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AE 482
Structural Option

Advisor: Andres Lepage
URS Office Building
October 5, 2006

FINAL REPORT

Building Systems Report

URS - ARENA DISTRICT OFFICE BUILDING

Columbus, Ohio

architecture

Ohio Basic Building Code
5 floors + penthouse
size - 102,678 SF
use - mercantile + office
materials - masonry exterior
industrial glass
roof - single ply EPDM
polyisocyanurate
board insulation

structural

FOUNDATION
- spread footing of various size at the columns along with grade beams

MAIN
- 5" Slab on grade
- 2" steel deck + 3-1/4" light weight concrete composite floor system
- 30'x30' typical bay
- wide-flange structural steel
- braced frames to resist lateral forces

ROOF
- 20 gage steel roof deck on steel beams (W16 typical)

project team

owner - Nationwide Realty Investors
architect - URS Greiner Woodward Clyde
engineer - URS Geiner Woodwark Clyde
general contractor - Continental Building Systems

construction

project delivery - design, bid, build
construction date - from march 2000 to january 2001
total project cost - 7 million dollars

mep

- 7 roof top units varying from 2000 to 43000 CFM capacity
- VAV control system
- central core contains elevators, stairs, restrooms
- 300 KVA transformer delivering 208/120V three phase power
- 277V, 4' wide, flourescent with T8 typical

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Structural Option

http://www.arche.psu.edu/thesis/eportfolio/2007/portfolios/DWL141/

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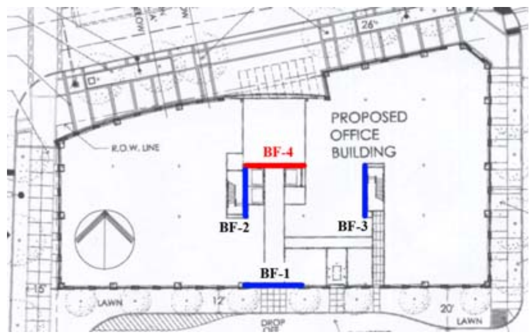
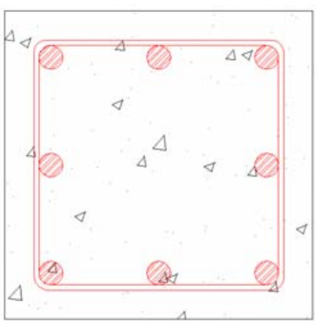
Executive Summary

Steel frame structure clad with masonry, the 5 story 100,000 SF URS Office Building is a multiuse structure containing mercantile area on the ground floor and office space from floor 2 through 5. Completed construction within a year this 7 million dollar project located in Columbus, Ohio leads as a prototype building in the developing Arena District.

Structural steel composes beams, columns, girders, and bracing members. Composite floor system is in use utilizing shear studs and spread footings with grade beams are the foundation system. Lateral system consists of 3 braced frames and 2 moment frames. Moment frames were used due to architectural constraints. Also due to asymmetric layout of lateral system, effects of torsion were sizeable.



Contained in this document is the redesign of lateral system in steel and concrete. Primary considerations are reducing the impact torsion has on the structure, cost, and schedule. First proposed change is rearranging the current lateral system. By adding an additional braced frame, BF-4, the eccentricity is reduced and moment frames are eliminated.



Second solution proposed is to construct the building in concrete and using shearwalls as the new lateral system. For this solution complete redesign of the structure is necessary. Applied loads will be recalculated, gravity members will be sized, and finally shearwalls will be designed.

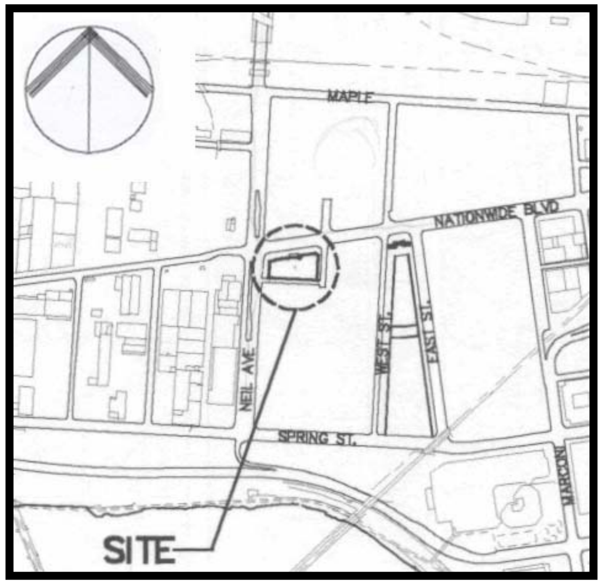
Due to the original schedule and costs being unavailable, construction breadth focused on creating schedules for the original steel project and comparing it to the new concrete alternative. Cost, time, and constructability are the primary concerns for this study.

Mechanical breadth consists of evaluating and verifying the minimum outside air requirement, reducing equipment size and saving on energy cost by reducing building volume. Using post-tensioned concrete slab reduces floor depth significantly so smaller fan will be considered if viable.

After all the analysis, steel is recommended over the concrete alternative. Steel erection is very quick and cost difference of $150,000 gives advantage to steel. Also the existing building is the testament to the steel structure's integrity.

EXISTING CONDITIONS

Building Statistics



URS – Arena District Office Building is located in Columbus Ohio. The 100,000 square feet, 5 story building is the prototype design for the Arena District being developed by Nationwide Realty Investors. Designed as mercantile/office building, the URS Office Building provides retail area on the first floor and office area from second through fifth floor. Completed construction in January 2001, this design, bid, build project's total cost was $7 million.

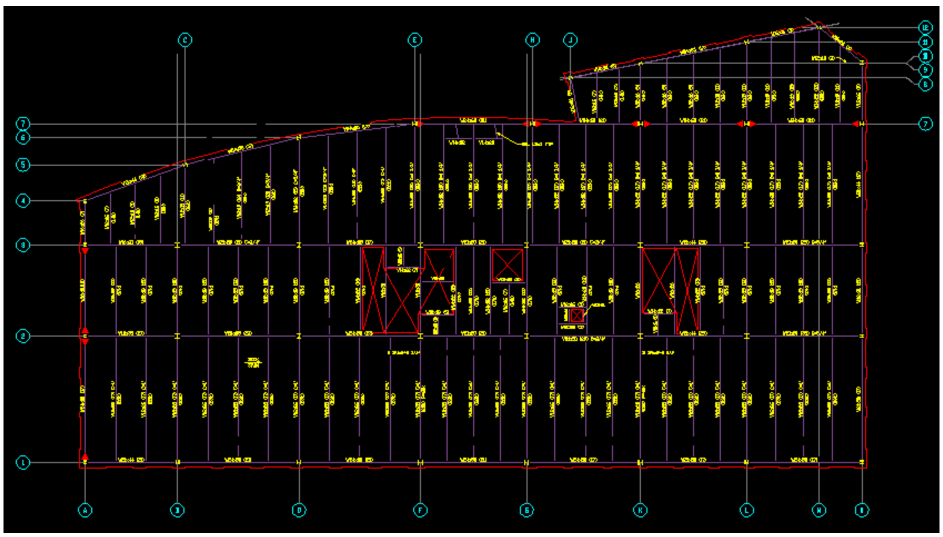
For the URS Office Building structural design was performed under Ohio Basic Building Code 1998 (OBBC). OBBC was created by adopting BOCA National Building Code 1993. Structural standards for structural steel, cast in place concrete, pre-cast concrete, metal deck, and masonry are shown below in *Table 1*.

