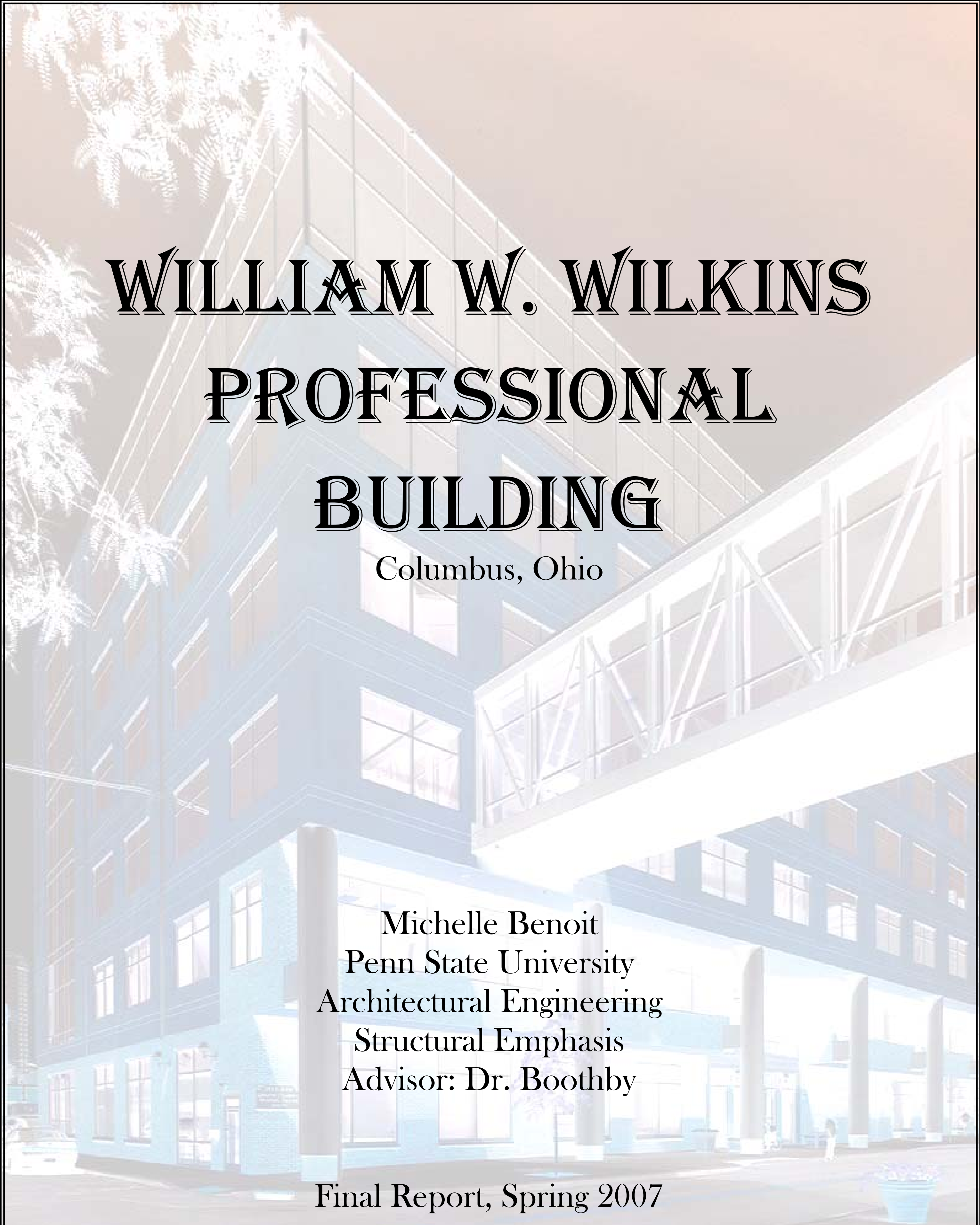

William W. Wilkins Professional Building

Columbus, Ohio

Michelle Benoit
Penn State University
Architectural Engineering
Structural Emphasis
Advisor: Dr. Boothby

Final Report, Spring 2007

William W. Wilkins
Professional Building
Columbus, OH

Building Information:
- 6 story medical office building
- Roughly 112,000 sq. ft.
- $7.4 Million plus tenant improvements
- Design-Bid-Build
- Construction Dates: 3/02—1/03

Project Team:
Owner: State Sixth LLC
Occupant: Grant/Riverside Methodist Hospitals
Architect/Engineers: URS
Geotechnical Engineer: CTL Engineering, Inc.
CM/GC/Developer: The Daimler Group

Construction:
- Building footprint almost exact size of lot
- 2 cranes used, 1 for steel and 1 for precast panel erection
- Constructed over buried building remains

Architecture:
- Brick veneer, precast concrete and spandrel glass façade
- Tinted insulated punched windows
- Pedestrian bridge connecting from third floor to hospital across Sixth Street
- Roof: ballasted EPDM membrane over 3” decking with rigid insulation

Lighting:
- Central lobby lighting utilizes 6” recessed fluorescent down lights
- Tenant spaces are lit with 2’x4’ recessed fluorescent grid troffers
- Emergency lighting capable of operating for 90 minutes is provided in corridors and large meeting areas

Structure:
- The foundation consists of caissons drilled 25’ on average, connected with grade beams
- Main floor system is 3 1/2” concrete slab on 2” 18 gage composite decking reinforced with 6x6-W2.1xW2.1 WWF
- Steel framing composite with slab, typical beam is a W16x31, typical girder is a W24x55
- Lateral system consists of steel braced frames
- Average bay is 30’-9” x 32’-4” divided into 3 equal spans

Mechanical:
- (4) 105 ton VAV rooftop units
- 28,000cfm, 6,000cfm outdoor air intake
- VAV boxes at each floor for zone control
- 2’x2’ architectural ceiling diffusers

Electrical:
- Enters through (12) 4” conduits
- 480/277V and 208/120V
- 39 panel boards
- (6) 480V transformers, 1 per floor

Michelle Benoit — Structural Option
http://www.arche.psu.edu/thesis/eportfolio/2007/portfolios/mzb126

Executive Summary

The William W. Wilkins Professional Building is a 6 story, 112,000 sq. ft. medical office building located in Columbus, Ohio. Costing approximately $7.4 Million, it is essentially an addition to the Grant Riverside hospital across the street. These buildings are connected by a pedestrian bridge from the third floor. Enclosed by brick veneer, precast concrete and spandrel glass panels the exterior is non-load bearing.

The existing floor system is steel framing designed for composite action supported on W12 columns. Lateral framing consists of five braced frames utilizing tube steel. Two frames run North-South with the remaining three running East-West. Two of the frames running East-West are located on the exterior of the building.

The purpose of this thesis report is to consider the structural redesign of the Wilkins building to reinforced concrete skip-joists. The goals of this process are to determine if skip-joists are a feasible alternative to steel framing. Included in this is a look at the potential to open up all bays of the exterior for natural lighting by using moment connections.

Results of this study confirm that concrete skip-joists are a feasible alternative to steel framing. A 4.5″ slab with 7″x14″ joists framing into 16″x26″ girders is sufficient in most places. Pan's with a slab span of 53″ will be used creating a module size of 60″ or 5′. The new lateral system, moment connections, presents no problem due to concretes inherent properties. Columns ranging in size from 16″x16″ to 22″x22″ are sufficient to carry gravity loads while resisting lateral forces. The cost of the new system is $400,000 less than the existing system. However, construction duration is increased by 75 days.

In addition to the main depth study of this thesis two breadth studies were performed. The first was a look at integrating some form of photovoltaics into the building. Roof top photovoltaic units were designed for placement as well as wiring to integrate the units into the buildings electrical grid. The effect the units have on the environment was also investigated.

The second breadth study performed was a construction management comparison between one-way concrete slab framing methods. This included a cost and schedule comparison between concrete skip-joists and concrete beams. The use of preformed pans to place skip-joists creates a small savings in material/labor costs over beams. However, more noticeable is the considerable savings obtained in the significantly shortened construction duration. This confirms why concrete skip-joists are the preferred one-way slab framing method in industry.

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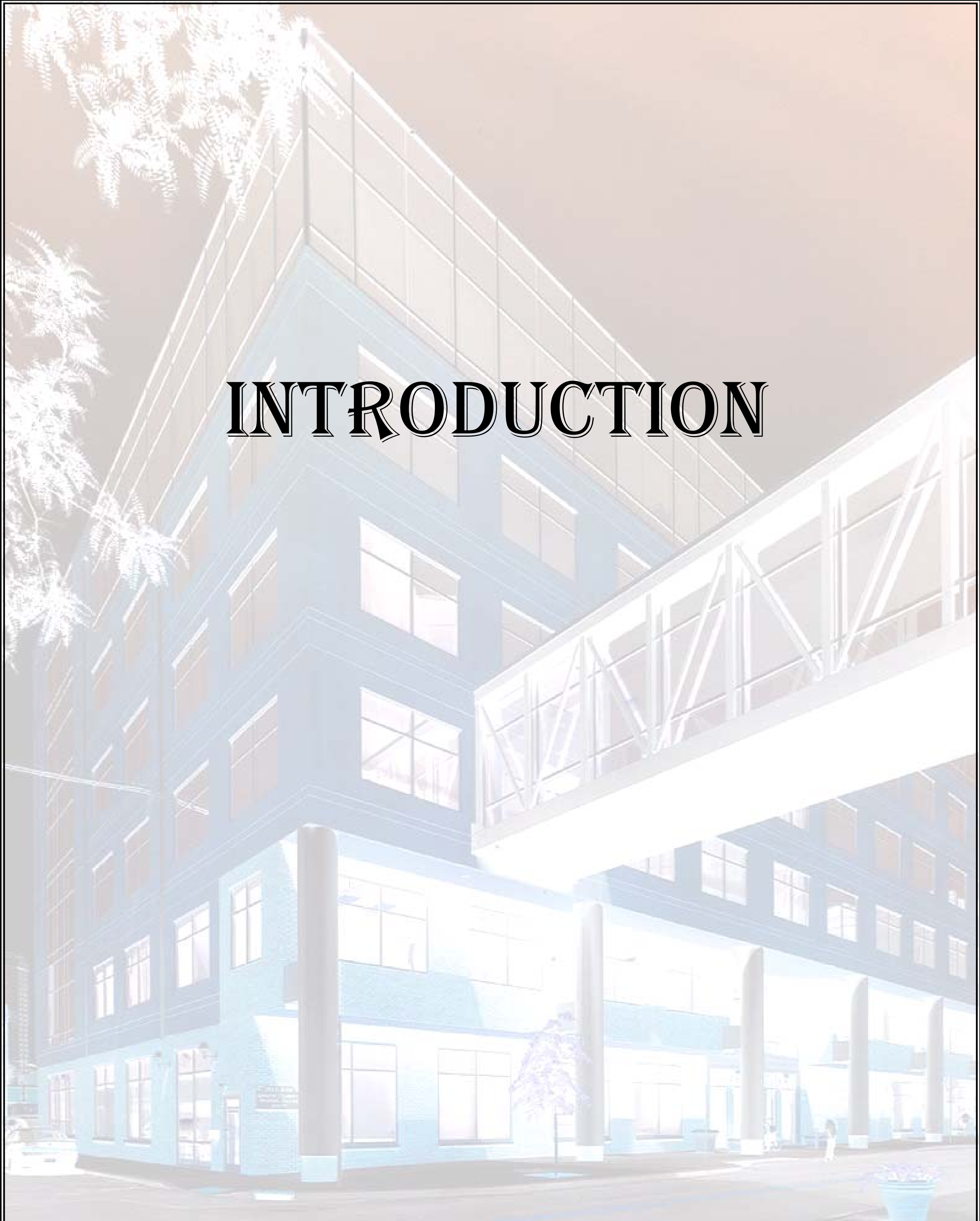
INTRODUCTION



Building Information:
The William W. Wilkins Professional Building is a six-story, 112,000 sq. ft. medical office building located on Sixth Street in downtown Columbus, Ohio. Rising 84'-8" in the air the Wilkins building is bordered by main roads on three sides. Costing approximately $7.4 million, it is an addition to the Grant Riverside hospital across Sixth street. The two buildings are connected by a pedestrian bridge from the third floor.

Project Team:
Owner: State Sixth LLC
Occupant: Grant/Riverside Methodist Hospitals
Architect/Chief Engineers: URS Corp
Geotechnical Engineer: CTL Engineering
Site Engineer: Bird & Bull
CM/GC/Developer: The Daimler Group
Demolition: Darby Creek Excavating
Steel Supplier: Ferguson Steel Co.
Concrete Supplier: Scioto Darby Concrete, Inc.

Architecture:

This six-story building is designed around a central lobby area. The main entrance off Sixth Street opens into a vestibule on the East side of the building that proceeds into the main lobby area. The lobby is furnished with subtle carpeting and ceramic tiles. The lighting scheme, recessed can lights accented with suspended chandeliers, creates a warm feeling. This contrasts the stark silver elevators located in the lobby. The floors are broken down into various medical offices with treatment areas, reception/waiting areas, exam rooms, and doctors' offices.

