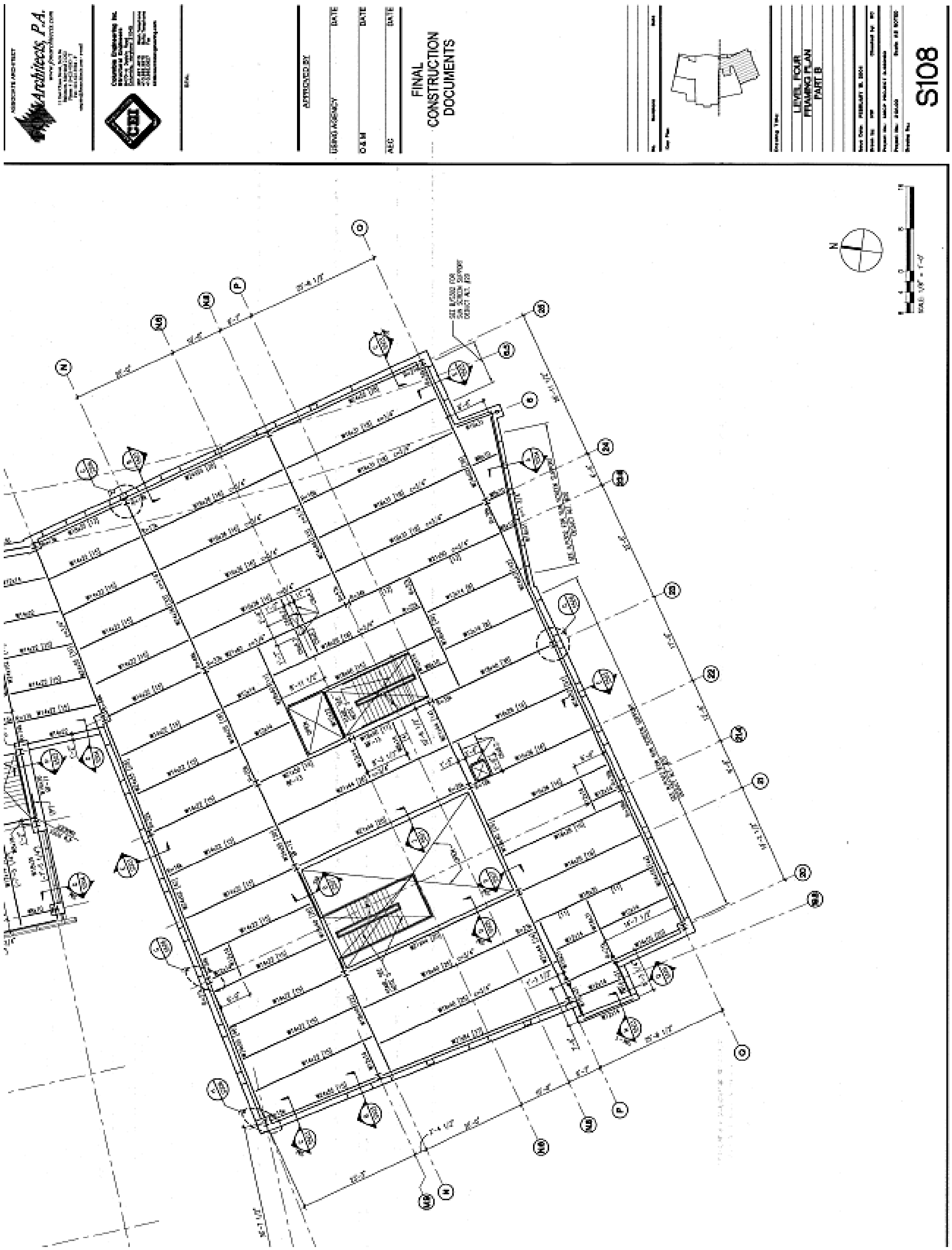
214

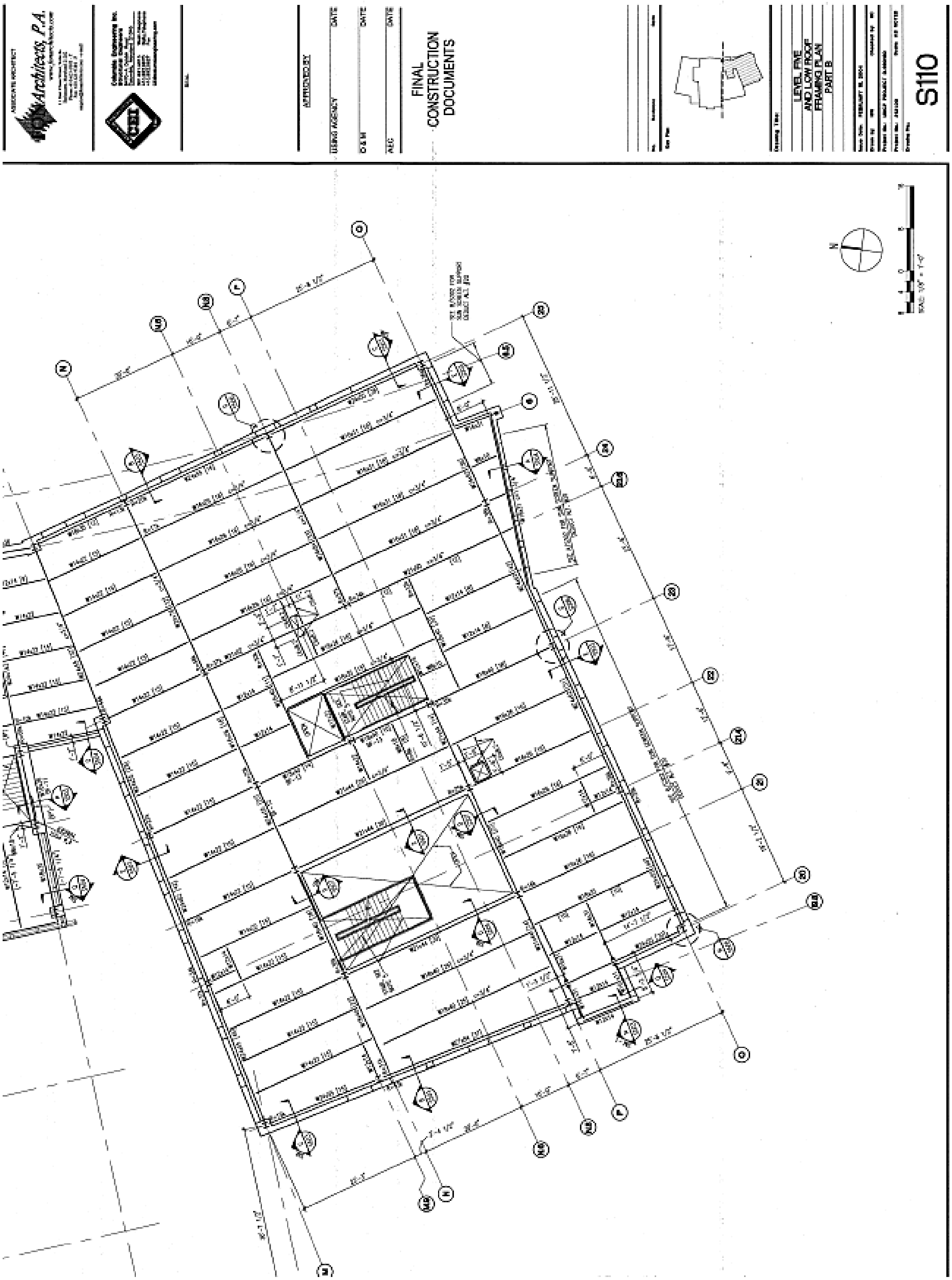
Appendix H: Relevant Floor Plans

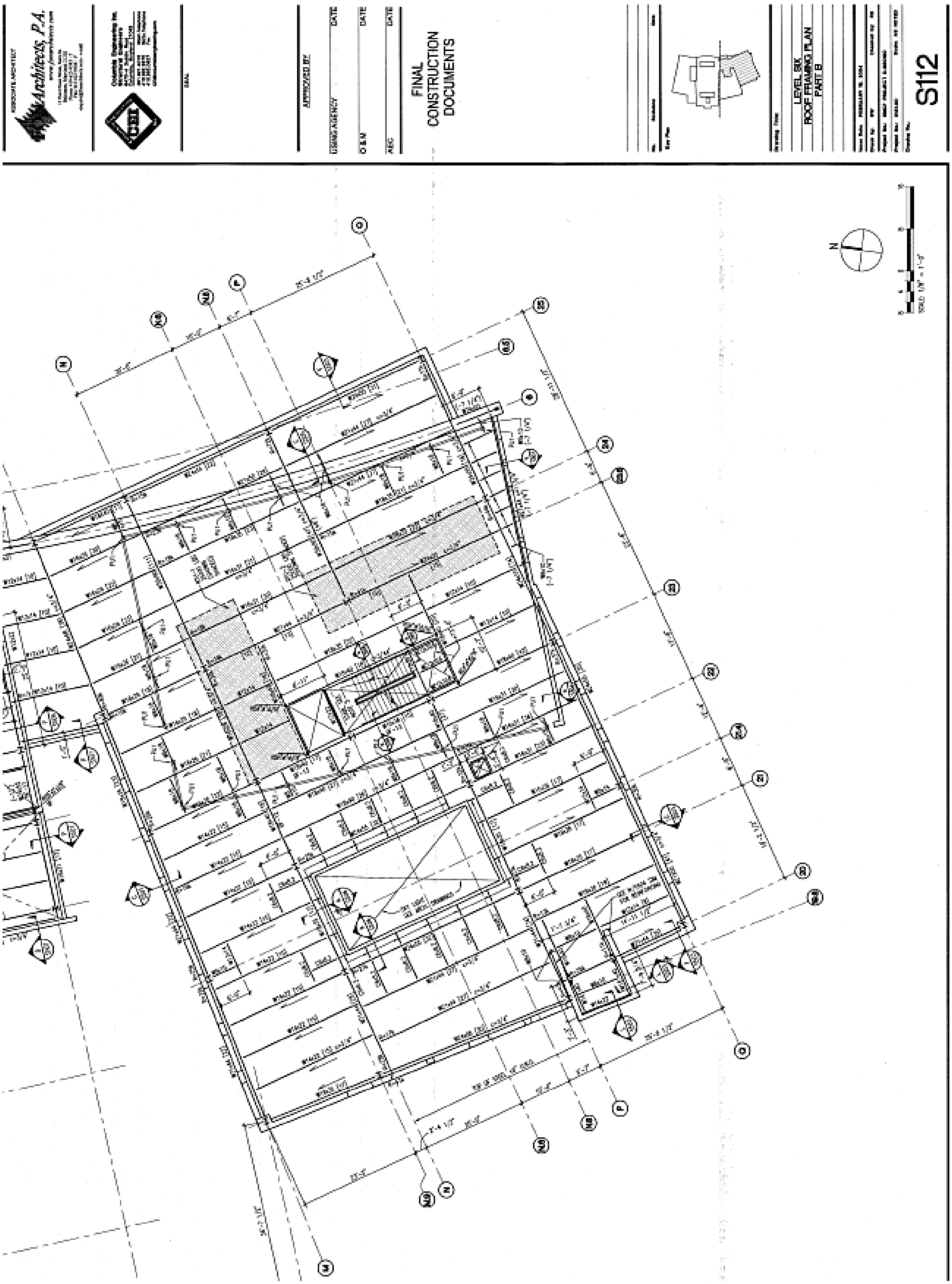



FINAL
CONSTRUCTION
DOCUMENTS