OHIO BASIC BUILDING CODE 1998		
STRUCTURAL STEEL	• AISC Manual of Steel Construction	
CAST IN PLACE CONCRETE	• ACI 318 – "Building Code Requirement for Reinforced Concrete" • ACI 301-89 – "Specification for Structural Concrete for Buildings"	
PRECAST CONCRETE	• ACI 318 – "Building Code Requirement for Reinforced Concrete" • PCI MNL 120 – "PCI Design Handbook Pre-cast and Pre-stressed Concrete"	
METAL DECK	• AISI – "Specification for the Design of Cold-formed Steel Structural Members" • SDI – "Design Manual for Composite Decks, Form Deck, and Roof Decks"	
MASONRY	• ACI 530.1/ASCE 6/TMS 602	*Table 1*

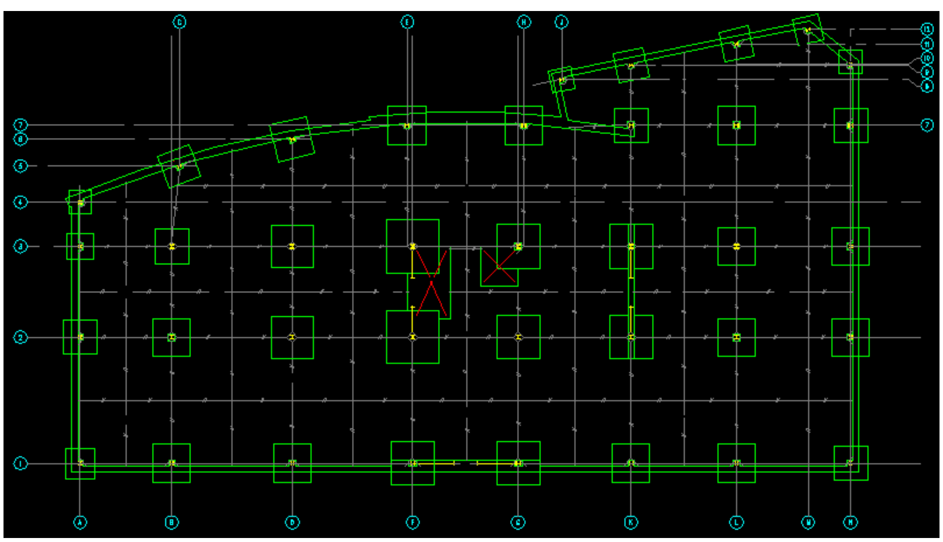
In January 1 of 2002, State of Ohio adopted the 2000 International Building Code (IBC). Current building code in Ohio is the 2005 Ohio Building Code (OBC) based on 2003 IBC. Therefore throughout the report, 2003 IBC along with ASCE 7-05 will be used as the structural standard. Although the original calculations were performed using ASD 9th Edition of the steel manual, steel design in this report will be determined by LRFD 3rd Edition of the steel manual.

Project Team

Owner	Nationwide Realty Investors	www.nationwide.com
General Contractor	Continental Building Systems	www.continental-realestate.com
Architect	URS Greiner Woodward Clyde	http://www.urscorp.com
Structural Engineer	URS Greiner Woodward Clyde	http://www.urscorp.com
Mechanical Engineer	URS Greiner Woodward Clyde	http://www.urscorp.com
Electrical Engineer	URS Greiner Woodward Clyde	http://www.urscorp.com
Civil Engineer	EMH&T	www.emht.com



Typical Floor Plan



Foundation Plan

Architecture

Architecture: Among the first buildings completed in the Arena District by the Nationwide Realty Investors, URS Office Building leads the design for future buildings to come. The building footprint follows the site with the help of the set back. This set back and the elegant curvature that makeup the street side façade gives distinction to the otherwise rectangular building. URS Office Building is a steel frame construction with masonry façade. Large openings punched into the brick allow for plenty of natural light to enter. The brick work, by color, distinguishes the mercantile area on the ground floor from the office space on upper floors. Protruding brick strips in between the windows imitate long columns. Also the entry canopy and the arched entrance on a curving wall draw focus to the lobby.

Envelope: This steel frame structure is surrounded by brick masonry veneer along with large punched windows which incorporate industrial windows. Light colored brick compose and highlight the center of the structure and ground floor mercantile area from the dark colored brick that envelops the rest of the building. Low slope steel roof deck covered with ethylene propylene diene monimer (EPDM) single ply membrane with polyisocyanurate board insulation comprises the roofing system.

Architectural Restraints: Primary requirements for the URS Office Building were to provide the client with large spans to allow for the open floor plan and to keep the street facing facades North and West walls free of obstruction.

Electrical/Lighting

Proper transformer vault provided by the electric company is located east of the building. The utility-owned transformer then connects to the main 3000A – 480V, 3 phase, 4 wire switchboard in the ground floor electrical room. From the switch board, 400A runs to a 300KVA transformer which steps down 480/277V, 3 phase, 4 wire primary service to 208/120V, 3 phase, 4 wire secondary service.

Lighting the office spaces are 277V, 4 inch fluorescent up-light with two T8 lamps typical. In the lobby areas and certain emergency lights employ incandescent lighting. Around the perimeter of the building are in-ground lights which are metal halide lamps.

Mechanical

Located on the roof of URS Office Building is variable air volume cooling unit. Two 40 ton units along with two 20 ton units condition the air for the 4 floors of office space and a mercantile floor. The two 40 ton units supply conditioned air to all five floors and the two 20 ton units will become active as the anticipated kitchen on the mercantile area opens up. Mechanical ducts travel up and down through the slab openings next to the stairwells (see *Figure 1*).
The variable frequency drive motors control the centrifugal fans' rotational speed pushing appropriate volume to the spaces. In this building cooling load controls the design.

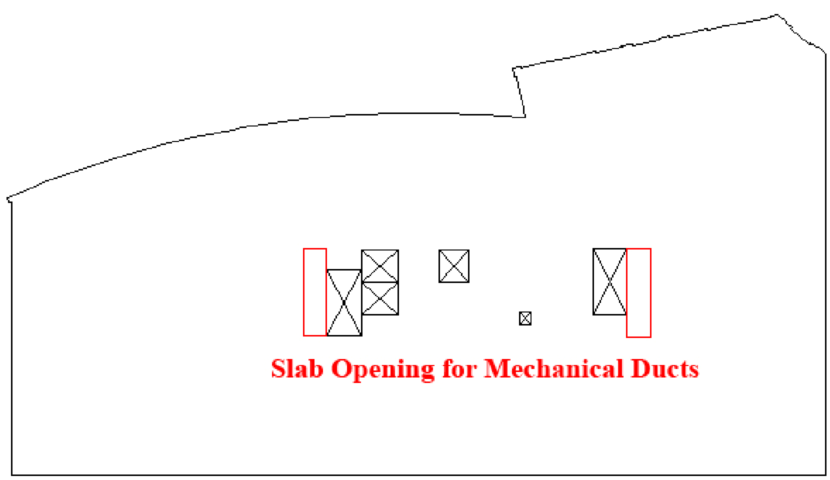


Figure 1

Structural

URS Office Building is a steel frame structure enclosed with brick masonry and industrial glass. Structural steel used for beams, columns, and girders is ASTM A572 grade 50 wide-flange with yield strength of 50,000psi. Longest beam spans 33'4" and longest girder spans 32'. Typical bays are 30'x 30' with most bays being approximately a square. The chevron bracings resisting lateral loads are ASTM A500, Grade B tube steel with yield strength of 46,000psi.

The design bearing capacity of the soil was determined to be 4000 PSF by the sub-surface investigation. Due to poor quality of the soil minimum of 7 feet of lean concrete was to be placed under all foundations. Spread footings with minimum compressive strength at 28 days of 3000psi together with grade beams are employed as the foundation system. The size of footing varies from 6'x 6'x 16" to 14'x 14'x35". The grade beams also vary in width as well as depth. Both the spread footings and grade beams utilize bars #6, #7, #8, or #9 with #4 stirrups. The slab on grade has required minimum compressive strength at 28 days of 4000psi and the composite slabs are to be lightweight concrete with minimum strength of 3000psi. First floor slab on grade is 5" concrete slab reinforced with welded wire fabric. The composite slabs on floors 2 through 5 are composed of 2"x 20 gage steel deck and 3-1/4" light weight concrete also reinforced with welded wire fabric.

Steel roof decks are galvanized 20 gage ASTM A653 grade 33 G90 zinc coated steel. The composite steel floor decks are galvanized 20 gage ASTM A653 grade 33 G60 steel. Headed studs ¾"ϕ x 4" spaced evenly across the steel members are used to achieve composite action.