Building Envelope:
The main entrance into the building is located on the East face of the Wilkins building. Slightly off center, the entrance consists of clear tempered glass double doors with tinted insulated glass on either side. There is a large arch framed in stone above the doors and windows with a bottom row of spandrel glass and more tinted insulated glass above shaped to match the stone arch. There are seven columns approximately 26'-8" tall encased in metal siding supporting the upper levels. The upper levels overhang creating a covered walkway in front of the building. The underside of this overhang is encased with EIFS. The balance of the façade is a combination of precast concrete panels, brick veneer, and spandrel glass. In general, the lower portion of the façade is brick veneer, progressing into precast concrete followed by spandrel glass in the upper levels. Windows are symmetrically placed to create a horizontal delineation at each floor level.

The roof is a basic ballasted system. It is comprised of 3″ steel decking and rigid insulation covered with a Carlisle rubber EPDM membrane held down with gravel ballast.

Electrical:
The electric power for the Wilkins building enters through (12) 4″ conduits from the Division of Electricity transformer vault. Four of these conduits enter the data room for Time Warner and Ameritech service. The remaining (8) proceed to the electrical room. Located in the electrical room on each floor are six panel boards utilizing 480/277V or 208/120V. The first floor also contains (2) 208/120V emergency power panel boards located in the generator room. The generator room houses a standby natural gas generator, a main circuit breaker and an automatic transfer switch that feeds the emergency panel boards.

The main switchboard feeds the (2) 480/277V panel boards at each floor that service lighting and variable air volume boxes. From there the power is sent to a 480V delta transformer with an output of 208/120V to the remaining (4) panel boards which service the receptacles. The elevators, circulating pump and roof top units are fed directly from the main switchboard. The 20A receptacles are located along the walls with a minimum of one per wall.

Lighting:
The lighting in the Wilkins building is relatively straightforward. The ground lobby/elevator area is illuminated with 6″ recessed fluorescent down lights to give a warm inviting atmosphere. The lobby/elevator area at each successive floor retains some of this feeling with a few down lights with the majority of the light coming from 4′-0″ standard fluorescent strip lighting. The 4′-0″ strip lighting is also utilized in the data and electrical rooms on each floor.

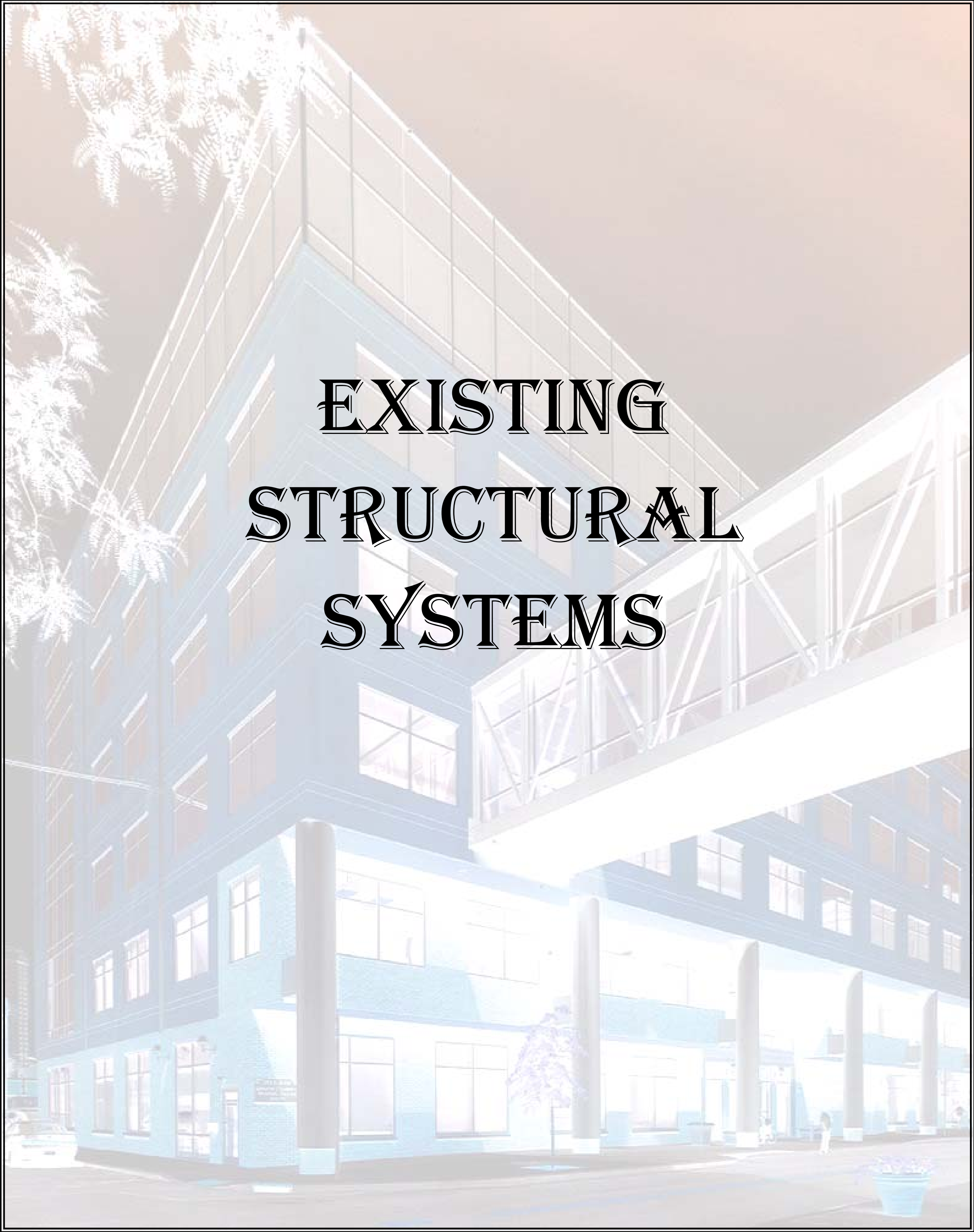
Lighting in tenant spaces is simple consisting mainly of 2′x4′ recessed fluorescent grid troffer lighting. The ground floor treatment rooms use 2′x2′ recessed fluorescent grid troffer lighting and surface mounted incandescent light fixtures. Exam rooms on the third floor also use surface mounted incandescent lights. Emergency lighting capable of operating for 90 minutes is provided in all corridors, the library, classrooms and large meeting and office areas.

Mechanical:
The primary heating, ventilating and air conditioning (HVAC) system consists of four electric nominal 105-ton variable air volume (VAV) rooftop units. Units one and four supply the fourth through sixth floors while units two and three supply the remainder of the building. Units one and two supply the South portion of the building; units three and four supply the North portion of the building. Each of the units can supply 28,000cfm with a minimum outdoor air intake of 6,000cfm. The

rooftop units are the cooling-only type and have variable frequency drives for volume control. Each unit has smoke detectors wired to shut down the units upon smoke detection as well as power exhaust and integral pressure control to maintain proper building pressurization. A small, nominal 5-ton heat pump, also located on the roof, serves the elevator machine room. This smaller unit can supply 2,000cfm with a minimum outdoor air intake of 200cfm.

Air is distributed down vertical shafts in high-pressure rectangular ductwork. The ductwork transitions to round and flat oval exiting the shafts in a manner to minimize noise. Parallel fan assisted VAV boxes and straight through VAV boxes at each floor provide individual zone control for the majority of the spaces. Boxes at the perimeter and in some interior spaces have electric heating coils to provide the source of heat for the facility. Series fan assisted boxes are employed in the lobby and entryways. Single duct boxes are used for electrical and communication closets. 2′x2′ architectural ceiling diffusers are used in the majority of spaces. The fan assisted boxes with heating coils are capable of providing after hours heating without starting the rooftop air handling units. A plenum return system is used to minimize the amount of return air ductwork. All fan assisted VAV boxes have smoke detectors to comply with the latest code specifications.

The majority of the HVAC functions are controlled through a standard direct digital control (DDC) system, which has minimal energy management functions. A portable operator's terminal (POT) enables programming changes to be made at DDC control panels.



Existing Structural Systems

Foundation:

The foundation for the William W. Wilkins Professional Building consists of reinforced concrete piers and grade beams supported on reinforced concrete caissons. See Figure 1 below. Caissons are drilled an average of 25′ to bear on sand/gravel with an allowable bearing stress of 16,000 psf. Concrete with a minimum 28 day strength of 3,000 psi was used for the caissons. Ranging in diameter from 48″ to 84″, these caissons are reinforced with #9, 10 or 11 bars with #3 or 4 ties at 12 or 18 inches. Piers and grade beams have a minimum 28 day strength of 3,500psi. On average, piers are 1′x1′ while grade beams vary from 12″×32″ to 24″ ×32″. Both are reinforced with #6, 7 or 8 bars with #3 stirrups at 12″.

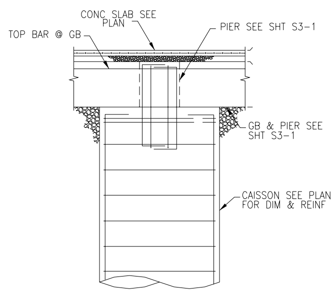


Figure 1: Typical Caisson Detail

Floor system:

The floor system in the Wilkins building is designed for composite steel-concrete behavior. Floor slabs consist of 3 ½″ normal weight concrete on 2″ 18 gage composite steel deck reinforced with W2.1xW2.1 welded wire fabric (WWF). Decking is welded to support steel. The slab on grade (SOG) varies slightly consisting of 4″ concrete on 6″ porous fill reinforced with W1.4xW1.4 WWF. Both the floor slabs and SOG are built with concrete having a minimum 28 day compressive strength of 3500 psi. A typical bay consists of W16x31 beams spanning 32′-4″ in the East-West direction framing into W24x55 girders spanning 30′-9″ in the North-South direction. ¾″ diameter by 4 ½″ long headed studs are spaced evenly along members to transfer loads. Roof framing of a typical bay uses the same size members designed as non-composite. On the East face there is a slight overhang supported by W12x14 beams framing into W16x26 girders. Moment connections are used where beams connect to columns and girders. A typical framing plan is shown below in Figure 2.

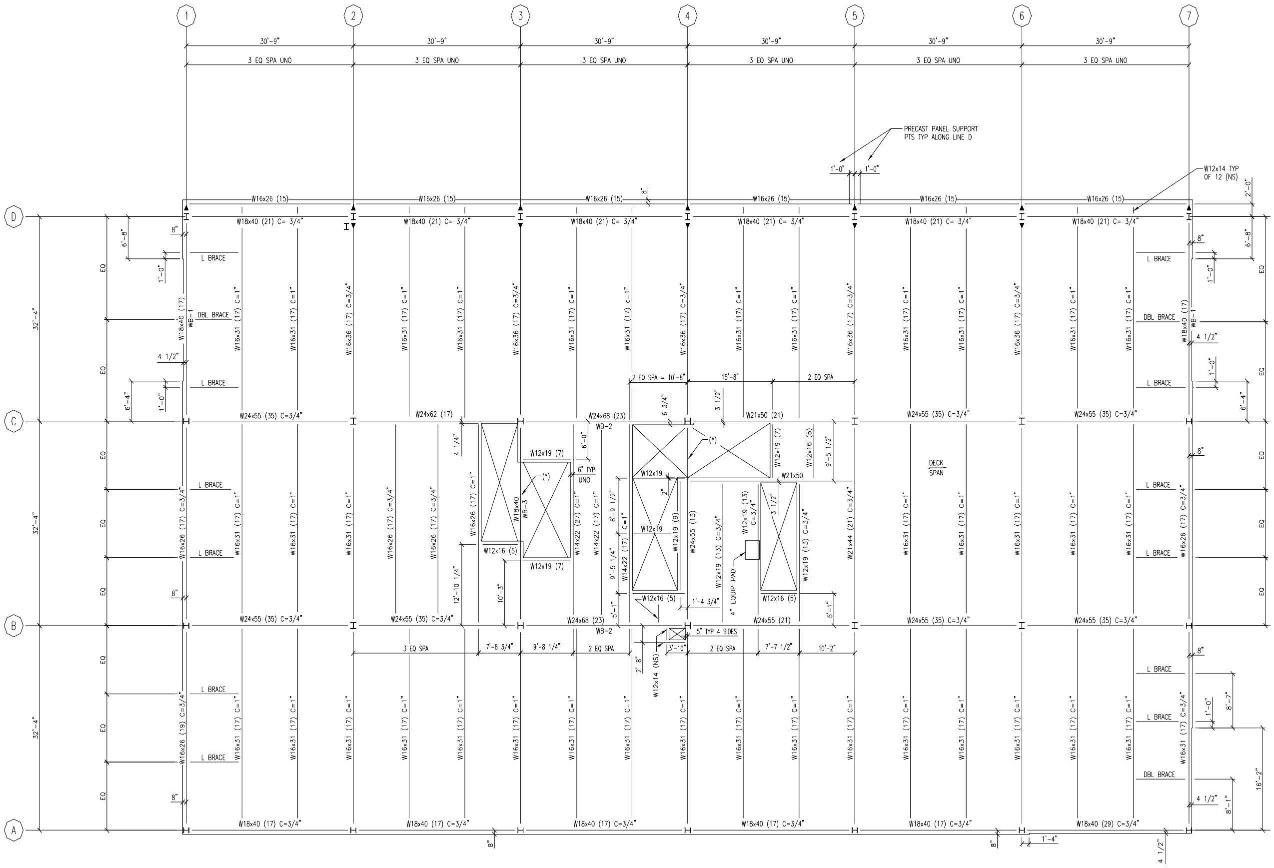


Figure 2: Typical Floor Plan

Columns:

Columns are ASTM 992 Grade 50 rolled W12 steel shapes with splices on the third and fifth floors. Splice connections use welds and ¾″ diameter A325 bolts. Web bolts are slip critical, to connect plates. (See Figure 3 below). The largest columns are W12x136 and are part of the lateral system. Gravity columns range from W12x40s at the roof level to a maximum size of W12x106 at ground level. Base plates are either 18x18 or 20x20 with thicknesses ranging from 1 ¾″ to 2 ½″. Connections consist of (4) anchor bolts of varying sizes.

