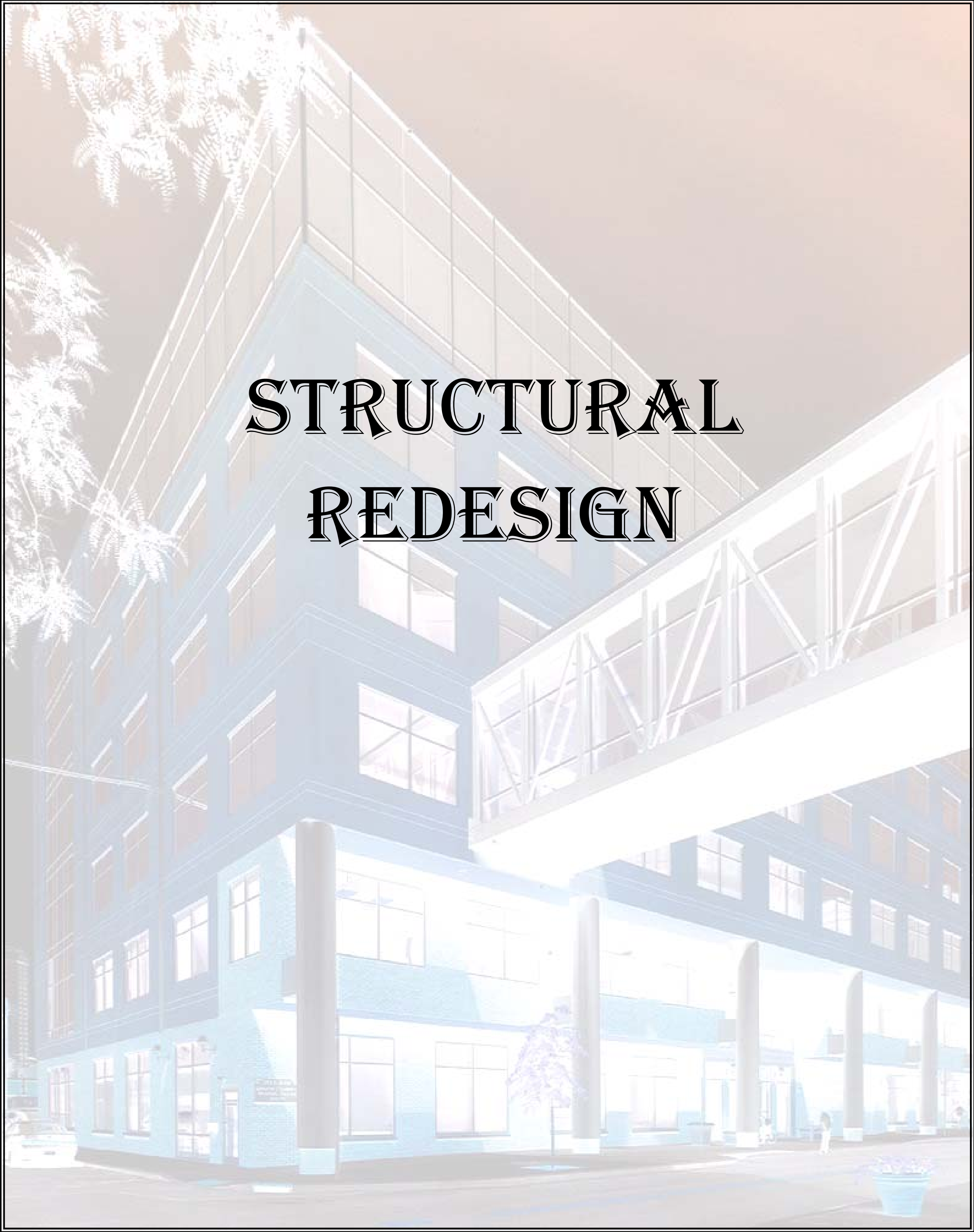

Structural Redesign

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.

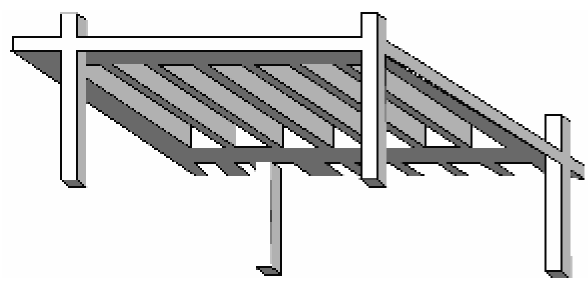

Figure 5: Finished Bay Using the Wide-Module Frame System

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)
$A_s = M_u/[\varnothing f_y(d-a/2)]$ (Eq. 5)
$\varnothing M_n = \varnothing A_s f_y(d-a/2)$ (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.