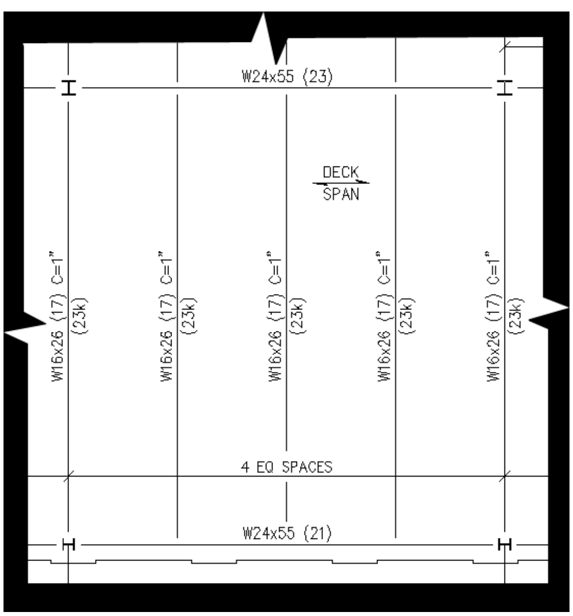


Figure 2a

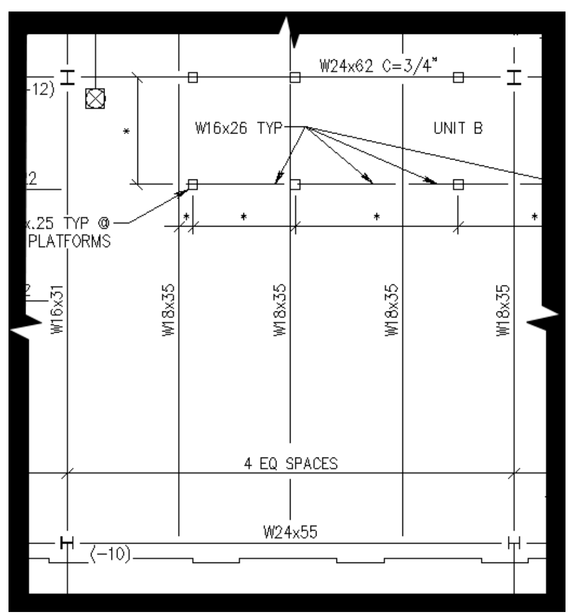


Figure 2b

Typical Floor Framing

Ground floor is composed of 5" slab on grade. Second through fifth floor has identical framing plan, shown in *Figure 2a* and the roof framing plan is shown in *Figure 2b*. The largest bays are 32'x 33'4". Typical girders are W24 and typical beams are W16.

Lateral System

Concentric braced frames are used to resist most of the lateral loads in the URS Office Building. Three chevron bracing and along with 2 moment frames compose the complete lateral system (*see Figure 3*). The bracing members are rectangular hollow structural sections ranging from 6"x 6"x 0.25" to 8"x 8"x 0.375" and moment frame elements are W-shapes. Brace frame 1 resists the east-west lateral loads. Brace frames 2 and 3 provide lateral resistance in the north-south direction. Moment frames 1 and 2 exist to provide stability against torsion. Moment frames were employed due to architectural constraint. North face of the building being the street façade prevented the use of braced frame. The composite floor system provides a rigid diaphragm to distribute the lateral loads to the frames.

Brace frame 1 (WB-1) resists east/west lateral loads and brace frames 2 and 3 (WB-2 and WB-3) resists north/south lateral loads.

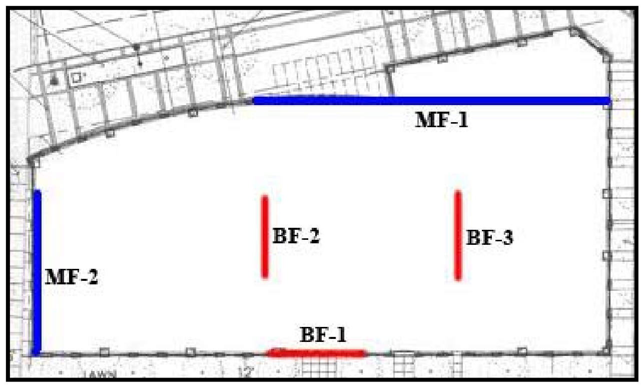
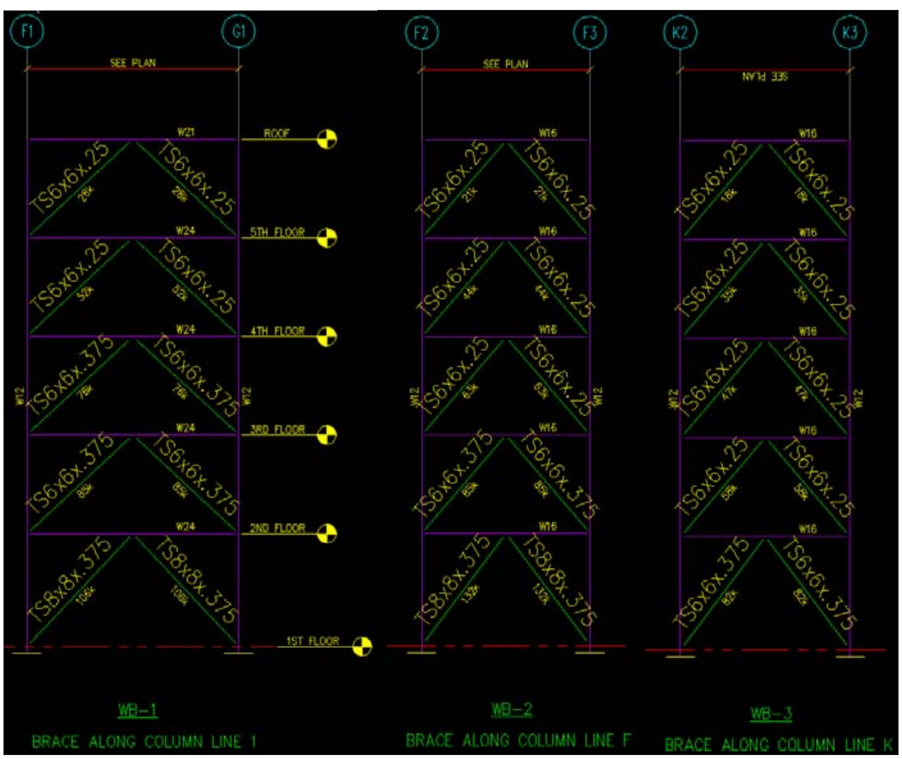


Figure 3



Load Combinations

Load combinations were taken directly out of the ASCE 7-05. Applicable loads in this report include dead, live, wind, and seismic.

1. $1.4(D+F)$
2. $1.2(D+F+T)+1.6(L+H)+0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D+1.6(L_r \text{ or } S \text{ or } R)+(L \text{ or } 0.8W)$
4. $1.2D+1.6W+L+0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D+1.0E+L+0.2S$
6. $0.9D+1.6W+1.6H$
7. $0.9D+1.0E+1.6H$

Gravity Loads

Dead Loads (PSF) – actual weight of the permanent building components

- Structural Steel ------ 6.5 PSF
- Metal Deck ----------- 3 PSF
- Concrete -------------- 43 PSF
- MEP ------------------- 15 PSF
- Partition --------------- 20 PSF

- Total Dead Load ----- 87.5 PSF

Live Loads (PSF) – from 2003 IBC: Table 1607.1

- Roof Snow ----------- 25 PSF
- Office Floor ---------- 50 PSF
- Corridor --------------- 100 PSF
- Lobby ----------------- 100 PSF
- Retail ------------------ 100 PSF
- Penthouse Floor ------ 250 PSF
- Mechanical Unit ----- 150 PSF + weight of equipment

Snow Load (PSF) – from ASCE 7-05: Chapter 5

- Primary concern for snow load is next to the penthouse where drifting may occur. Following the guidelines in ASCE 7-05 snow load was calculated and drift was considered.
- The maximum drift calculated was 59.3 PSF

Lateral Loads

Upon investigation, north-south loading is controlled by wind and east-west loading is controlled by seismic. Through the use of RAM model, excel spreadsheet, and hand calculation the lateral loads below were calculated. All three methods provided comparable numbers which validate the numbers found in the construction document.

In the north-south direction, un-factored base shear due to wind is 175.86 kips. Multiplying the 1.6 factor, base shear turns out to be 281 kips. In the east-west direction seismic base shear controls with 169 kips.