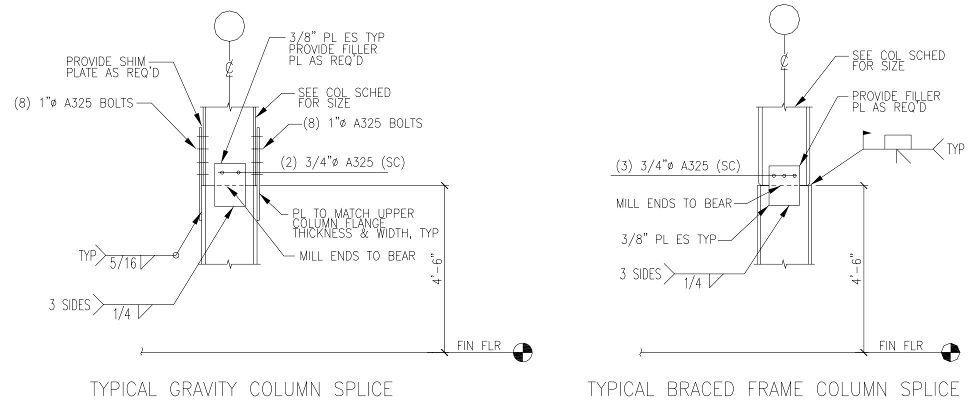


Figure 3: Typical Column Splice Details

Lateral System:

Lateral loads are resisted in the Wilkins building using concentric braced frames. Two frames spanning North-South are located near the elevator shafts. Frames spanning East-West are split with one by the elevator shafts, one on the exterior South-East bay and one on the exterior North-East bay. Lateral bracing in these frames are ASTM A500 Grade B tubes ranging in size from HSS5x5x.1875 to HSS8x8x.25. A typical braced frame is shown in Figure 4 below. The tube steel is welded to gusset plates that connect to main framing members.

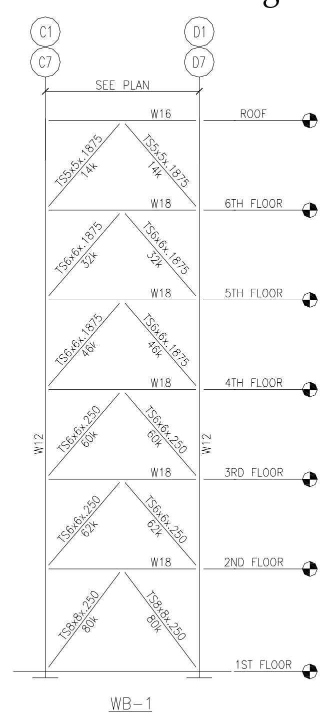
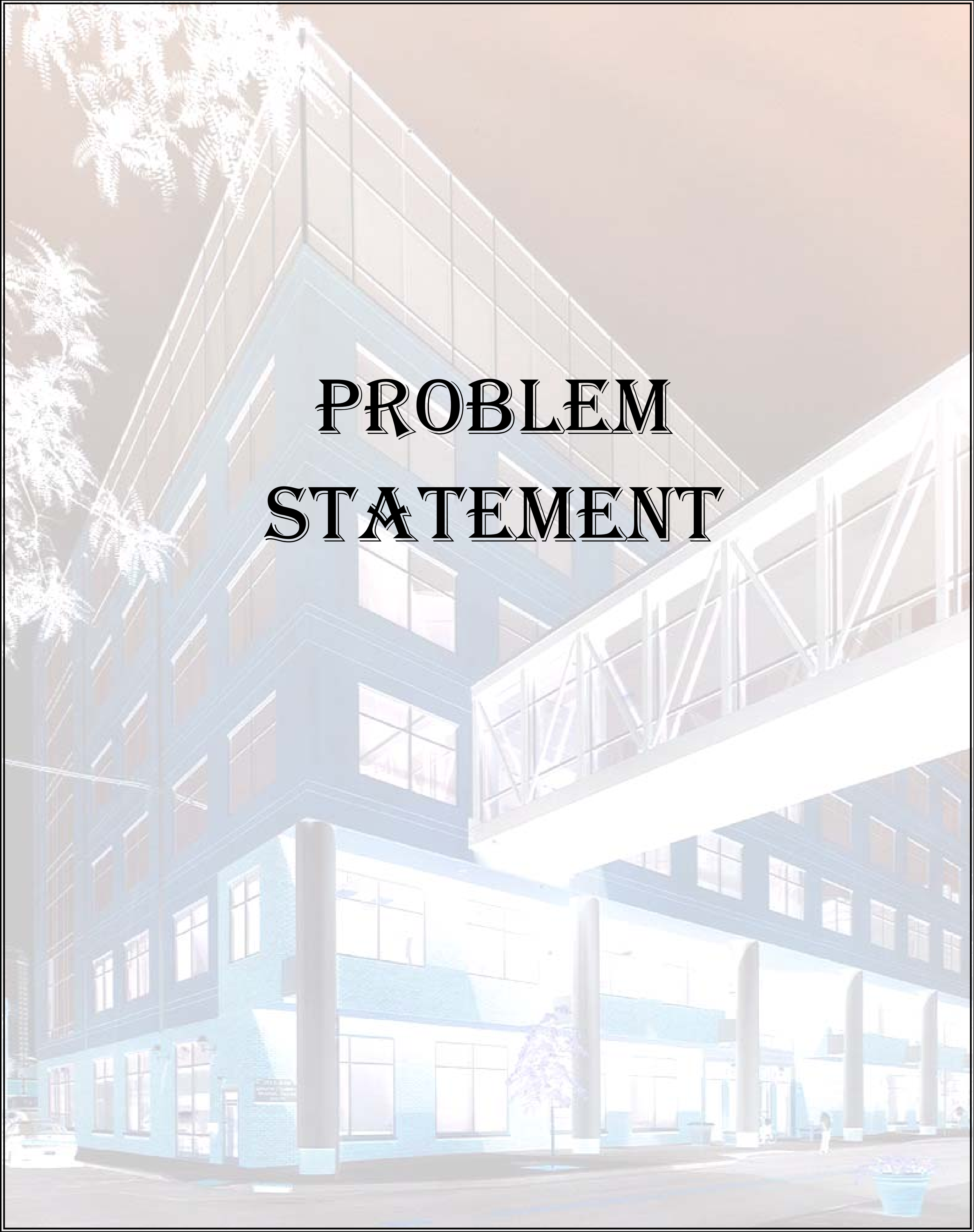


Figure 4: Typical Braced Frame

PROBLEM STATEMENT

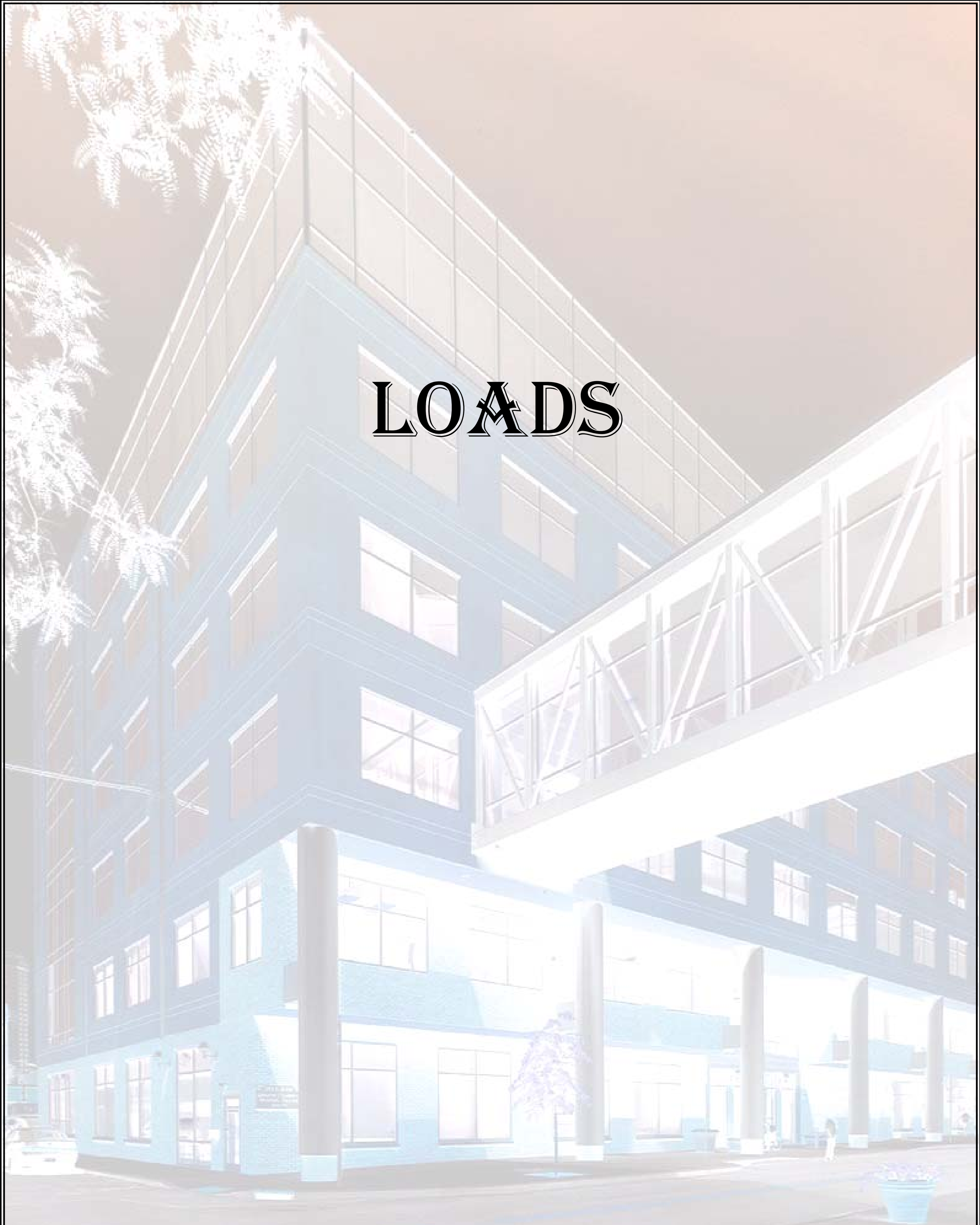


The floor system used in the Wilkins building may or may not be the most efficient and cost effective system. A different system will be investigated for its incorporation into the Wilkins building. Both the integration into the floor framing and the effects the systems have on the lateral system will be looked at.

The current lateral system used in the Wilkins building consists of concentric braced frames. As noted in the Existing Structural Systems section, this system includes braced frames located in two exterior bays. These frames reduce the exterior wall space available for windows. This decrease in windows lessens the amount of natural lighting available, which may adversely affect worker productivity. Furthermore, changing the structural system will alter the weight of the structure, which in turn changes the seismic forces. For these reasons, a new lateral system will be investigated.

Changing the floor and lateral systems will alter the weight of the structure. All concrete floor systems investigated in Technical Assignment #2 induced a greater weight than the existing composite structure. As a result, the foundation will potentially need to be redesigned as well.