LEVEL TWO
FRAMING PLAN
PART B
S104

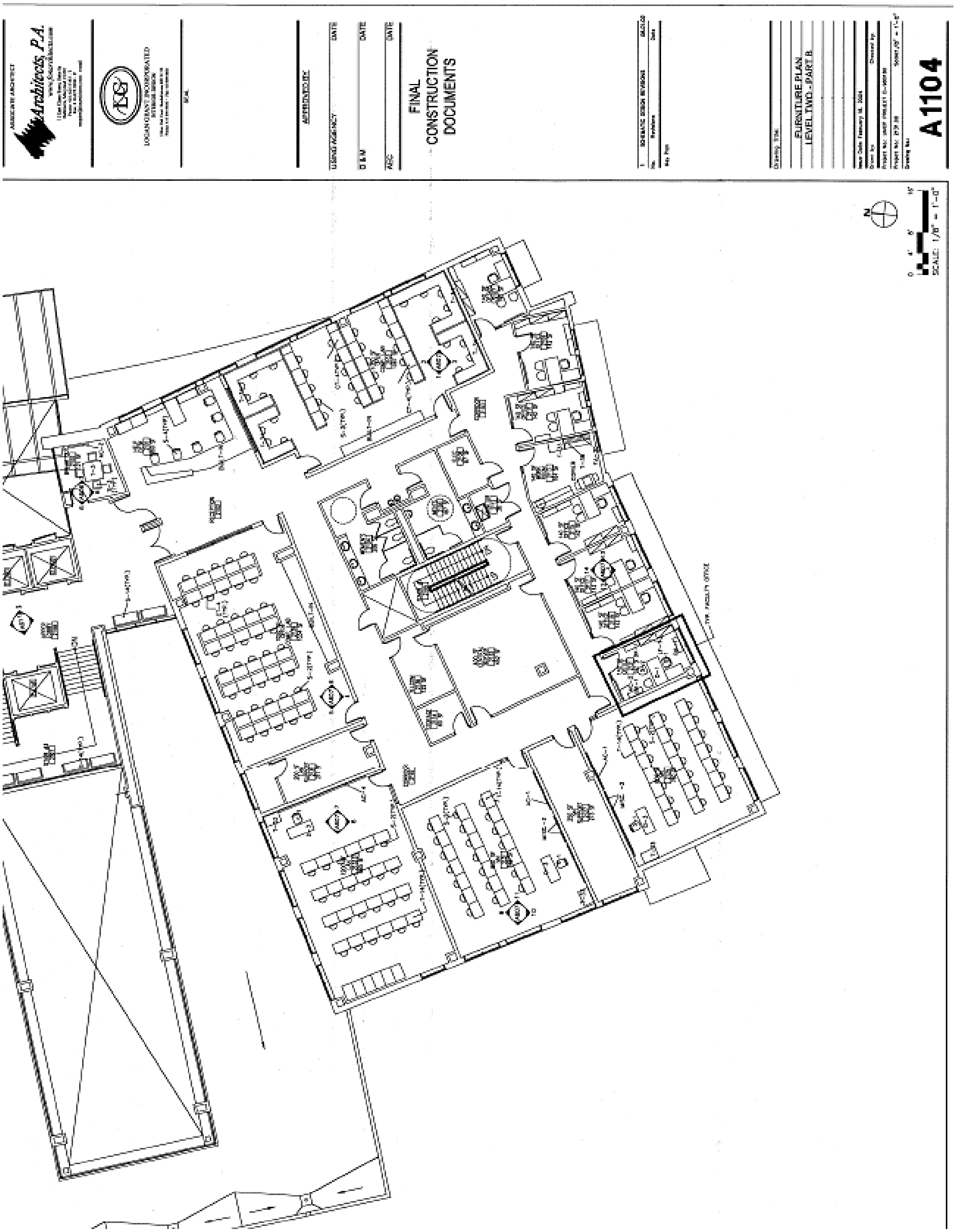

FINAL
CONSTRUCTION
DOCUMENTS
LEVEL THREE
FRAMING PLAN
PART B
S106

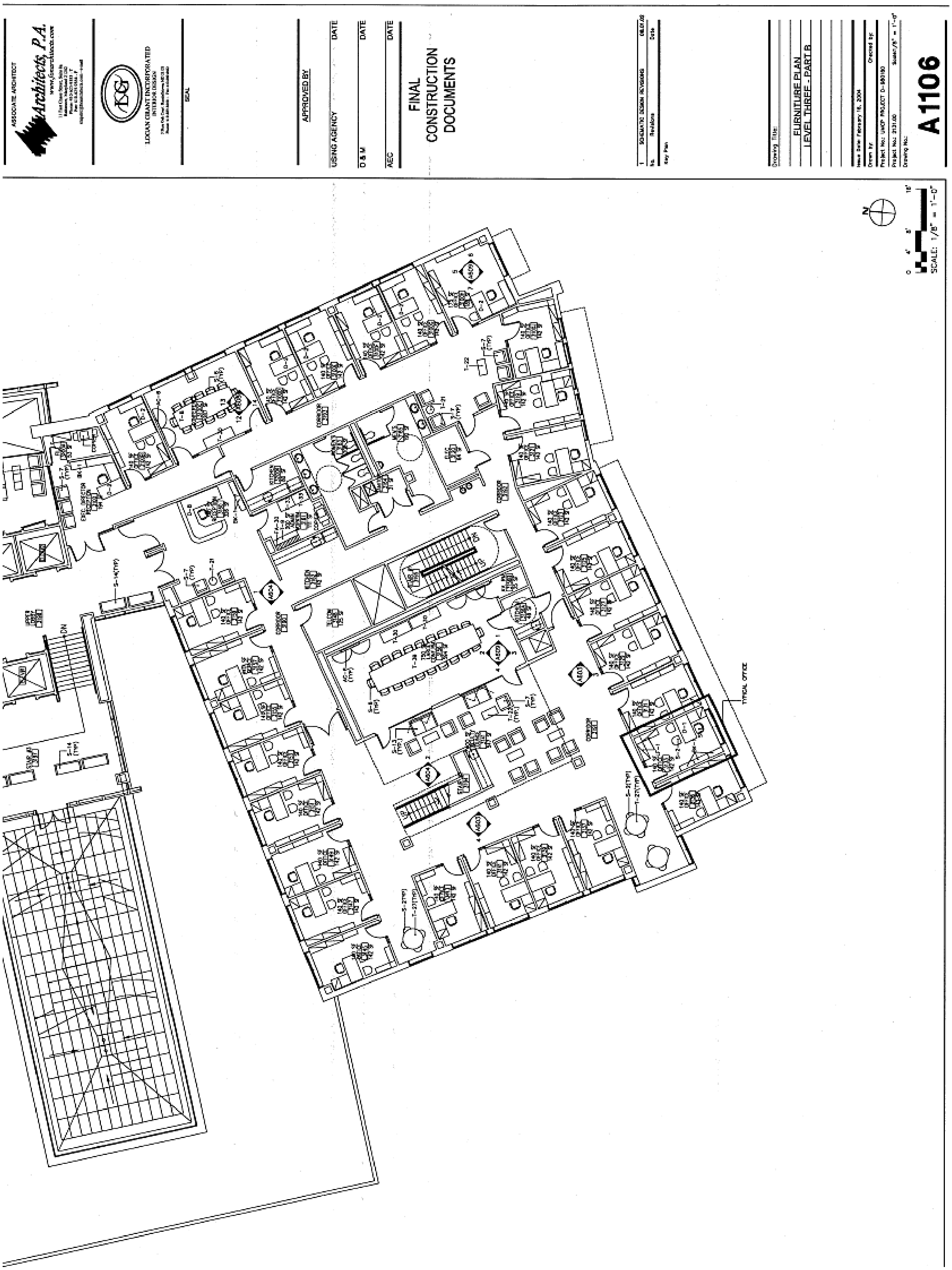

FINAL