Lateral Load Direction	Base Shear Due to WIND	Base Shear Due to SEISMIC
North-South Loading Considered Y	281.38 kip	169.30 kips
East-West Loading Considered X	142.8 kips	169.30 kips

Relative stiffness method was employed to distribute the computed lateral loads. Stiffness was calculated from the positions of frames to the center of rigidity (see *Figure 4*). For the north-south direction because of the rigid diaphragm provided by the floor system, braced frames 2 and 3 were assigned equal stiffness. After running the numbers, the moment frame only resisted 6.3% of the north-south lateral load which turned out to be 18 kips leaving 264 kips to be resisted by the braced frames. In the east-west direction braced frame 1 resisted 150 kips and the moment frame 19 kips.

Logical load path for the URS Office Building would be lateral loads against the façade relaying the loads to the floor system. Then the floor acting as a diaphragm would distribute the loads to the braced frames and the moment frames. And ultimately the loads will travel down the lateral frames into the foundations.

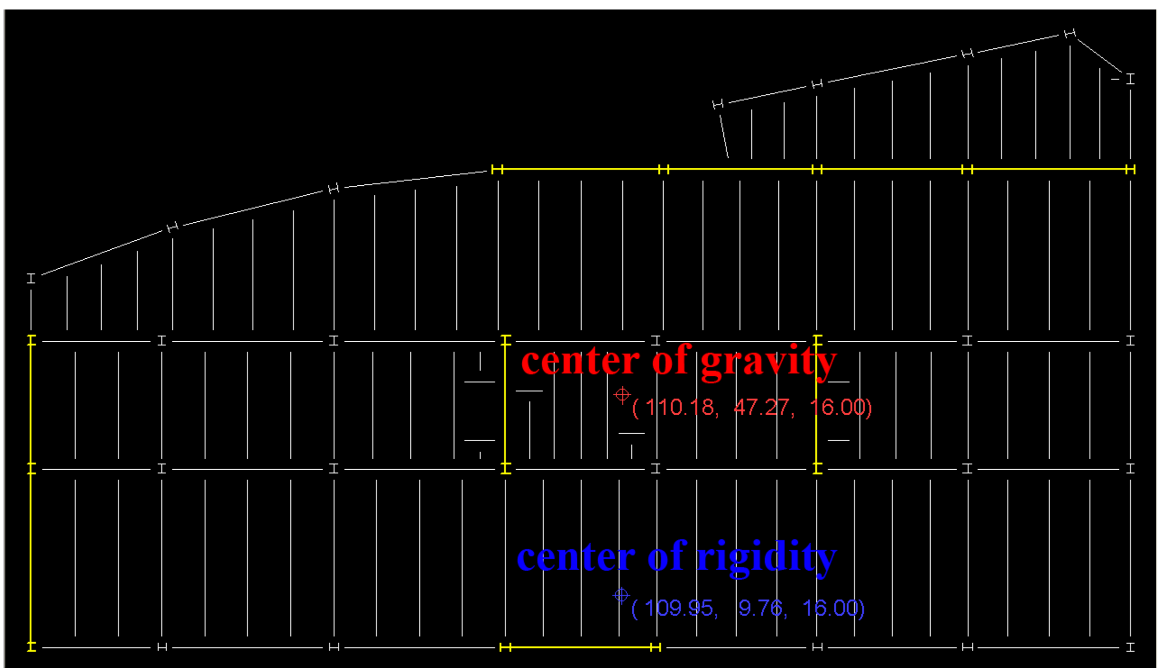


Figure 4

Strength and Serviceability

Structural members were checked for strength and serviceability with computer software as well as spot checks done by hand. Strength, drift, story drift, overturning, as well as foundations were checked. In strength design, most members were more than sufficient to carry the computed loads. Shown in *Figure 5* is the deflected shape of the frames at scale factor of 100.

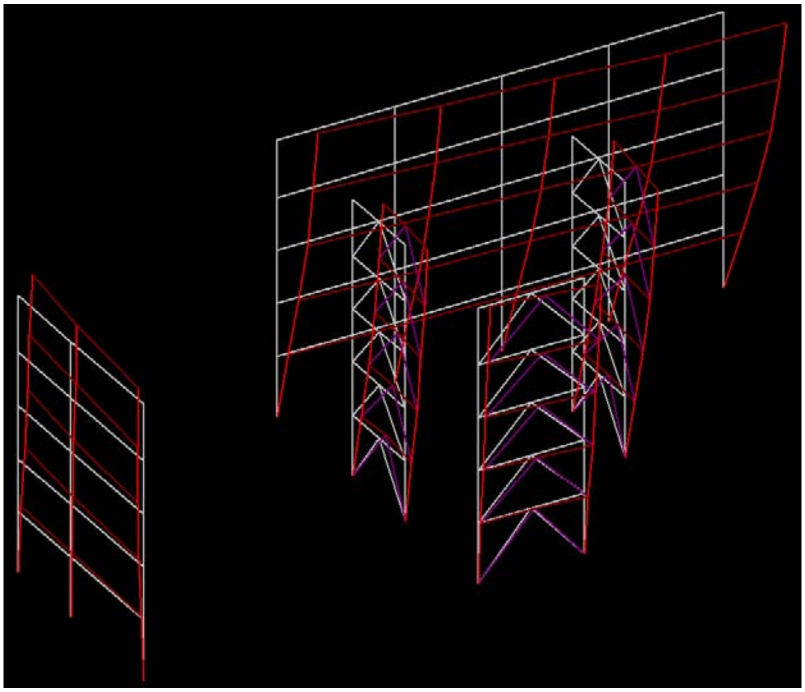


Figure 5

Drift limit for wind was set to H/400. This is a rule of thumb for building drift commonly used in the industry. Maximum allowed drift was 2.16" and largest displacement due to controlling lateral load was 1.5". Story drift was also calculated and typical allowable story drift was 0.42". Actual story drift was less than 0.4". Also seismic story drift limitation in chapter 12 of ASCE 7-05 was met.

Drift Calculations from RAM

Floor	Height (feet)	Floor to Floor Height	Max Displacement		H/400 Drift	Story Drift		H/400 Story Drift
			X (inch)	Y (inch)		X (inch)	Y (inch)	
R	72	14	1.456	1.019	2.16	0.224	0.164	0.42
5	58	14	1.232	0.855	1.74	0.275	0.185	0.42
4	44	14	0.957	0.67	1.32	0.316	0.232	0.42
3	30	14	0.641	0.438	0.9	0.337	0.229	0.42
2	16	16	0.304	0.209	0.48	0.304	0.209	0.48

Overturning moments were calculated using only the controlling lateral loads. North-south direction controlled by wind resulted in 12,655 foot-kips of overturning and 264 kip column force. In the east-west direction seismic controlled which produced 8,962 foot-kips of overturning and 320 kip column force.

With the aid of CRSI Design Handbook, foundation spot check was performed. Using bearing capacity of 4000 PSF along with square footing sizes in the structural drawing, capacity was found in page 13-7. Comparing the axial load calculated to the capacity, footings were found to be adequate.

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Torsional Analysis

Due to the asymmetrical layout of the frames torsion had to be accounted for in this report. Torsion due to wind and seismic loading were calculated. Wind load normal to east or west face of the building produced 1,842 foot-kips. In the north-south direction torsion was 6,479 foot-kips.

For the seismic loading case, eccentricity was taken as the distance between center of mass and center of rigidity. Also accidental torsion was taken into account as 5% of the dimension normal to lateral load multiplied by the lateral load. The total torsion in the east-west direction is 7308 foot-kip and in the north-south direction is 1768 foot-kip.

PROPOSED STUDY

Problem Statement

Designed by experienced engineers working for the largest global engineering design firm URS Office Building was cost efficient, structurally sound, and met all of the architectural constraints. After completing the technical assignments, decision was made to alter the current lateral system. Although adequate in resisting lateral loads determined with the aid of Ohio Building Code 2005 (adaptation of IBC 2003) and ASCE 7-05, effect of torsion due to eccentricity in the y-direction is significant.

Proposed Solution

Redesign of the lateral system is proposed. Two viable solution considered are **altering the current lateral system**, and **redesign of building in concrete with shearwalls** as the lateral system.

In order to reduce torsion and to enhance constructability, moment frames will be replaced with a single braced frame. Primary goal of eliminating the moment frames is to reduce mistakes in the field, increase productivity, and reduce cost. The other objective is reducing eccentricity which in turn reduces the impact of torsion.

The concrete redesign will employ post-tensioned slabs along with cast in place columns and shearwalls. Redesign in post-tensioned slab will require changes in loads applied, column sizes, and, foundation reevaluation. Shearwalls are planned to coincide with the elevator shafts and if necessary other locations to minimize eccentricity. Post-tensioning was chosen to maintain large open space without interruptions. Added benefit of this floor system is the shallow floor depth.