LOADS



General:

	Loads(psf)
Live:	
Office floor	50
Corridors	80
Lobbies & Stairs	100
Roof	20
Snow	20
Dead:	
Floor	103
Partitions	20
Ceiling	2
MEP	5
Misc.	2
Total:	132
Roof Dead:	102

Table 1: Gravity Loads

Wind:

$P=qGCp - q_i(GCp_i)$						
	Windward		Leeward		Total	
height	N-S	E-W	N-S	E-W	N-S	E-W
0-15'	6.83	6.83	-4.68	-7.04	11.51	13.87
20'	7.43	7.43	-4.68	-7.04	12.11	14.47
25'	7.91	7.91	-4.68	-7.04	12.59	14.95
30'	8.39	8.39	-4.68	-7.04	13.07	15.43
40'	9.11	9.11	-4.68	-7.04	13.78	16.15
50'	9.71	9.71	-4.68	-7.04	14.38	16.75
60'	10.19	10.19	-4.68	-7.04	14.86	17.23
70'	10.67	10.67	-4.68	-7.04	15.34	17.71
80'	11.15	11.15	-4.68	-7.04	15.82	18.19
84.67	11.27	11.27	-4.68	-7.04	15.94	18.31

Table 2: Wind Loads

For complete calculations, see Appendix.

Seismic:

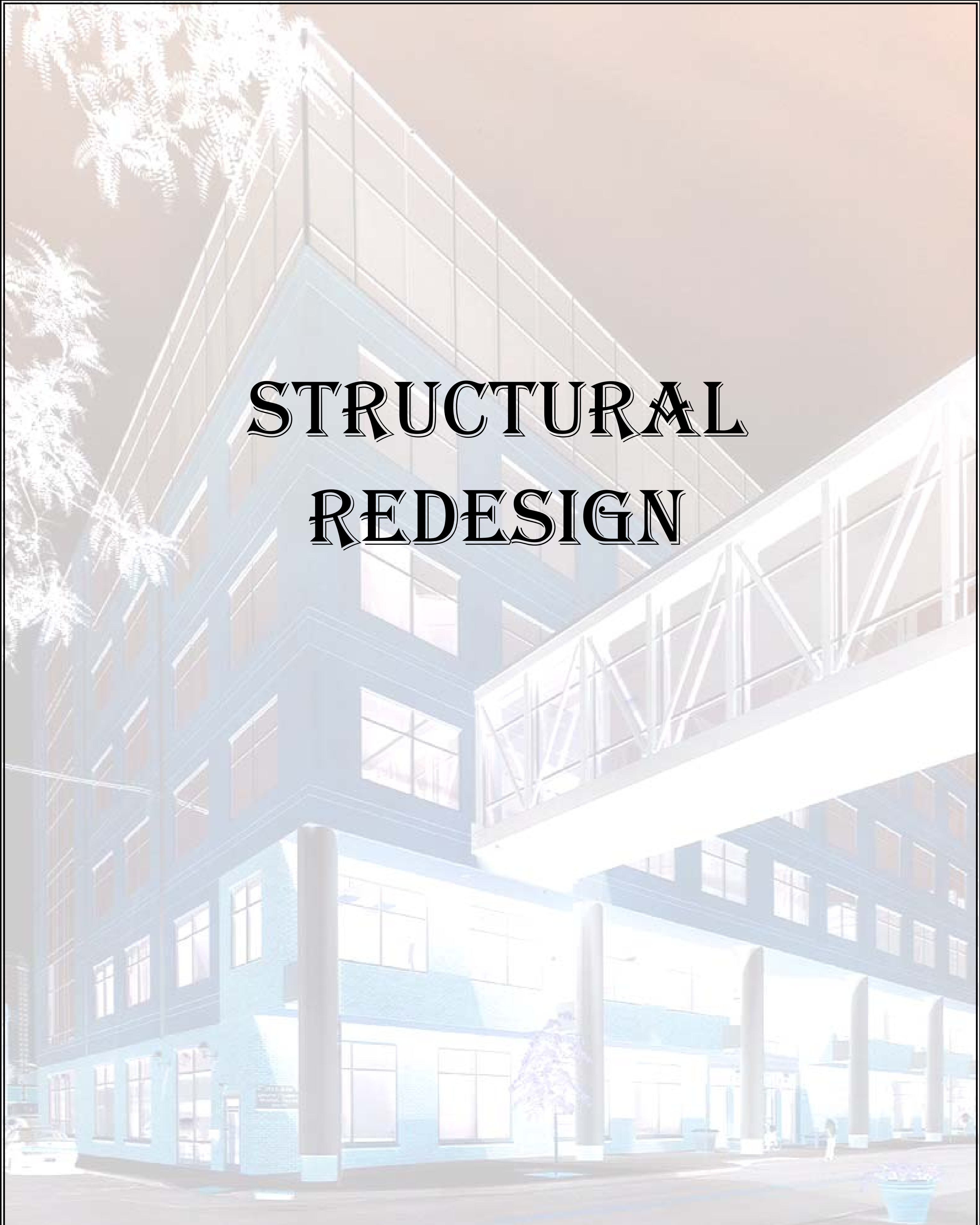
Story	F_x(k)
2	9.53
3	20.29
4	31.70
5	43.66
6	56.16
R	53.07

Table 3: Seismic Loads

For complete calculations, see Appendix.

Based on ASCE 7 the applicable load cases are:

1.4D	(LC 1)
1.2D + 1.6L +0.5S	(LC 2)
1.2D + 1.6S + (0.5L or 0.8W)	(LC 3)
1.2D + 1.6W + 0.5L + 0.5S	(LC 4)
1.2D ± 1.0E + 0.5L + 0.2S	(LC 5)
0.9D ± (1.6W or 1.0E)	(LC 6)



Structural Redesign

The alternate structural system evaluated is a one-way slab with reinforced concrete skip-joists. The alternate lateral system investigated is reinforced concrete moment frames. Skip-joists are also referred to as wide-module joists. The advantage to this system over a one-way slab with beams arises in the forms. Pan forms are available, as rentals or to purchase, in standard slab span and joist sizes. The most common forms come in 53″ and 66″ widths. Used with joist widths of 7″ and 6″ respectively, 5′ and 6′ modules are produced. Typical slab thickness is 4.5″ designed as a one-way slab. Joists are designed as beams that conform to ACI requirements for reinforcement. Concretes' properties make moment frames a natural choice for lateral force resistance. An example of a floor system using wide-module frames from Ceco Concrete, a form supplier, is shown in Figure 5 below. The chosen forms are Ceco's Wide Flange forms. These are offered in both 53″ and 66″ widths. Figure 6 shows a potential pan set up.

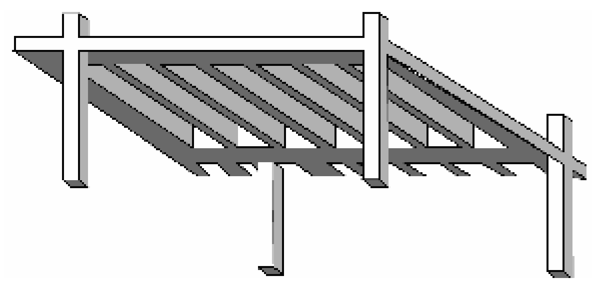


Figure 5: Finished Bay Using the Wide-Module Frame System

Figure 6: Wide-Module Frame System

Flexural:

Before beginning an in-depth design of the joists, the CRSI design handbook was consulted for initial joist and pan module sizes. These initial sizes were determined based on loading, clear span, and deflection limits. Deflection limitations of thickness ≥ $l_n/18.5$ for end spans and $l_n/21$ for interior spans were used. From CRSI a 53″ form with 7″ ribs, 16″ deep with a 4.5″ top slab would be suitable for the typical loading criteria. To check the flexural design of the slab the following equations from ACI 318 were used:

Slab Design:

Minimum thickness = $l_n/24$ (Eq. 1)
$M_u = wl_n^2/10$ (Eq. 2)
$A_{smin} = 0.0018bd$ (Eq. 3)
$a = A_s f_y / 0.85f'_c b$ (Eq. 4)
$A_s = M_u/[\text{Ø} f_y(d-a/2)]$ (Eq. 5)
$\text{Ø}M_n = \text{Ø}A_s f_y(d-a/2)$ (Eq. 6)

On the conservative side, a clear span of 66" was assumed for use in Eq. 1. This gave a thickness of 2.75" making a 4.5" thick slab sufficient.

As the William W. Wilkins building is a medical office building an open floor plan is essential. In designing the new floor system, a live load of 80psf was assumed everywhere. This creates over design is some portions while providing the flexibility of floor plan needed as tenants change.

For flexure, the controlling load cases are LC 2 and LC 3. Accordingly, a moment of $0.7'^k$ was found from Eq. 2. After solving Eq. 3-5 it was determined the slab should be reinforced with #4's @ 18". This gives a moment capacity of $20'^k$, which is far greater than the actual moment of $0.7'^k$.

The following equations from ACI 318 were used to check the flexural design of the joists and girders:

Joist/Girder Design:

$M_u = wl_n^2/X$ (Eq. 7)
where X is the appropriate coefficient from ACI 318-8.3.3
Equations 4 thru 6 are used here as well.

As noted above in Eq. 6, ACI coefficients were used to determine the design moments of various joists and girders. The façade of the building is supported at each floor. This means the East and West exterior joists and the North and South exterior girders support the façade as well as floor loads. For this reason, several different joists and girders were looked at. Figure 7 below shows the various locations of joists and girders analyzed.

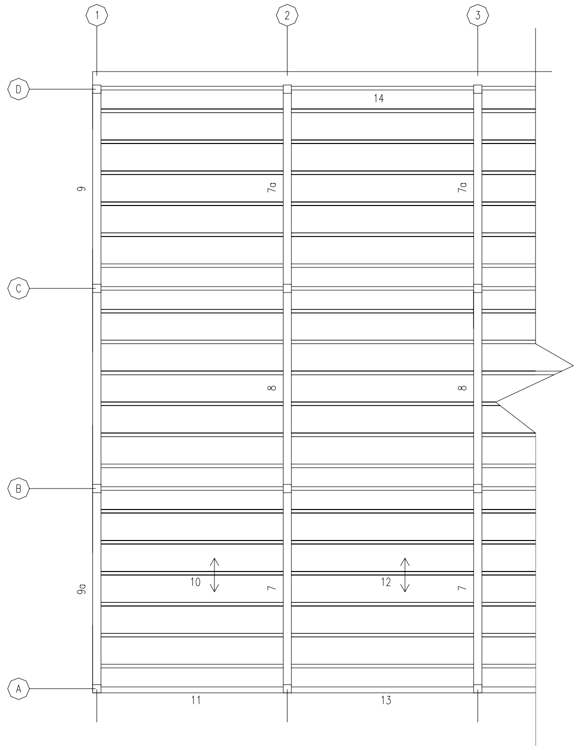


Figure 7: Joist and Girder Designations

After assuming an (a) value and calculating Eq. 7, Eq. 5 was used to calculate an approximate area of steel. Using this value (a) was back figured through Eq. 4, thus beginning an iteration process to refine the value of A_s. From here, Eq. 6 was used to design joists 10-14 and girders 7-9. Reinforcement details are given in Tables 4 and 5 below.

Joist Reinforcement Schedule						
	10		11		12	
	Floor	Roof	Floor	Roof	Floor	Roof
M^+	(2) #7	(2) #5	(2) #8	(2) #5	(2) #7	(2) #5
M^- left	(2) #7	(2) #5	(2) #8	(2) #5	(2) #8	(1) #6, (1) #7
M^- right	(2) #8	(1) #6, (1) #7	(2) #10	(1) #6, (1) #7	(2) #8	(1) #6, (1) #7

	13		14	
	Floor	Roof	Floor	Roof
M^+	(2) #8	(2) #5	(2) #10	(2) #5
M^- left	(2) #10	(2) #5	(2) #8	(1) #6, (1) #7
M^- right	(2) #10	(1) #6, (1) #7	(2) #10	(1) #6, (1) #7

Table 4: Joist Reinforcement Schedule

Girder Reinforcement Schedule						
	7		8		9	
	Floor	Roof	Floor	Roof	Floor	Roof
M^+	(6) #10	(4) #9	(3) #10, (2) #9	(4) #9	(4) #9	(3) #8
M^- left	(10) #10	(2) #10, (3) #9	(8) #10	(2) #10, (3) #9	(4) #9	(3) #8
M^- right	(3) #10, (2) #9	(4) #8	(8) #10	(2) #10, (3) #9	(3) #10, (2) #9	(3) #9

Table 5: Girder Reinforcement Schedule

<u>Note:</u>
7a & 9a are the opposite of 7 & 9. I.e. M^- left on 7 is M^- right on 7a and M^- right on 7 is M^- left on 7a. The same applies to 9 and 9a.

From the above described calculations it was determined that a 53" form with 7" joists was sufficient in the majority of locations. However, the exterior joists needed to be slightly larger due to the added weight of the façade. On the East, the joists are 8" whereas on the West they are 10". Girders were determined to be 16"x26" at all locations. If designed as t-beam sections the joist depth can be reduced from the assumed 16" depth to 14". However, to meet deflection criterion the exterior bays must remain at a depth of 16". Figure 8 is a typical floor plan layout with the new reinforced concrete skip-joist system.