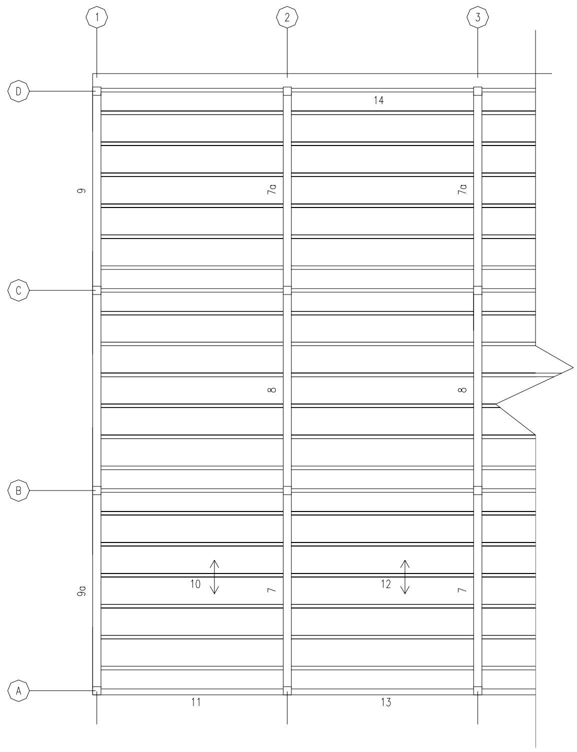

Figure 7: Joist and Girder Designations

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.

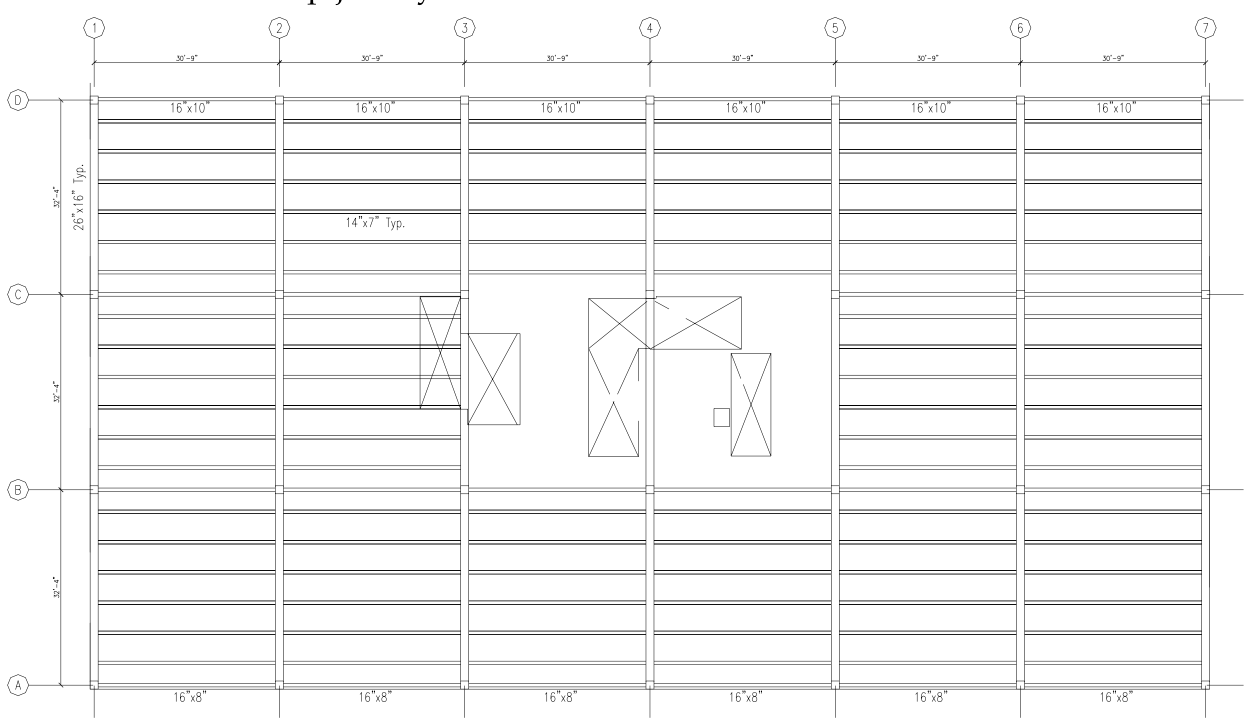

Figure 8: Typical Skip-Joist Floor Plan

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.