Design Criteria

- ❖ Design must comply with Ohio Building Code 2005 (adaptation of IBC 2003)

- ❖ Architectural constraints imposed on original design team must be followed
    - Street facing facades free of obstruction
    - Large open space to allow flexibility in floor layout

- ❖ Design as an Architectural Engineer – consider all aspects of construction

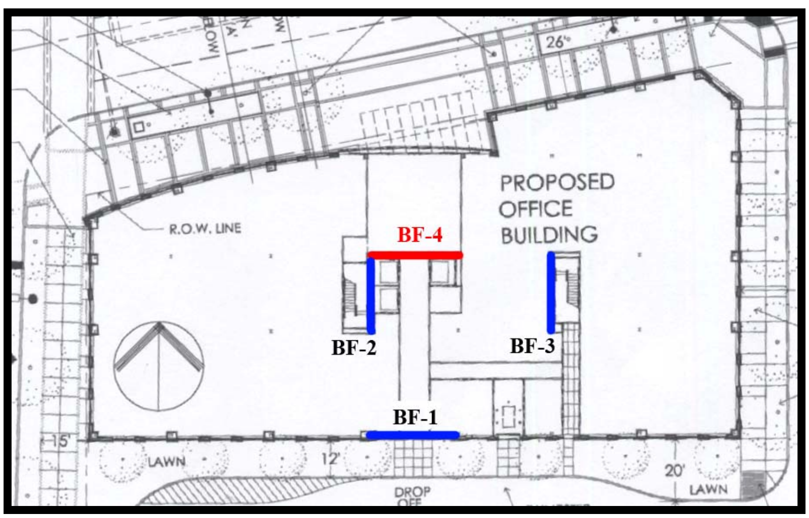
DEPTH STUDY

Design Process

In order to comply with the design criteria above, construction documents were reviewed. Possible locations for the new lateral systems were chosen. RAM model was created for the alternate steel lateral framing and hand calculations were performed to verify RAM output. Strength and serviceability checks were performed.

Due to the increase in self weight of the concrete design, dead load and lateral loads were redefined. Applying the calculated loads, post-tensioned slab, columns, and shear walls were design according to ACI 318-05.

Alternate Steel Lateral Framing



Both moment frames are eliminated and braced frame **BF-4** is added. Ideal location for the new braced frame would be at the perimeter of the building. However the restrictions imposed by the architect, does not allow braced frames on the perimeter of the building. After extensive study of the architectural and structural drawing, final location of the braced frame BF-4 was chosen where it would not interfere with the open floor system. The added lateral frame maintains the 8' width corridor between the elevators and has 8' clearance height which exceeds code minimum ceiling height of 7'6" (see *Figure 6*).

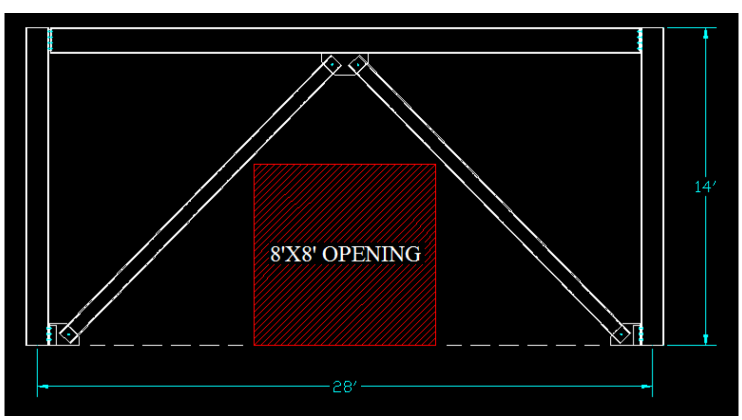


Figure 6

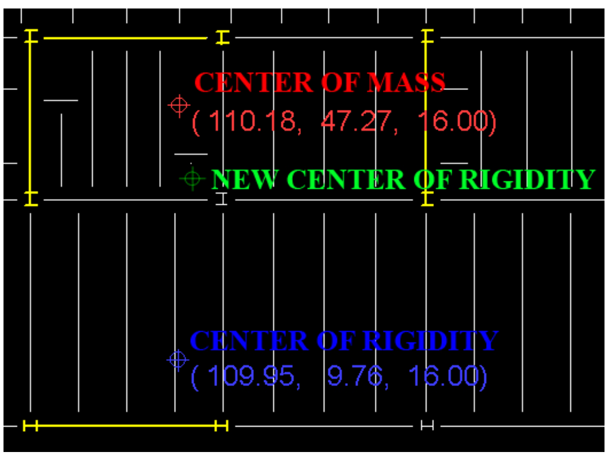
Loads

All the previously calculated loads for the existing building were applied. Live load did not change. Dead loads were not altered since no member sizes were changed. In the case of columns, most were controlled by load combination 2; therefore all the members remained the same size. With the same loads applied to the floor system, beams and girder sizes also remained the same. Wind loads were not impacted by the redesign and no changes in building weight kept the seismic loads the same.

1.4(D + F)
1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R)
1.2D + 1.6(Lr or S or R) + (L or 0.8W)
1.2D + 1.6W + L + 0.5(Lr or S or R)
1.2D + 1.0E + L + 0.2S
0.9D + 1.6W + 1.6H
0.9D + 1.0E + 1.6H

Torsional Analysis

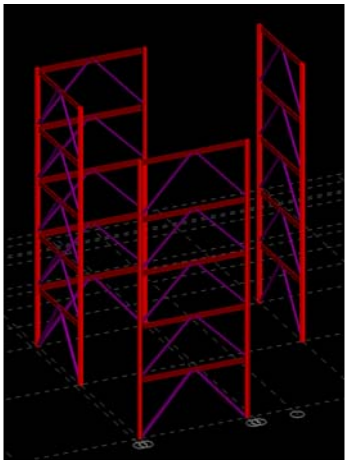
Performing the hand calculations, columns of the braced frames are considered infinitely stiff. With this assumption center of rigidity was found. Although the moment frame previously occupying the north façade was further away from the braced frame BF-1, due to the increased stiffness in the braced frame BF-4 the center of rigidity moved further away from braced frame BF-1 and closer to the center of rigidity. Due to the decrease in eccentricity controlling applied torsion plus the accidental torsion reduced from 8205 ft-k to 4112 ft-k. In the original design as well as the new design the controlling case was the seismic load plus accidental torsion due to seismic loading. And shown in *Figure 7* in white is the shear force due to torsion for the original design and in colors is shear force due to torsion for the new design.



Existing Proposed	Torsion (ft-k)	r (ft)	k	Torsional Shear Force (k)
WB-1	8205	9.04	1025.26	29.82
WB-2	8205	21.97	904.8	63.97
WB-3	8205	36.03	659.46	76.45
MF-1	8205	80.29	21.06	29.84
MF-2	8205	110.3	4.74	12.51
WB-1	4112	28.7	1.39	1.62
WB-2	4112	31.32	1.17	1.49
WB-3	4112	26.68	1	1.09
WB-4	4112	28.7	1.39	1.62

Figure 7

Strength Serviceability



The new steel lateral frame system was modeled in RAM Structural Systems and hand calculations were accompanied to prove the legitimacy of the model. The structural steel members were found to be adequate in axial, shear, and bending. Due to near identical loading and load combination 2 controlling the lateral frame design, this is to be expected. Maximum displacement as well as story drift was calculated for the new lateral frame system. As before wind controlled design for loads normal to North-South direction and seismic for loads normal to East-West direction.
Calculated deflections were compared to industry standards. For seismic story drift, deflection amplification factor, Cd, was multiplied to the story drift determined according to the elastic analysis. This number is then divided by the importance factor and compared to 2% of story height. The seismic story drift limitation was met and total displacement in X and Y directions exceed h/590 which meets industry standards.