CONSTRUCTION
DOCUMENTS
LEVEL FOUR
FRAMING PLAN
PART B
S108
APPROVED BY
USING AGENCY
DATE
O & M
DATE
A/E
DATE

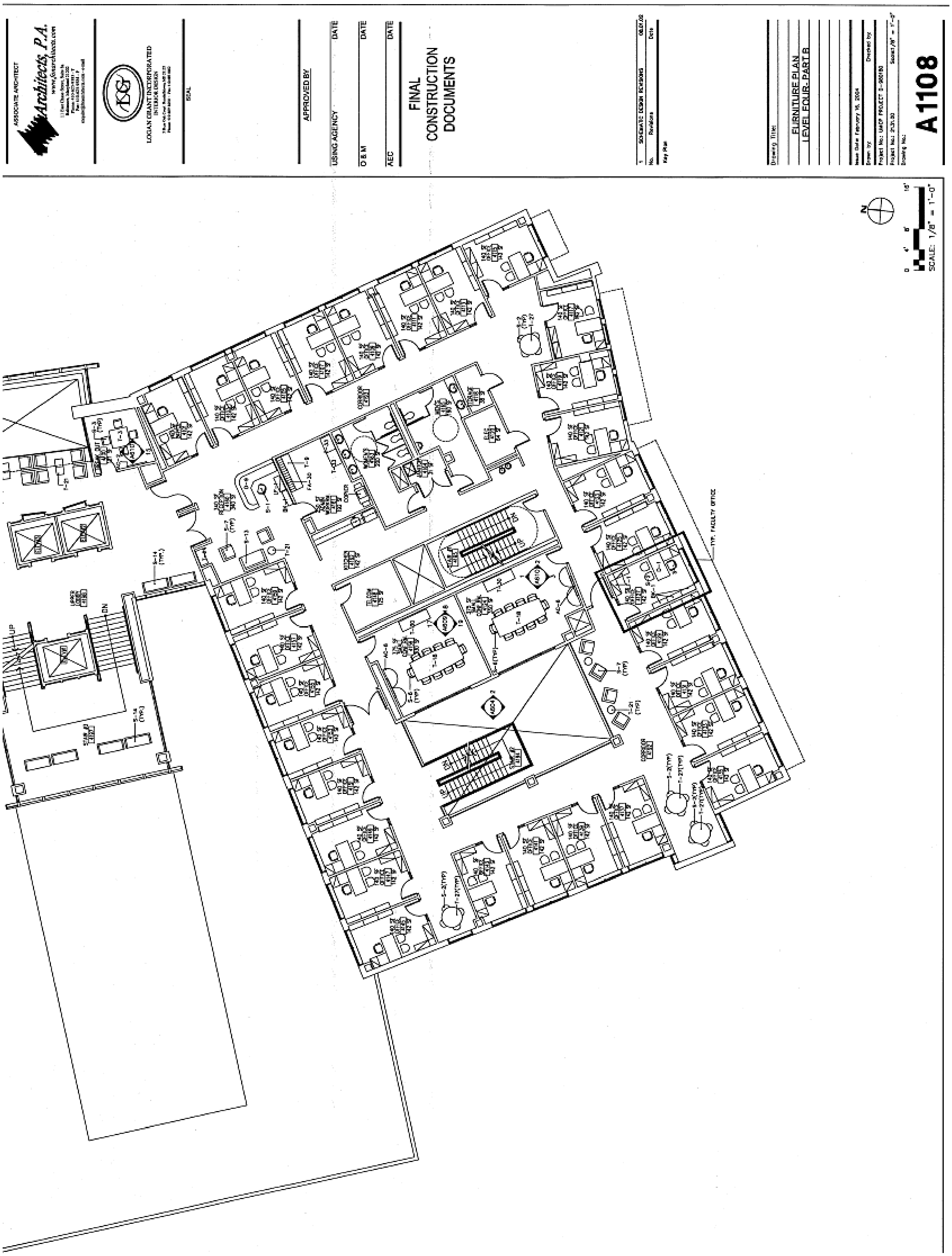

FINAL
CONSTRUCTION
DOCUMENTS
LEVEL FIVE
AND LOW ROOF