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

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.

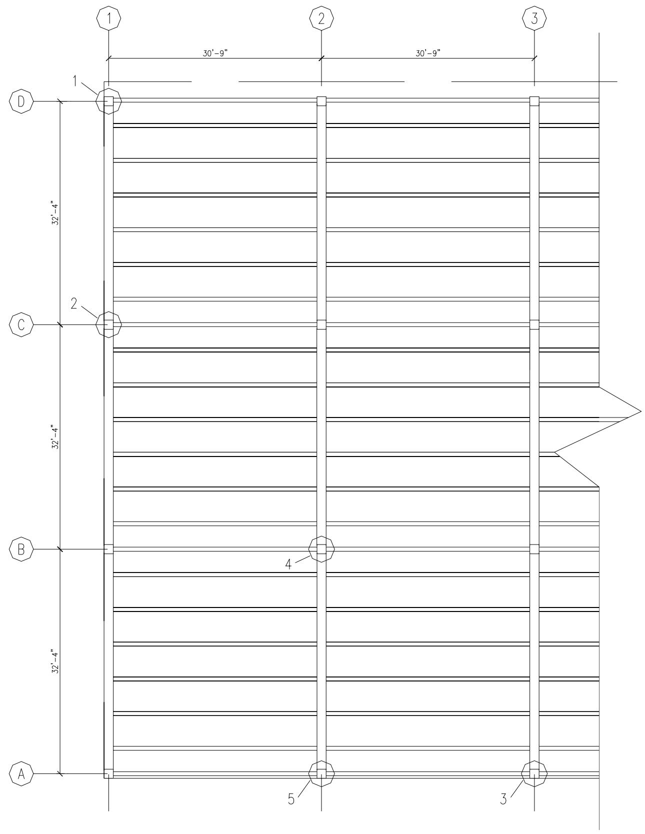

Figure 9: Column Designations

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.