Floor	Height (feet)	Floor to Floor Height	Max Displacement		H/400 Drift	Story Drift		Seismic Story Drift Limit	H/400 Story Drift
			X (inch)	Y (inch)		X (inch)	Y (inch)		
R	72	14	1.449	1.011	2.16	0.709	0.159	3.36	0.42
5	58	14	1.237	0.852	1.74	0.867	0.184	3.36	0.42
4	44	14	0.948	0.668	1.32	0.993	0.230	3.36	0.42
3	30	14	0.625	0.438	0.9	1.02	0.226	3.36	0.42
2	16	16	0.301	0.212	0.48	0.959	0.212	3.84	0.48

Foundation Impact

Although few spread footings were over designed, accounting for constructability the footing sizes were maintained.

Concrete Design

Loads

In light of the huge increase in the self weight of the structure, new loads were calculated for the concrete design. Concrete density was assigned 150 PCF.

Dead Loads (PSF) – actual weight of the permanent building components

- Slab Self Weight ---- 106 PSF
- MEP ------------------- 15 PSF
- Partition --------------- 20 PSF

- Total Dead Load ----- 141 PSF

Live Loads (PSF) – from 2003 IBC: Table 1607.1

- Roof Snow ----------- 25 PSF
- Office Floor ---------- 50 PSF
- Corridor --------------- 100 PSF
- Lobby ----------------- 100 PSF
- Retail ------------------ 100 PSF
- Penthouse Floor ------ 250 PSF
- Mechanical Unit ----- 150 PSF + weight of equipment

Snow Load (PSF) – from ASCE 7-05: Chapter 5

- Primary concern for snow load is next to the penthouse where drifting may occur. Following the guidelines in ASCE 7-05 snow load was calculated and drift was considered.
- The maximum drift calculated was 59.3 PSF

Wind Load (PSF) – from ASCE 7-05: Chapter 6

- Basic Wind Speed = 80 mph
- Exposure Category B
- Importance Factor = 1
- Building Height = 76'
- Given below are the unfactored wind loads

E-W LOAD Level	Elevation	Applied Force in kips
1	0	0
2	16'	18.35
3	30'	16.14
4	44'	17.39
5	58'	18.26
Roof	72'	19.11
Base Shear Force		89.25

N-S LOAD Level	Elevation	Applied Force in kips
1	0	0
2	16'	34.06
3	30'	32.22
4	44'	35.02
5	58'	36.54
Roof	72'	38.02
Base Shear Force		175.86

Seismic Load (PSF) – from ASCE 7-05: Chapter 11 and 12

- Site Class D
- Ss = 0.12g
- S1 = 0.05g
- Importance Factor = 1
- Seismic Use Group I
- Seismic Building Category B
- Given below are the seismic loads

FLOOR	LOAD (PSF)	AREA (SF)	w (k)	h (ft)	wh^x	Cvx	F (k)	STORY SHEAR (k)
2	126.25	20000	2525	16	40400	0.073	23.6	323.1
3	126.25	20000	2525	30	75750	0.136	43.9	299.5
4	126.25	20000	2525	44	111100	0.2	64.6	255.6
5	126.25	20000	2525	58	146450	0.264	85.3	191
R	126.25	20000	2525	72	181800	0.327	105.7	105.7

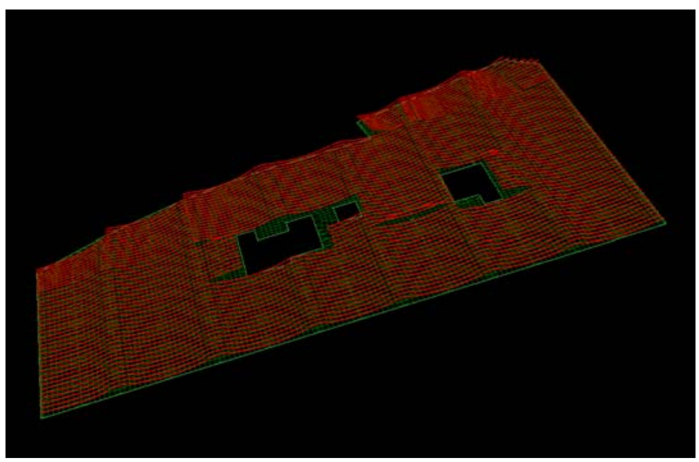
Controlling Lateral Load

- Accounting for the load factors for the concrete structure seismic load controls design in both the X and Y direction

LOAD TYPE	Factored Load In X direction	Factored Load In Y direction
WIND	143 kip	281 kip
SEISMIC	323 kip	323 kip

Post-tensioned Slab Design

In order to maintain long spans with minimal floor depth 2 way flat plate post-tensioned slabs are used. The slabs were design by methods prescribed in Naaman's text, Prestressed Concrete Design and Analysis. The hand calculations and excel worksheet were aided by the computer software RAM ConcePT.



The slab is designed with normal weight concrete. Light weight concrete was considered to reduce the dead load of the building, however the premium imposed in Columbus made it impractical. Concrete strength of 5000psi is used to design the slab as, Class U, uncracked section. Suggested span to depth ratio of 45 was used to calculate appropriate slab thickness. With the thickness determined, gravity loads were calculated. At this point, 90% of the slab's self weight was chosen to be balance by pre-stressing the concrete.

Then a feasibility domain (see *Figure 8*) was created to obtain all possible eccentricities and forces. From the feasibility domain, tendon profile (see *Figure 9*) is obtained.

From the suggested span to depth ratio, slab is designed as an 8.5” section. To provide 2 hour fire rating imposed on the assembly, 1.25” of cover is provided. Accounting for the cover, maximum eccentricity is determined to be 6” and required effective pre-stressing force of 24.30 k/ft. Due to varying spans, eccentricities as well as the number of tendons were changed to prevent overstressing the slab. Tendons spanning East to West are distributed evenly and the tendons spanning North to South are banded. Due to the curvature and set-back on the North façade, banded tendons are avoided in East-West direction.

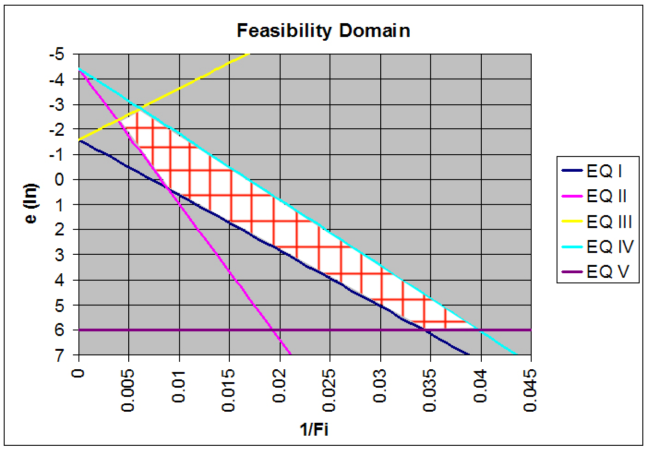


Figure 8

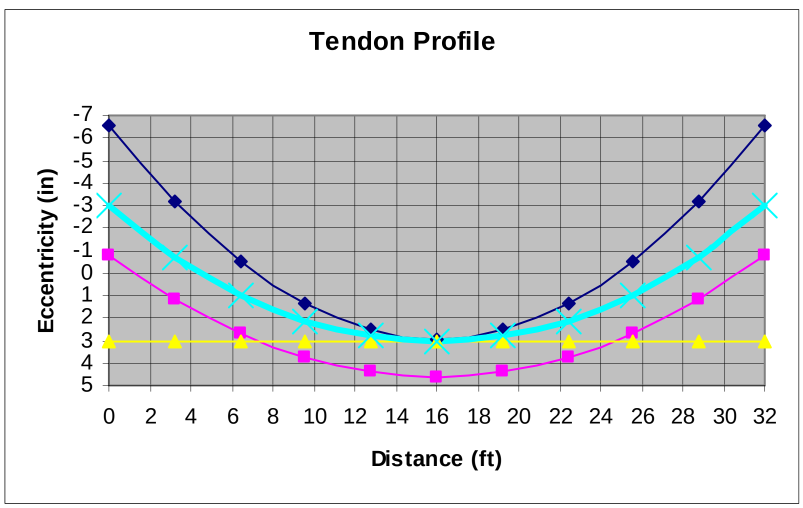
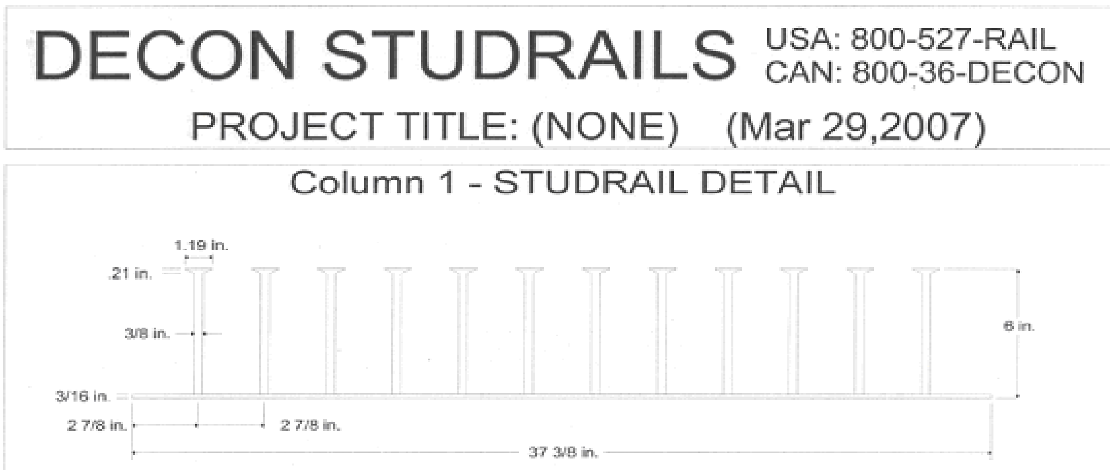