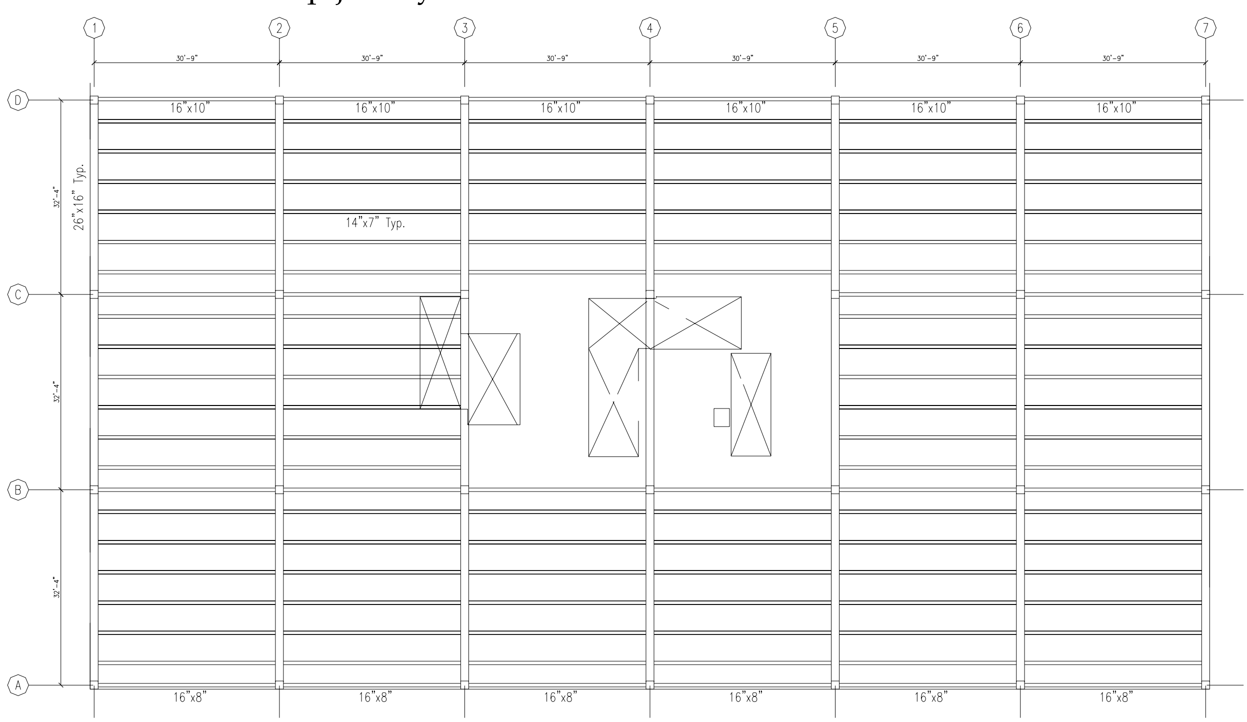


Figure 8: Typical Skip-Joist Floor Plan

Table 6 summarizes the joist and girder sizes. Members are designated as in Figure 7 on page 16.

Member	Size (in.)
7	16x26
8	16x26
9	16x26
10	7x16
11	8x16
12	7x14
13	8x14
14	10x14

Table 6: Member Sizes

Development lengths of the various reinforcement bar sizes were determined using the following equations from ACI 318.

$$l_d = f_y \lambda \psi \varphi d_b / (25\sqrt{f'_c}) \qquad \text{(Eq. 8)}$$
$$l_d = f_y \lambda \psi \varphi d_b / (20\sqrt{f'_c}) \qquad \text{(Eq. 9)}$$

where $\psi = 1.0$
$\varphi = 1.0$ for positive reinforcement and 1.3 for negative reinforcement
$\lambda = 1.0$

Eq. 8 is used for #6 and smaller bars. For #7 and larger bars Eq. 9 is to be used. Based on these values the following development lengths were obtained:

	Diameter	Negative l_d (in.)	Positive l_d (in.)
#5	0.625	30.81	23.72
#6	0.75	36.98	28.46
#7	0.875	53.46	41.13
#8	1	61.10	47.00
#9	1.128	68.92	53.02
#10	1.27	77.60	59.69

Table 7: Development Lengths

Reinforcement is not usually continued along the entire length of the member. Large savings can be had from terminating reinforcement bars where no longer needed. According to ACI 318, for positive moment reinforcement 1/3 of the reinforcement must extend at least 6" into the support. The remainder of the reinforcement must extend the full development length past the point of maximum moment. In most cases, this is the center of the joist. Negative moment reinforcement termination is slightly more difficult. It has been noted by many designers that a safe cutoff point for negative reinforcement is 1/3 the clear span past the support. For a typical joist, this means negative reinforcing bars must be 236″ or 19′-8″ long, one positive bar will be l_d and one will be 360″ long.

Shear:

From ASCE 7-05 table 12.2-1 the response modification factor (R), used in calculating seismic forces, for concrete moment frames varies from 3 to 8. For an ordinary concrete moment frame, the R-value is 3. This produced a base shear of 371^k. If an intermediate concrete moment frame was used the R-value increases to 5. The base shear for this combination became 214^k, a significant reduction from using an ordinary concrete moment frame. For this reason, intermediate reinforced concrete moment frames are considered. To use intermediate frames stricter detailing requirements for shear are required.

Detailing requirements for beams from ACI 318 are listed below:

$S \leq d/4$
$\leq$ 8 times the diameter of vertical reinforcement bars
$\leq$ 24 times the diameter of hoop bars
$\leq$ 12″

Further requirements are $S \leq d/2$ over the full length of the member and first hoop placed not farther than S/2 from the support.

The joists and girders were designed based on these requirements and using several equations from ACI 318 listed below.

$$ØV_c = Ø2bd\sqrt{f_c} \quad \text{(Eq. 10)}$$
$$V_u = wl_n/2 \quad \text{(Eq. 11)}$$
$$ØV_s = V_u - ØV_c \quad \text{(Eq. 12)}$$
$$S = ØA_vf_yd/ØV_s \quad \text{(Eq. 13)}$$
$$ØV_n = ØV_c + (ØA_vf_yd)/S \quad \text{(Eq. 14)}$$

It was found, in most cases, the spacing required by Eq. 13 was significantly greater than d/2. This being the case the spacing requirements listed for intermediate moment frames were used. Eq. 14 was used to determine the shear values where different spacings could be used. This was done to save on reinforcing hoops by increasing the spacing where possible.

Table 8 below gives a summary of the shear reinforcement requirements.

Shear Reinforcement			
	Size	# and spacing (in.), ends	Spacing for middle (in.)
Floor Joist	# 3	(1) @ 2, (6) @ 4	9
Roof Joist	# 3	(1) @ 2, (6) @ 4	9
Ext. Floor Joist	# 3	(1) @ 2, (6) @ 4	9
Floor Girder	# 4	(1) @ 2, (16) @ 5, (9) @ 9	11
Roof Girder/Ext. Floor Girder	# 4	(1) @ 2, (10) @ 5, (7) @ 8	11
Ext. Roof Girder	# 4	(1) @ 2, (10) @ 5	11

Table 8: Shear Reinforcement Schedule

For the slab, shear capacity was checked using Eq. 10 and the following:

$$V_u = 1.15wl_n/2 \qquad \text{(Eq. 15)}$$

It was determined the shear capacity was far greater than the design shear.

Columns:

An initial column size was determined based solely on gravity loads. The total live load is smaller than 75% of the dead load. This means the maximum moments are due to full live plus dead load eliminating the need for pattern loading. Neglecting unbalanced moments and using Eq. 16 given below it was determined that 16x16 columns would be sufficient.

$$P_o = 0.85f'_c(A_g - A_s) + A_sf_y \qquad \text{(Eq. 16)}$$

The next step in designing the columns was to consider the unbalanced moments at different locations. Figure 9 below designates the critical columns looked at.

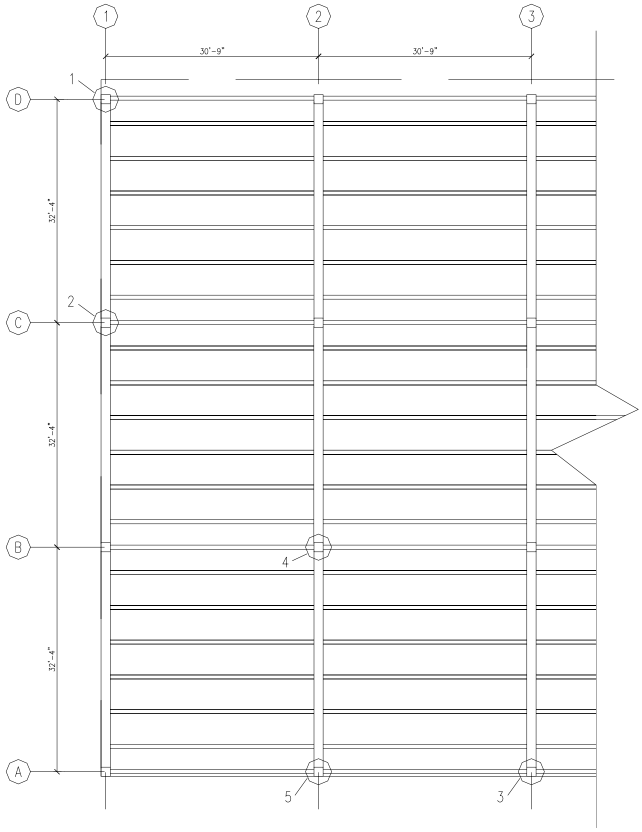


Figure 9: Column Designations

Column interaction diagrams were constructed for 16", 20" and 22" square columns. The cumulative loads and moments at each floor level was calculated and plotted for each of the five critical columns. Figure 10 shows a typical interaction diagram.

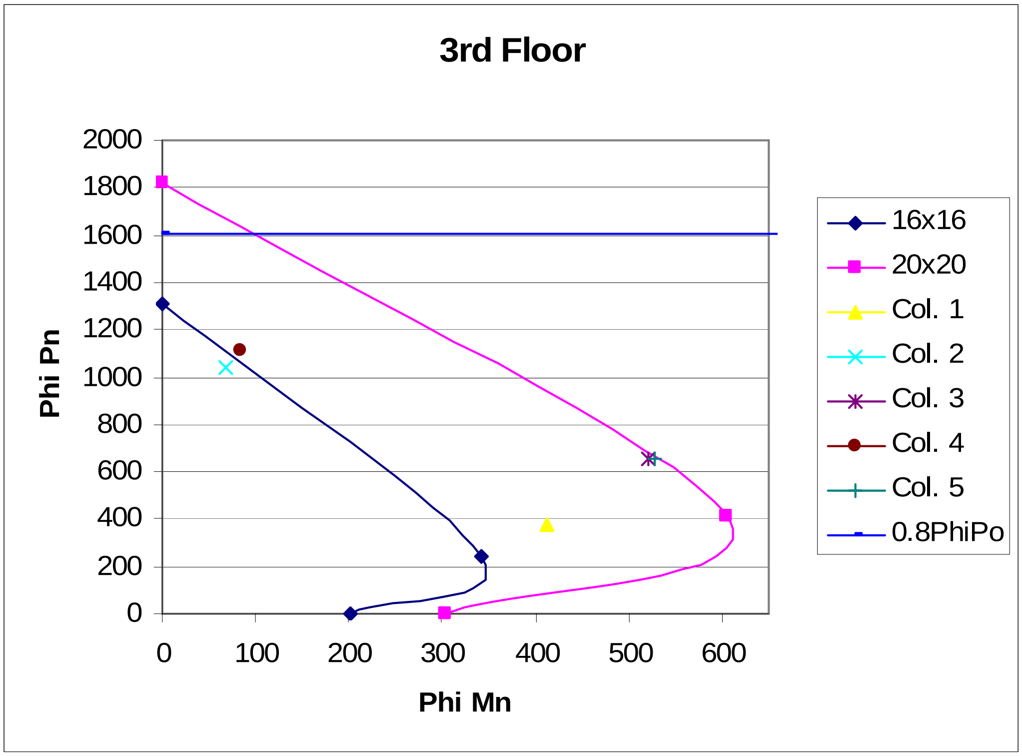


Figure 10: Column Interaction Diagram

From these diagrams, it was determined that 16", 20" and 22" square columns would be sufficient at every location. Table 9 gives the final column sizes based on gravity loads. Figure 11 shows typical reinforcing details.