FRAMING PLAN
PART B
S110

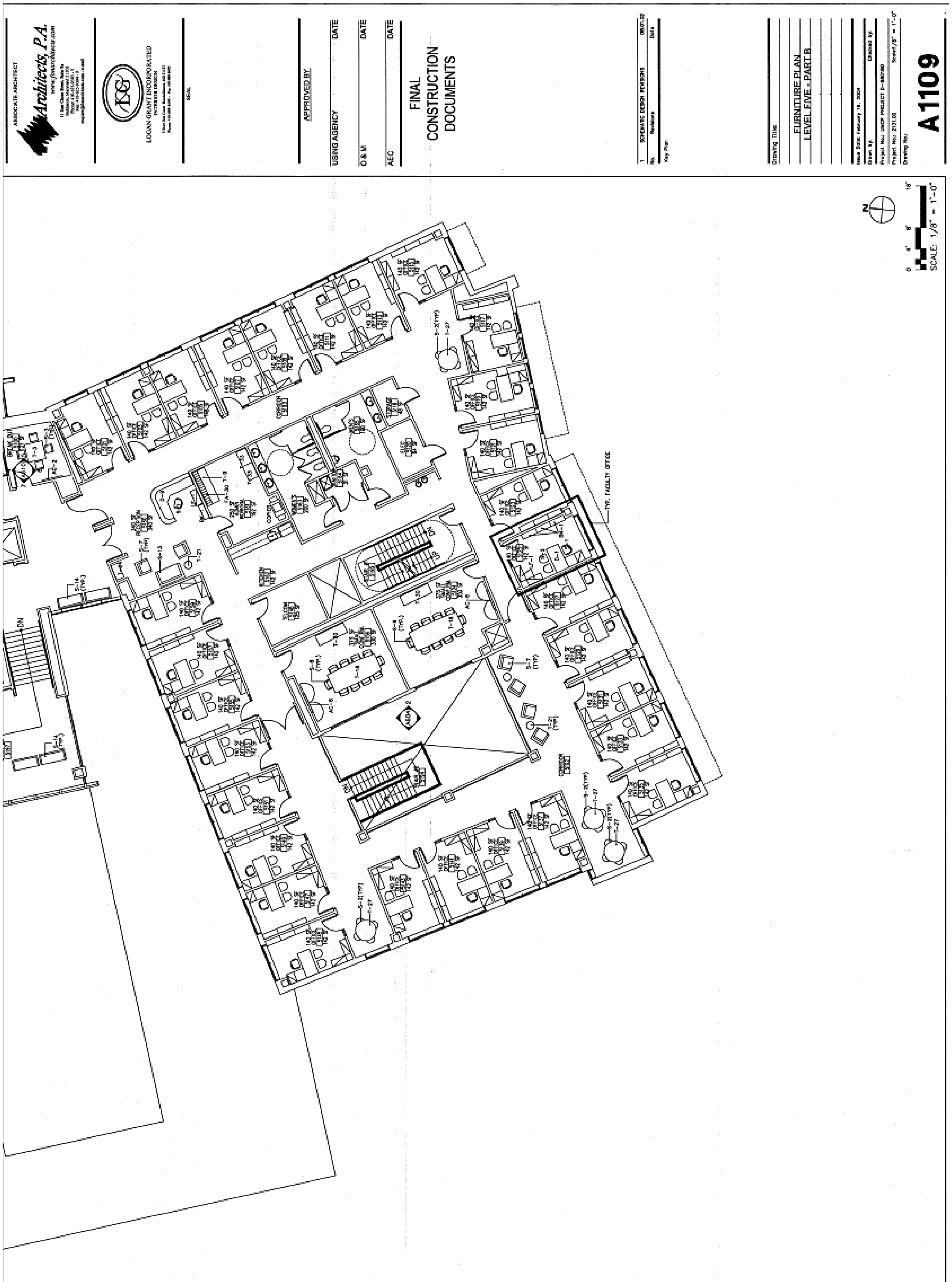

FINAL CONSTRUCTION DOCUMENTS
APPROVED BY
USING AGENCY
DATE
O & M
DATE
A/E/C
DATE
LEVEL SIX ROOF FRAMING PLAN PART B
S112


FINAL
CONSTRUCTION
DOCUMENTS
FURNITURE PLAN
LEVEL ONE - PART B
A1102
SCALE: 1/8" = 1'-0"


FINAL
CONSTRUCTION
DOCUMENTS
FURNITURE PLAN
LEVEL TWO - PART B
A1104


APPROVED BY
USING AGENCY
DATE
D & M
DATE
AEC
DATE
FINAL
CONSTRUCTION
DOCUMENTS
FURNITURE PLAN
LEVEL THREE - PART B
A1106
TYPICAL OFFICE
SCALE: 1/8" = 1'-0"


FINAL
CONSTRUCTION
DOCUMENTS
FURNITURE PLAN
LEVEL FOUR- PART B
A1108


Architects, P.A.
LOGAN GRANT INCORPORATED
APPROVED BY
USING AGENCY
DATE
O & M
DATE
AEC
DATE
FINAL
CONSTRUCTION
DOCUMENTS
FURNITURE PLAN
LEVEL FIVE - PART B
A1109
SCALE: 1/8" = 1'-0"
TYP. FACULTY OFFICE