Figure 9

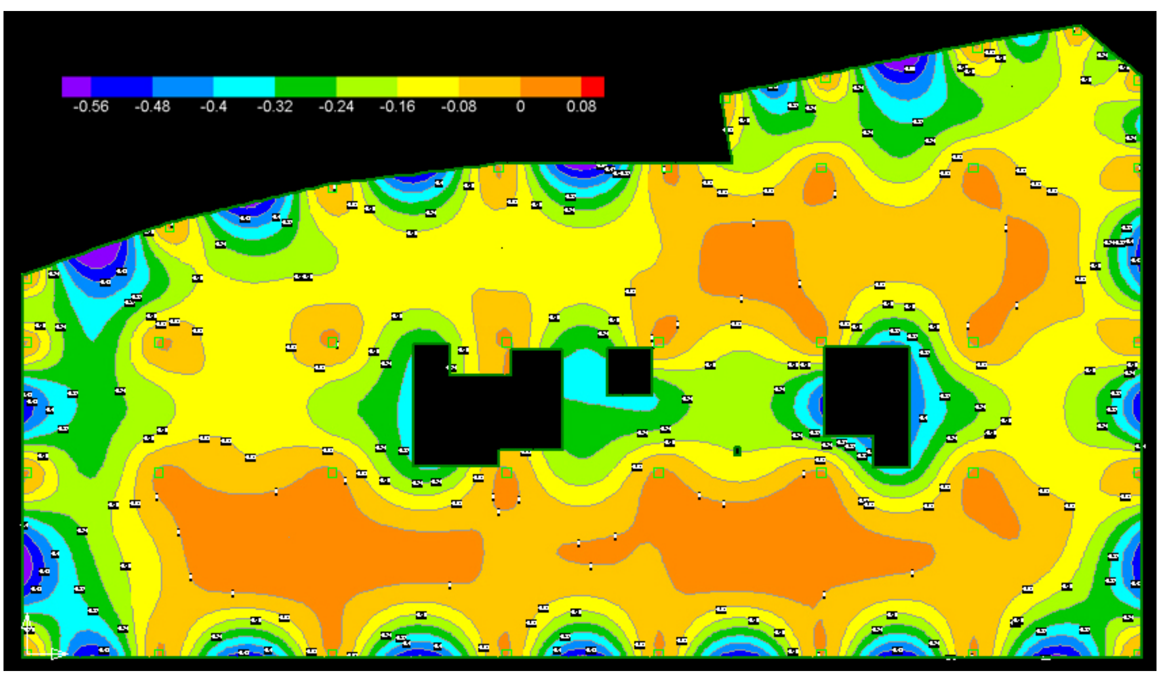
Strength calculations at initial, sustained, and service loads were calculated. Through the help of the feasibility domain, the four critical stresses were determined. 8.5” slab was able to handle the initial pre-stressing forces as well as loads at service. Also at ultimate the post-tensioned slab has enough capacity.

$$\varphi Mn \geq Mu$$
$$\varphi Mn \geq 1.2\ Mcr$$

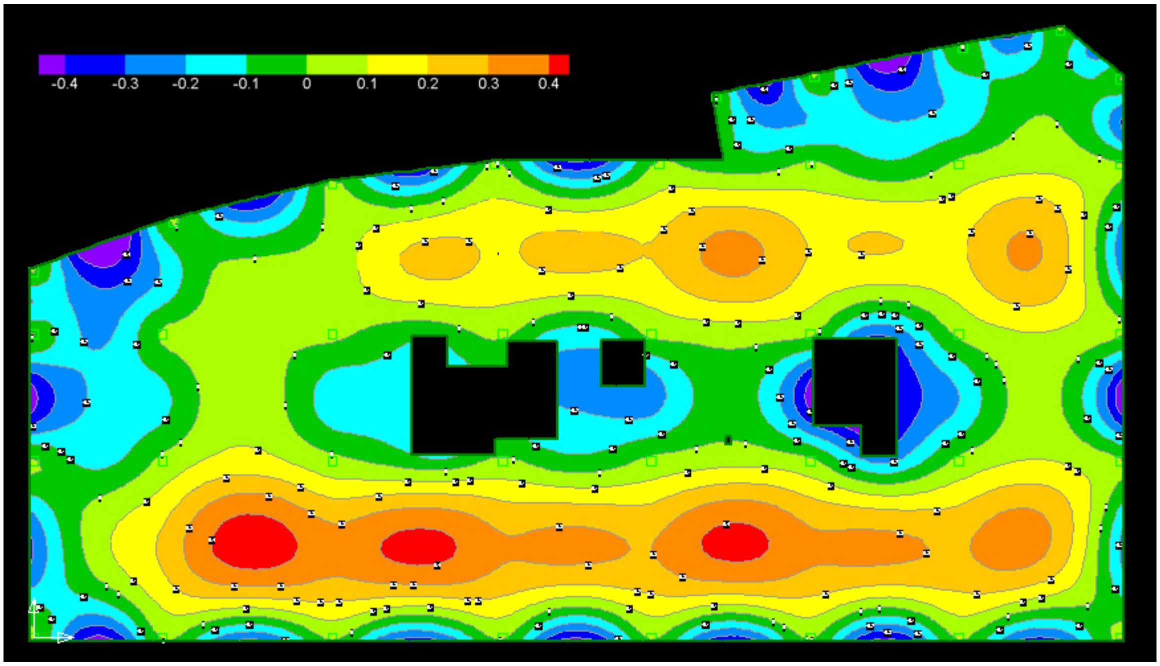
Punching shear was also considered in the design of the post-tensioned slab. After the columns were sized for gravity loads, the slab was analyzed for the necessity of shear reinforcement. Taking the column with the largest tributary area, punching shear was calculated. It was determined that shear reinforcement is need. Decon studrails were employed to maintain 20'x 20' column size.