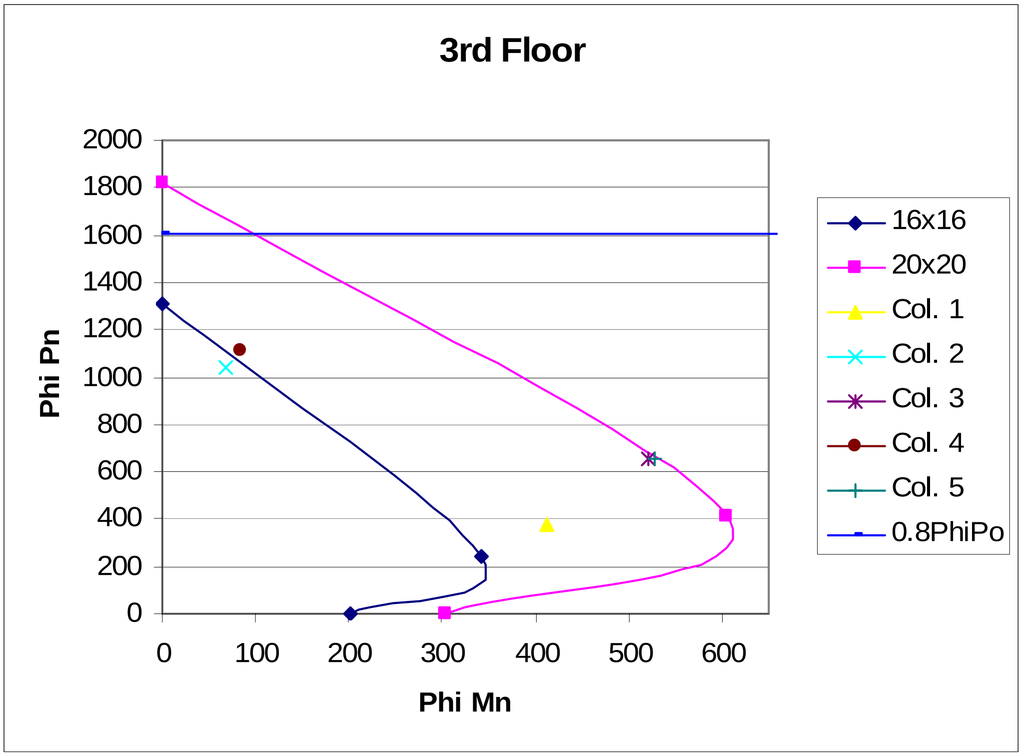

Figure 10: Column Interaction Diagram

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

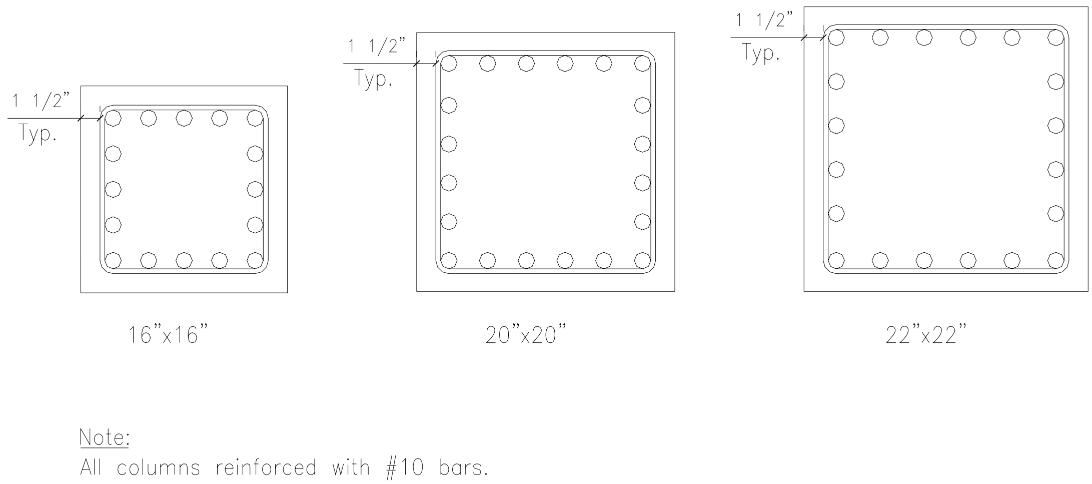

Figure 11: Column Details

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.

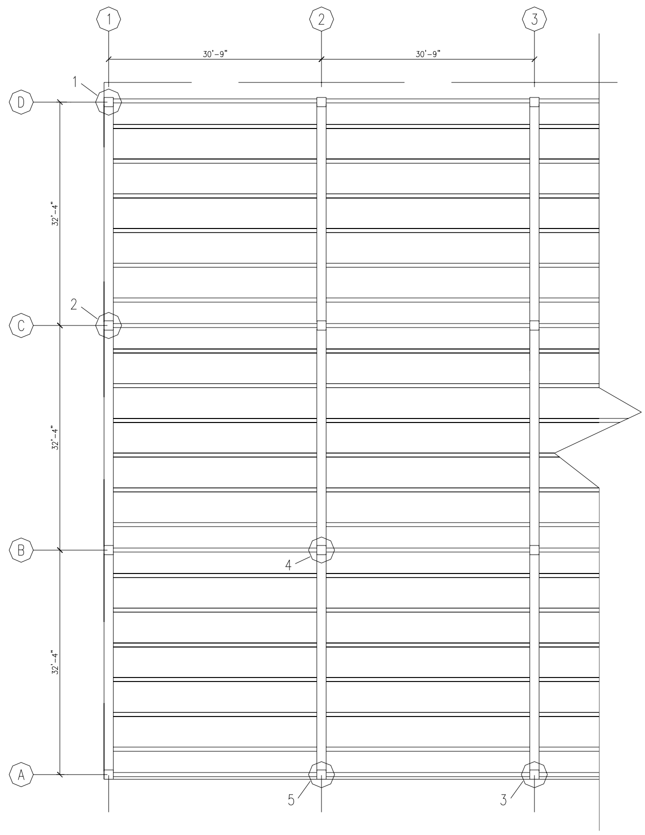

Figure 9: Column Designations

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.