Column Schedule					
	1	2	3	4	5
6th Floor	16x16	16x16	16x16	16x16	16x16
5th Floor	20x20	16x16	20x20	16x16	20x20
4th Floor	20x20	16x16	20x20	16x16	20x20
3rd Floor	20x20	20x20	20x20	20x20	20x20
2nd Floor	22x22	22x22	22x22	22x22	22x22
1st Floor	22x22	22x22	22x22	22x22	22x22

Table 9: Column Schedule

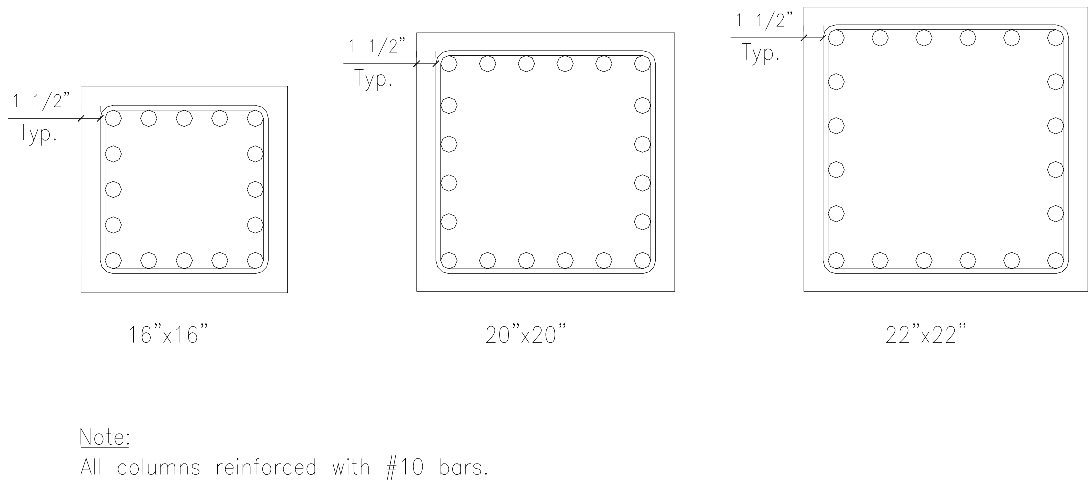


Figure 11: Column Details

To maintain the ability to use an R-value of 5 in calculating seismic loads, stricter shear detailing requirements were needed for the columns as well. These are listed below:

$S_o \leq b/2$
$\leq$ 8 times the diameter of vertical reinforcement bars
$\leq$ 24 times the diameter of hoop bars
$\leq$ 12"

over a length l_o defined below

$l_o \leq l_n/6$
$\leq b$
$\leq$ 18"

After l_o
$S \leq b$
$\leq$ 16 times the diameter of vertical reinforcement bars
$\leq$ 48 times the diameter of hoop bars

Further requirements are $S \leq d/2$ over the full length of the member and first hoop placed not farther than S_o /2 from the bottom/top of the column.

Based on these requirements Table 10 below gives a summary of the shear reinforcement required for the various column sizes used.

Column Shear Reinforcement						
	Size	S_o	l_o	After l_o	d/2	Spacing used (in)
16x16	# 3	8	16	16	6.75	6
20x20	# 3	9	18	18	8.75	8
22x22	# 3	9	18	18	9.75	9

Table 10: Column Shear Reinforcement Schedule

The main lateral force resisting elements are the columns. This is in the form of shear resistance. As these columns are subjected to flexure, bending, and axial compression, the shear capacity was calculated using Eq. 17.

$$ØV_c = Ø2bd\sqrt{f_c}[1 + N_u/(2000A_g)] \qquad \text{(Eq. 17)}$$

As shear capacity is a function of the column size, the 16" square columns will have the lowest shear strength. Eq. 17 then simplifies to

$$ØV_c = 17{,}759.4 + 0.0347N_u \qquad \text{in lbs.}$$

Based on the relative stiffness of columns 1-5 at each floor the lateral distribution of seismic and wind forces was determined. Table 11 below shows the lateral force distribution in the controlling column. It can be seen that the shear force generated from these loads is significantly lower than the 17.76^k available if N_u is neglected. Considering N_u will only increase the shear capacity making this a conservative approach.

5th Floor Columns 20x20		
	N/S (k)	E/W (k)
W	1.83	3.86
E	2.85	2.85

Table 11: Controlling Column Lateral Force Distribution

Thus, the column sizes designated in Table 9 are sufficient for both lateral and gravity loads.

Foundations:

The existing foundations for the Wilkins building are not sufficient for the increased loading from the new reinforced concrete system. For this reason, a new series of reinforced concrete caissons was designed. The following equations were used:

$$q \geq P/A \qquad \text{(Eq. 18)}$$
$$A_s = 0.0018A \qquad \text{(Eq. 19)}$$

From the existing geotechnical report the bearing capacity, q, is 16ksf. With varying loading cases the new caisson designs require diameters ranging from 7′ to 12′. This creates a large increase in cost from the original caissons which range from 4′-7′. To help reduce the costs belled caissons were considered. Reducing the shaft diameter to a consistent 4′ diameter and belling the bottom to the required diameter creates a savings of approximately $200,000.

Reinforcement for caissons was determined from minimum area of steel requirements. Critical column locations are shown below in a reproduced version of Figure 9. These correspond to the caisson details in Table 12. Shear reinforcement consists of #3 spiral reinforcing.

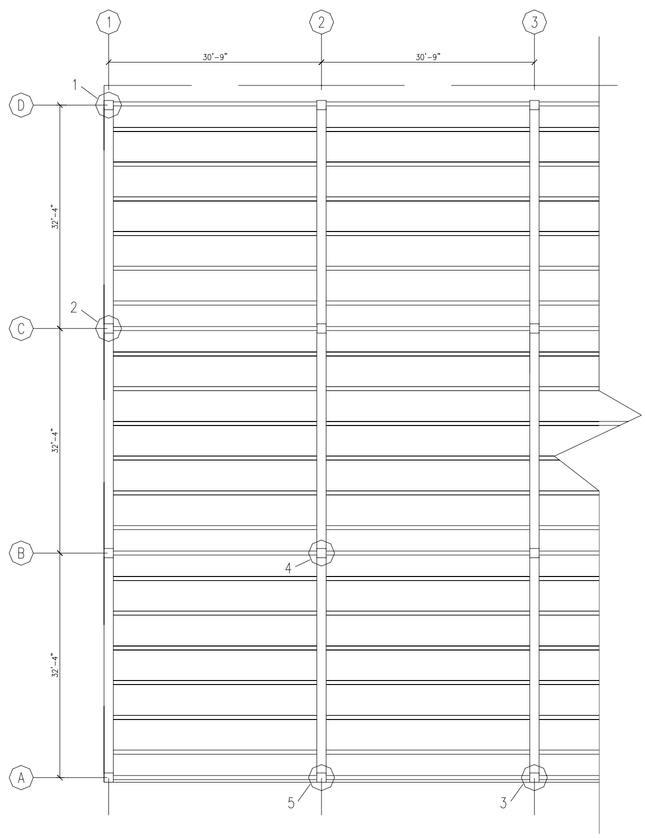


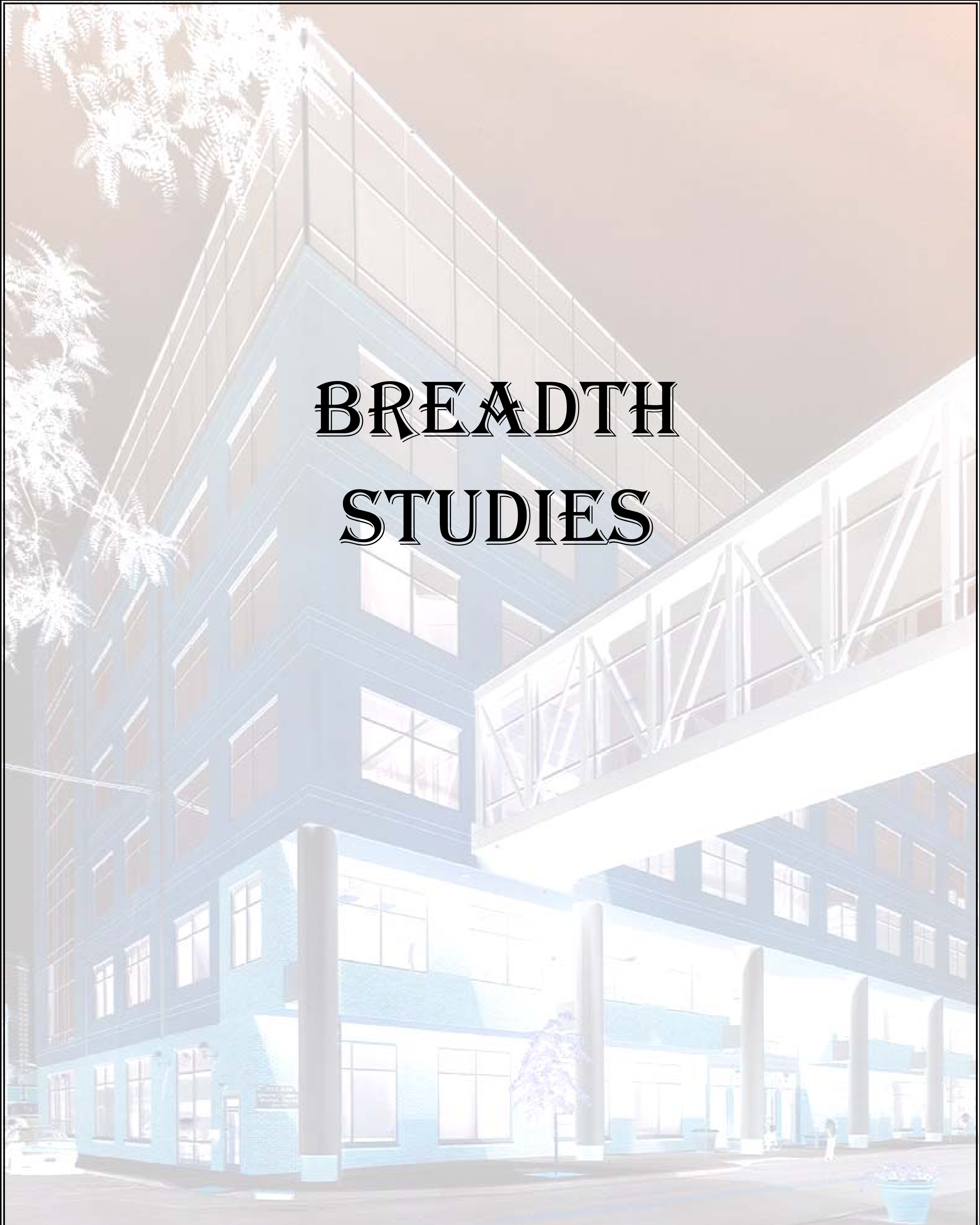
Figure 9: Column Designations

Caisson Schedule			
	Diameter (ft)	Taper Length (ft)	Reinforcement
1	7	6	(10) #9
2	11.5	10	(13) #10
3	9	15	(18) #11
4	12	16	(20) #11
5	9	15	(18) #11

Table 12: Caisson Schedule

In calculating the taper length, a 2:1 slope was considered. The reinforcement details listed in Table 12 are for the belled portion of each caisson. With a shaft diameter of 4’ far less reinforcement is required. Shaft reinforcement consists of (5) #8 bars.

A further study was conducted on potential savings had the existing foundation system used belled caissons. A relative value for each option, straight shaft vs. belled, as obtained from ICE Estimating was obtained. The existing straight shaft caissons have a cost of $101,762. Belled caissons would have a cost of $46,296. This creates a substantial savings of $55,466.



BREADTH STUDIES

Photovoltaic Design:

One of the most prevalent approaches to making a building "green" includes the use of photovoltaic (PV) cells. PV cells can be utilized either as panels on the roof or as a "skin" attached to the façade. There are various approaches to using the façade of a building to collect solar energy. One such approach was developed by Conserval Engineering Inc. Their product, Solar Wall PV/T, combines the effects of PV's with thermal panels. The thermal panels cool the PV panels by removing the "waste" heat and using it for practical heating purposes. This increases the efficiency of the system while decreasing greenhouse gas emissions.

Atlantis Energy has created similar solar products. The first, SUNSLATES, are electric tiles that may be attached to the roof or walls of a building. These slates create an aesthetic feel by reproducing the look of slate yet generating 10.07W/sq. ft. The next product is a building integrated photovoltaic (BIPV). Atlantis produces BIPV's as glazing units.

If PVs are to be applied to the roof, the most common approach is to use panels. One of the main producers of PV roof panels is BP Solar. They offer products manufactured with either multi-crystalline or monocrystalline solar cells. Monocrystalline panels are slightly more expensive yet offer a higher efficiency than multi-crystalline panels.

After considering the location of the Wilkins building, it was decided to use roof-mounted PV panels. In a downtown location, PVs on the façade would not receive enough sunlight to generate enough electricity to offset the costs. The solar panel selected is BP Solars' 4175 model shown in Figure 12 below.

Figure 12: PV Panel

The BP 4175 uses the higher efficiency monocrystalline cells. Panel construction consists of a 3mm tempered glass panel, 72 cells in a 6x12 matrix connected in series, with a tedlar backing surface. Figure 13 lists the electrical characteristics of the panel.

Electrical Characteristics[2]	BP 4175
Maximum power (P_{max})[3]	175W
Voltage at Pmax (V_{mp})	35.7V
Current at Pmax (I_{mp})	4.9A
Warranted minimum P_{max}	166.5W
Short-circuit current (I_{sc})	5.4A
Open-circuit voltage (V_{oc})	44.0V
Temperature coefficient of I_{sc}	(0.065±0.015)%/ °C
Temperature coefficient of V_{oc}	-(160±10)mV/°C
Temperature coefficient of power	-(0.5±0.05)%/ °C
NOCT (Air 20°C; Sun 0.8kW/m^2; wind 1m/s)	47±2°C
Maximum series fuse rating	15A (S, L)
Maximum system voltage	600V (U.S. NEC & IEC 61215 rating)

Figure 13: PV Panel Electrical Characteristics

To determine the number and placement of panels on the roof, consideration must be given to the shadows cast by the parapet, roof top air handling units, penthouse, and the surrounding panels. The parapet casts a shadow 6' long so the first row of panels should not be placed closer than that. Oriented so they face south, the panels can be placed flat or inclined. To gain the full potential of each panel they will be angled at the latitude of Columbus, Ohio, approximately 40°. For simplicity of calculations, a height of 3' to match the height of the parapet was assumed. This then creates a shadow 6' long as well. Based on this, the panels should be placed as shown in Figure 14 below. To gain extra space panels were placed on the roof top units as well as the penthouse.

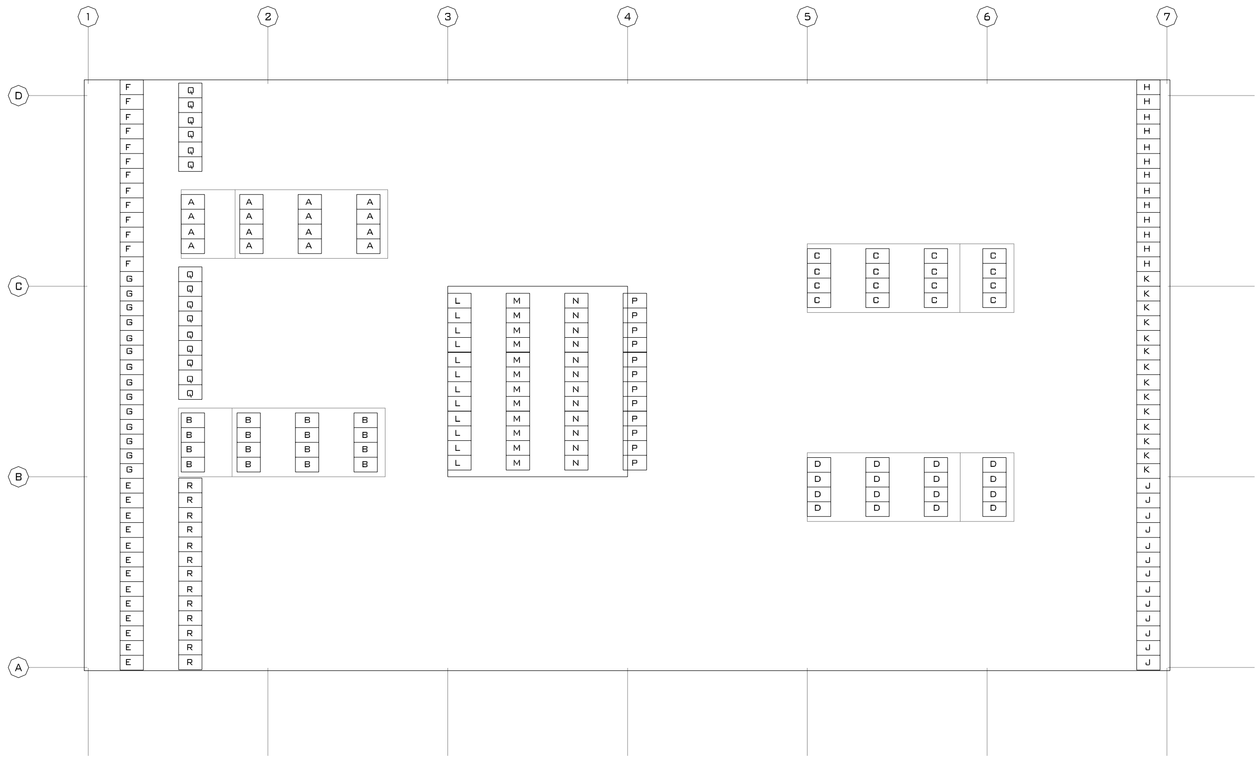


Figure 14: PV Panel Layout

This layout was chosen for several reasons. The shadow cast by the roof top units and the penthouse allow insufficient space to place any panels in the middle of the roof. Panels could be placed along either side of the roof, i.e. column lines A and D. These panels, however, are wired in series so they essentially act as resistors. If one panel does not see sun it resists the rest of the panels connected to it from collecting energy. Even if wired only to each other, these panels would only collect energy for half of the day. The gain from this is not likely to offset the cost of the PVs.

Integrating the energy collected from the panels into the building's electrical grid is relatively simple. A Sunny Boy Grid-tie Inverter can be used. The product chosen, the SB3800U (shown in Figure 15 below), allows up to 16 panels to be connected to each inverter.


Figure 15: Sunny Boy Inverter

The SB3800U is a 208V single-phase device that can supply up to 3500W. Sixteen inverters are required to accommodate the 220 panels on the roof. The distribution of the panels to inverters is designated in Figure 12 above; each letter (A-H, J-N, and P-R) goes to an inverter. Each panel will be connected to its respective inverter with #12AWG wire. The inverter connects to the panel boards with #12AWG wire as well. Panel boards 6AA and 6B are both open, so 8 inverters will be placed on each. From the panel boards, the electricity will be sent through transformer #6 to panel LP6 and back to the main distribution room.

Assuming each panel collects for 6hrs/day, 175W per panel, with an inverter efficiency of 95%, the total output would be 220 KWH/day. This is a savings of almost $20/day or $7,300/yr. For a building the size of the Wilkins building, this will not be a huge savings. However, this will also prevent roughly 100,000lbs/yr of carbon dioxide from being released into the atmosphere.

Construction Management:

Many different floor systems can be used in any given building. These include steel framing, one/two-way concrete slabs, and post-tensioned concrete slabs. In terms of one-way slabs with concrete framing there are two main options: concrete beams or skip-joists. As noted on page 14 concrete beams are no longer used in everyday design. More commonly used are skip-joists, also referred to as wide-module construction. It was also noted on page 14 that one of the main reasons for this is the cost and time associated with formwork. In today's market, forms are readily available for standard wide-module construction.

The first step to preformed formwork was initiated years ago. Pans were created for joists with slab spans of 24 - 36 inches on center. Most buildings do not have such high loading that a joist is needed every 2 - 3 feet. As a result, many times every other joist was blocked off. Thus, the term skip-joists came about. Today, on top of these pans, forms with a slab span of 53 – 66 inches have been developed.

This eliminates the need to block off every other joist making for a simplified installation process. These longer span forms are often referred to as wide-module pans.

The following is a more in-depth look into the cost and scheduling differences of a one-way slab framed with skip-joists verses concrete beams. First, a one-way system with concrete beams was designed for a typical bay. Each bay was divided into three equal spans then designed as t-beams. The structural redesign, using the same loadings as the skip-joist system design, results in the following member sizes:

Member	Size (in.)	Reinforcement
Beam	12x22	(7) #6
Girder	18x30	(3)#9, (4)#10

Table 13: One-Way Slab and Beam Schedule

A minimum slab depth of 4.5″ is required with this system. For skip-joists, a slab of 4.5″ was used as well. It must be noted that the minimum slab thickness required with skip-joists is 2.75″. However, most pan systems are used with a 4.5″ slab so 4.5″ was used in design. If floor to ceiling height needed to be optimized it would be possible to use a thinner slab of 3″ which would also lower the reinforcement requirements. As it is, both systems were considered with a 4.5″ slab reinforced with #4's at 18″ on center.

The typical bay designed with skip-joists consists of:

Member	Size	Reinforcement
Joist	7x14	(2) #8
Girder	16x26	(8)#10

Table 14: One-Way Slab and Joist Schedule

It can be seen that the size of members with the joists is much smaller. This has many advantages, which include a larger floor to ceiling height, a potentially shorter building if the same floor to ceiling height is maintained, or more cavity space for wiring and mechanical ducts. A slight disadvantage to using skip-joists is the increased number of joists in each bay. This creates a slightly larger amount of concrete.

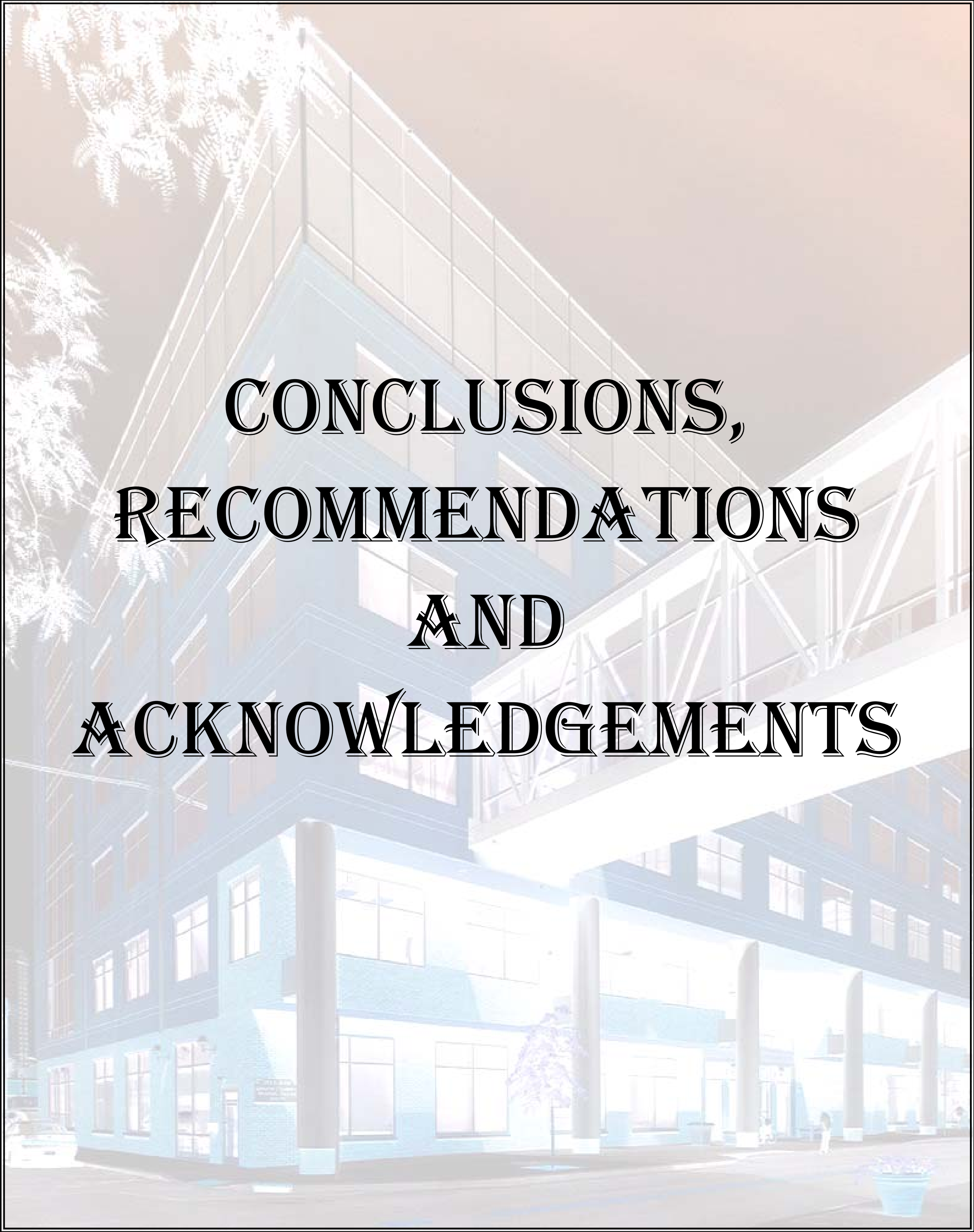
When comparing concrete beams with skip-joists cost and schedule duration were compared. The advantages of skip-joists over beams can be seen quite easily in Table 15 below.

	Cost ($)	Duration (days)
Beams	1.1 million	540
Joists	1.0 million	250

Table 15: Cost/Schedule Comparisons

These estimates were determined using ICE 2000 assuming everything else is equal i.e. foundations, columns, slabs. The base material and labor cost is roughly the same. The real savings occurs from cost avoidance due to reduced construction time. Millions of dollars can be incurred in interest alone throughout the construction of a building this size. By eliminating the time of constructing plywood forms the construction time was cut in half. A quick estimate was performed for the skip-joist system using plywood forms. This resulted in an increase of roughly $690,000 with a construction time increase of 280 days making beams a more profitable solution.

From the above tables it is easy to see why skip-joists are used over beams. The immense savings in time creates a large financial savings. The sooner tenants can move in, the sooner revenue can be created to offset the cost of construction. Cost and installation times for skip-joist pans were obtained from Ceco Concrete, a supplier of forms.



Conclusions, Recommendations and Acknowledgements

Conclusions & Recommendations

The goal of this thesis was to investigate concrete skip-joists as an alternative floor system. This included looking at a change in lateral system. One of the main issues with the current lateral system is the reduction in available exterior façade for windows. Creating more areas for natural lighting will help lower the lighting costs as well as increasing worker productivity.

The new reinforced concrete skip-joist system is a feasible alternative to the existing composite framing. An estimate comparison was performed using ICE 2000. The estimate of the new system showed a savings of $400,000 over the existing system. In contrast to this, a look at the construction schedule showed an increased duration of 75 days. However, with the new system there is far less lead-time in material procurement. Formwork pans come in standard spans and depths. As a result, once the designer has settled on using skip-joists it is a matter of determining the joist widths. Once dimensions are settled procurement of the pans can begin. Reinforcement details can be worked out after the pans have been ordered. This allows for a more flexible design process.

Many other things need to be considered when making a decision on which system to use. For example, skip-joists significantly increased the required foundation sizes. The jump from shallow foundations to deep foundations could impose a significant cost increase. Since the Wilkins building already rests on deep foundations the cost implications would not be as large as switching from a shallow foundation. However, in some cases caisson diameters double and/or triple to support the increased building weight. To help offset the cost of increased diameters the caissons can be belled. Other things to consider are the effect on the lateral system. In this case, switching to concrete introduces a cost savings. Concretes properties create moment connections with very little effort. As a result, the entire facade of the building is freed up. Furthermore, the effect on ceiling cavity height is important to consider. If the ceiling cavity is too short complications can arise for other trades such as the mechanical, lighting, and plumbing teams. By switching to skip-joists, the ceiling cavity does not change significantly one way or the other. Another incentive to using concrete is concretes fire retardant properties. The existing steel system requires spray on fireproofing everywhere.

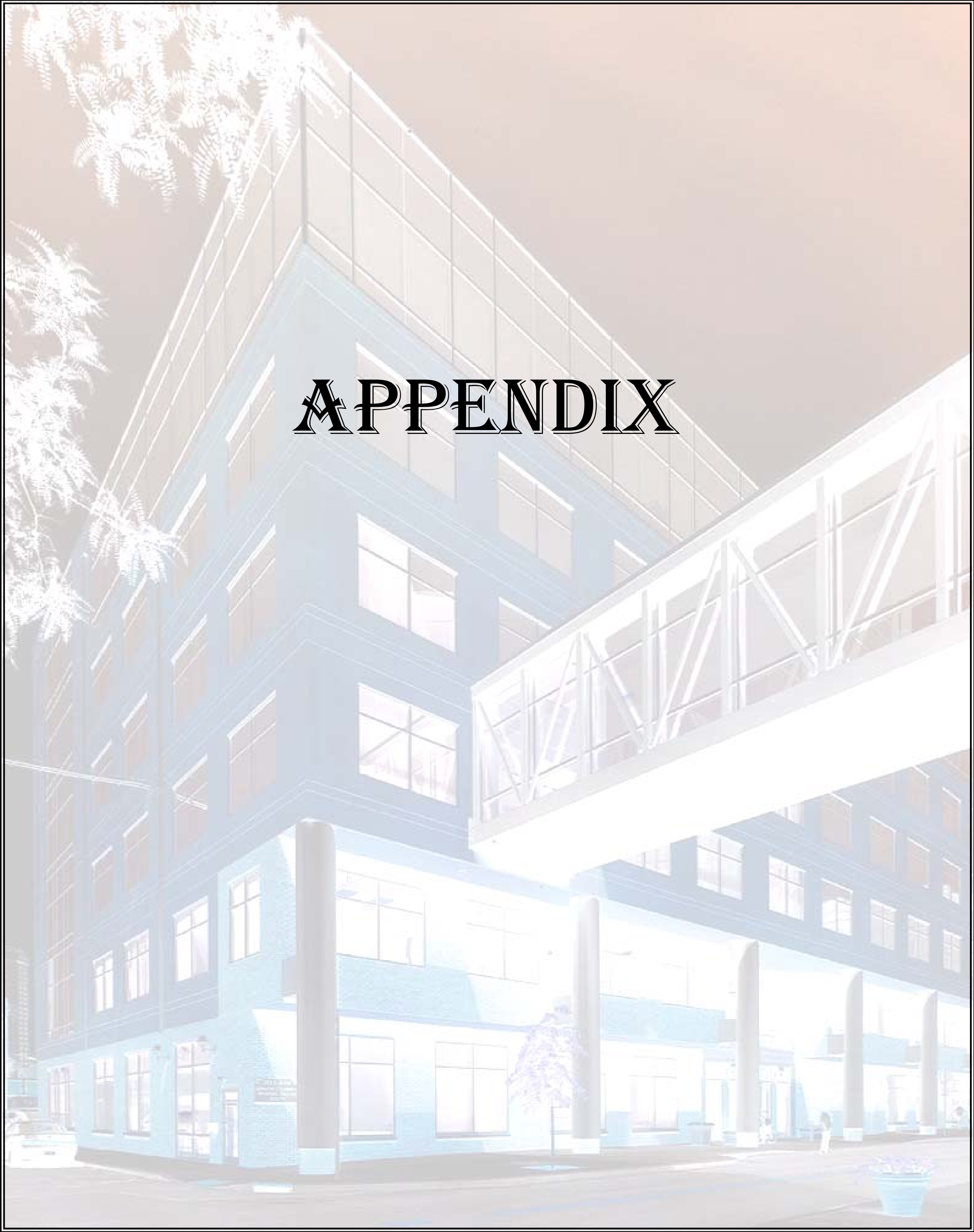
An important factor in settling on a floor system is the competence of workers in the area. For example, in Washington DC almost every building is constructed using concrete. Thus, the work force in DC is better equipped for concrete than steel buildings. In Columbus, both trades are used. Ceco Concrete has worked with many design firms to construct concrete skip-joist buildings in Columbus. This makes it more feasible to use skip-joists in the future.

In conclusion, either system will work for the Wilkins building. However, I feel it is important to have an uninterrupted façade allowing for as much natural lighting and ventilation as possible. For this reason, I would recommend reinforced concrete skip-joists with moment frames. The flexible floor plan is maintained while creating unobstructed views of Columbus from every location around the perimeter.

Acknowledgements

I would like to thank everyone who has helped me in this long process! This includes Bill Huckleberry and everyone at URS Corp., Krystal Berry from CTL Engineering, Dave Ward from Daimler Group, Rick Cevasco from Ceco Concrete, and all of the design professionals who took time out of their day to help answer questions. Thanks to everyone in the Architectural Engineering department here at Penn State who have educated me over the last few years. Finally, thanks to my family and friends for helping keep me sane through this!

APPENDIX



A1. Wind Loads

height	K_z	q_z
0-15'	0.57	10.05
20'	0.62	10.93
25'	0.66	11.63
30'	0.7	12.34
40'	0.76	13.40
50'	0.81	14.28
60'	0.85	14.98
70'	0.89	15.69
80'	0.93	16.39
84.67	0.94	16.57

G	0.85
Gcpi	0.18

Cp		
	Leeward	Windward
N-S	0.332	0.8
E-W	0.5	0.8

$P=qGCp - q_i(GCp_i)$						
	Windward		Leeward		Total	
height	N-S	E-W	N-S	E-W	N-S	E-W
0-15'	6.83	6.83	-4.68	-7.04	11.51	13.87
20'	7.43	7.43	-4.68	-7.04	12.11	14.47
25'	7.91	7.91	-4.68	-7.04	12.59	14.95
30'	8.39	8.39	-4.68	-7.04	13.07	15.43
40'	9.11	9.11	-4.68	-7.04	13.78	16.15
50'	9.71	9.71	-4.68	-7.04	14.38	16.75
60'	10.19	10.19	-4.68	-7.04	14.86	17.23
70'	10.67	10.67	-4.68	-7.04	15.34	17.71
80'	11.15	11.15	-4.68	-7.04	15.82	18.19
84.67	11.27	11.27	-4.68	-7.04	15.94	18.31

			Story Force (kip)		Cumulative Shear (kip)		OM (ft-kip)	
Floor	Height	Trib. Ht	N-S	E-W	N-S	E-W	N-S	E-W
1	0.00	0.00	0.00	0.00	110.30	236.70	5396.45	11508.87
2	16.33	14.84	18.00	39.70	110.30	236.70	293.94	648.30
3	29.67	13.34	18.10	39.20	92.30	197.00	537.03	1163.06
4	43.00	13.33	19.30	41.40	74.20	157.80	829.90	1780.20
5	56.33	13.34	20.30	43.20	54.90	116.40	1143.50	2433.46
6	69.67	14.17	22.50	47.60	34.60	73.20	1567.58	3316.29
Roof	84.67	7.50	12.10	25.60	12.10	25.60	1024.51	2167.55

Wind Loads

Bldg: $h = 84'-8" = 84.6783'$
$101'-6" \times 187'-2"$
E-W N-S

Category II => $I = 1.0$

$V = 90mph$

$K_d = 0.85$
$K_{zt} = 1.0$

Exposure = B
K_h = Case 2 -> 0.94

K_z - varies

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$

$= 0.00256 (1.0)(0.85)(90)^2 (1.0) K_z$
$= 17.6256 K_z$

flexible if $f \leq 1.0 Hz$ $f = \frac{1}{T}$

$T_a = C_t h_n^x = 0.02(84.67)^{0.75} = 0.56$

Table 12.8-2 $C_t = 0.02$ $f = \frac{1}{0.56} = 1.8 > 1.0$ ∴ Rigid
$x = 0.75$

$G = 0.85$

$GC_{pi} = \pm 0.18$ fig. 6-5

C_p: windward = 0.8
fig. 6-6 leeward: N-S = $187'-2" / 101'-6" = 1.84$ => 0.332
E-W = $101'-6" / 187'-2"$ => 0.5

$P = qGC_p - q_i(GC_{pi})$

for windward $q = q_z$
$q_i = q_h$

for leeward $q_i = q_h$
$q = q_h$

A2. Seismic Forces

Story	V	k	h_x	Area (sq. ft.)	W_x (k)	$h_x^k W_x$	C_{vx}	F_x(k)	OM (ftk)
2	214.41	1.19	16.33	18023.40	2379.09	65132.73	0.04	9.53	12986.2
3	214.41	1.19	29.67	18906.30	2495.63	138636.24	0.09	20.29	602.037
4	214.41	1.19	43.00	19029.30	2511.87	216599.21	0.15	31.70	1363.18
5	214.41	1.19	56.33	19029.30	2511.87	298279.46	0.20	43.66	2459.19
6	214.41	1.19	69.67	19029.30	2511.87	383712.57	0.26	56.16	3912.73
R	214.41	1.19	84.67	19029.30	1883.90	362591.43	0.25	53.07	4493.41

$\sum W_x =$	14294.22
$\sum h_x^k W_x =$	1464951.64

Earthquake Loads

Occupancy Cat. II => $I = 1.0$

Site Class C

$R = 3$ – ordinary moment, If Intermediate $R = 5$

$S_s = 0.15$

$S_1 = 0.06$

$F_v = 1.7$

$F_a = 1.2$

$S_{MS} = F_a S_s$

$= 1.2(0.15) = 0.18$

$S_{M1} = F_v S_1$

$= 1.7(0.06) = 0.102$

$S_{DS} = \frac{2}{3} S_{MS}$

$= \frac{2}{3}(0.18) = 0.12$

$S_{D1} = \frac{2}{3} S_{M1}$

$= \frac{2}{3}(0.102) = 0.0668$

Design Category

$S_{DS} = A$

$S_{D1} = A$

$C_s = \frac{S_{DS}}{\frac{R}{I}} = \frac{0.12}{3} = 0.04$; $\frac{0.12}{5} = 0.024$

$T_L = 0.016$

$C_s \le \frac{S_{D1}}{T_a\left(\frac{R}{I}\right)} = \frac{0.067}{0.87(3)} = 0.026$ ← controls ; $\frac{0.067}{0.87(5)} = 0.015$

$T_a = 0.016(84.67)^{0.9} = 0.87$

$k = 1$ for $T_a < .5$

$= 2$ for $T_a > 2.5$

$K = 1.185$ for $T_a = 0.87$