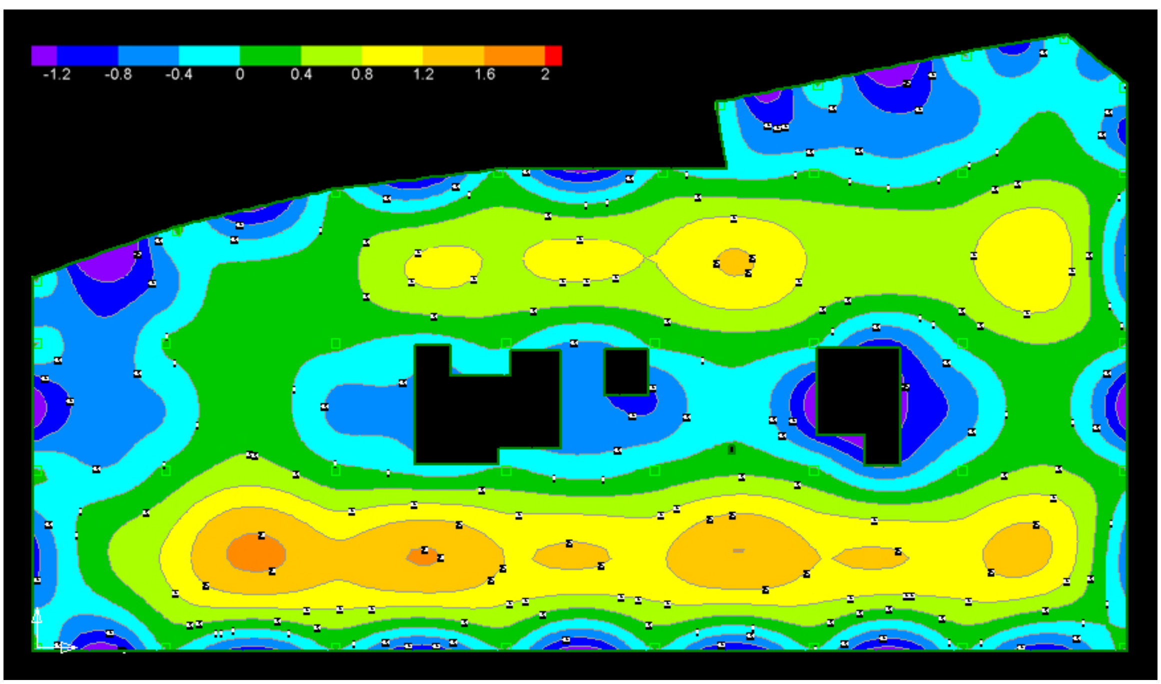
Finally slab deflections were investigated. Graphics below show, in order, deflection due to initial load, deflection due to sustained load, and long term deflection. RAM ConcePT was used to analyze and generate the color coded deflection diagram.



Initial

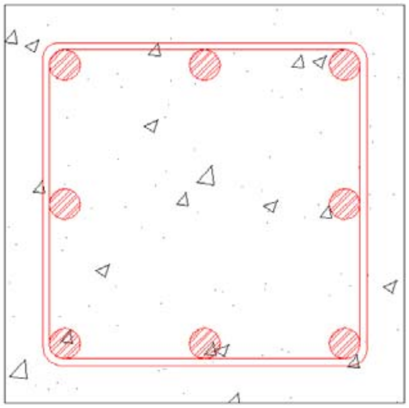


Sustained



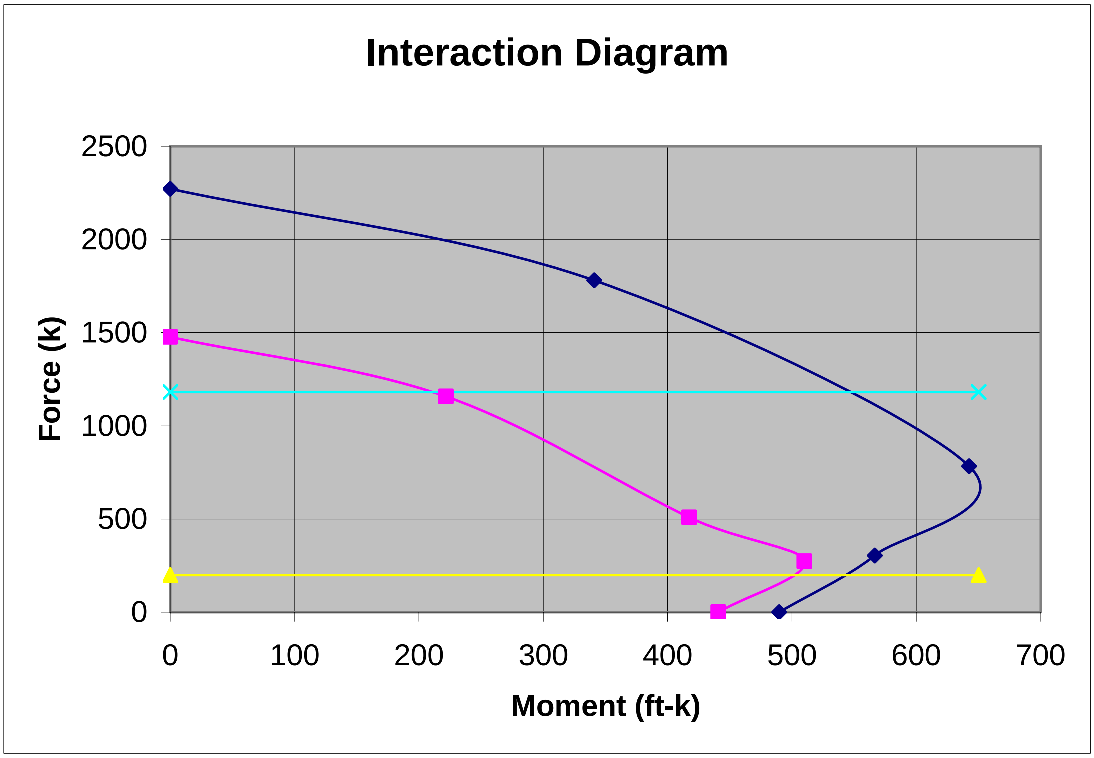
Long Term

Cast In Place Concrete Columns



Repetition in construction can reduce mistakes as well as increase productivity. Therefore one column size will be selected and reinforcements will vary. Axial force due to gravity load is applied to the columns and the moments from the post-tensioned slab framing into the columns are accounted for the design of columns. In calculating the axial force, reduction in live load was taken. Also in designing columns slenderness effect was taken into consideration. Using 20'x 20' column, moment multiplier was calculated to be 1.84. However due to small moment applied to the columns, the multiplier was not of great consequence. Using 5000psi strength concrete, column interaction diagram was created.

Calculated axial load for the interior column with greatest tributary area was found to be 1103 kips. The 20'x 20' column with 8 #11 bars was able to carry 1180 kips of axial load. The first two floors will use the specified reinforcing and the upper floors will contain columns with minimum reinforcing.



Shearwall Design

Cast in place shearwalls are located as shown in *Figure 10*. Two identical shearwalls resist seismic loads in North-South direction and two identical shearwalls resist seismic load in East-West direction. Applying the new lateral loads calculated for the concrete structure shearwalls resisting North-South lateral loads are 24’ long, 8” thick, and require minimum horizontal and vertical reinforcing which are #5 at 15” spacing. For the East-West lateral loads two 10’ long, 12” thick shearwalls with #5 at 10” spacing.

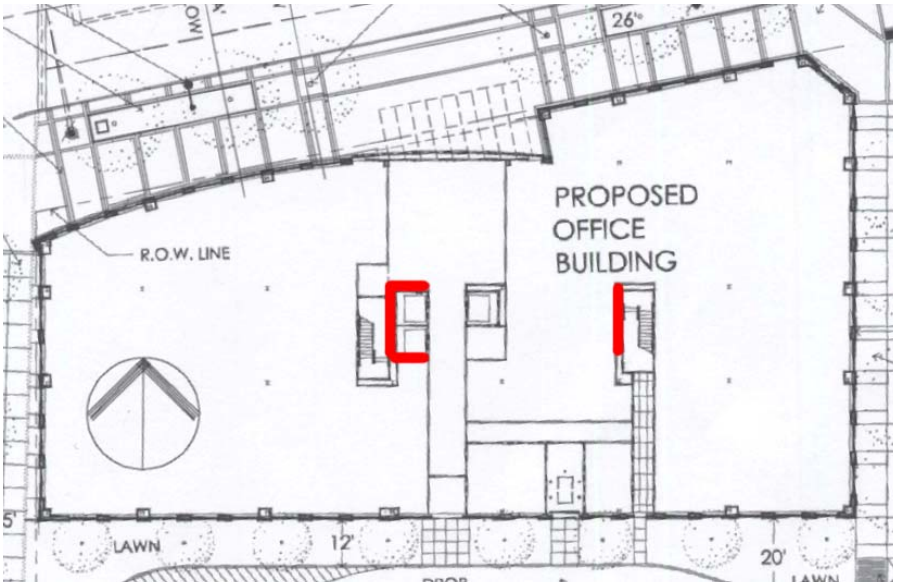


Figure 10

Foundation Impact

Due to a large increase in the weight of the building foundation sizes are increased. Some of the exterior and corner foundations which are over-designed for the sake of constructability remained the same size, however the interior spread footing sizes change dramatically. Approximately 350 CY of concrete is used for the foundations in the original steel structure. In concrete design approximately 1000 CY of concrete is placed.

BREADTH STUDY

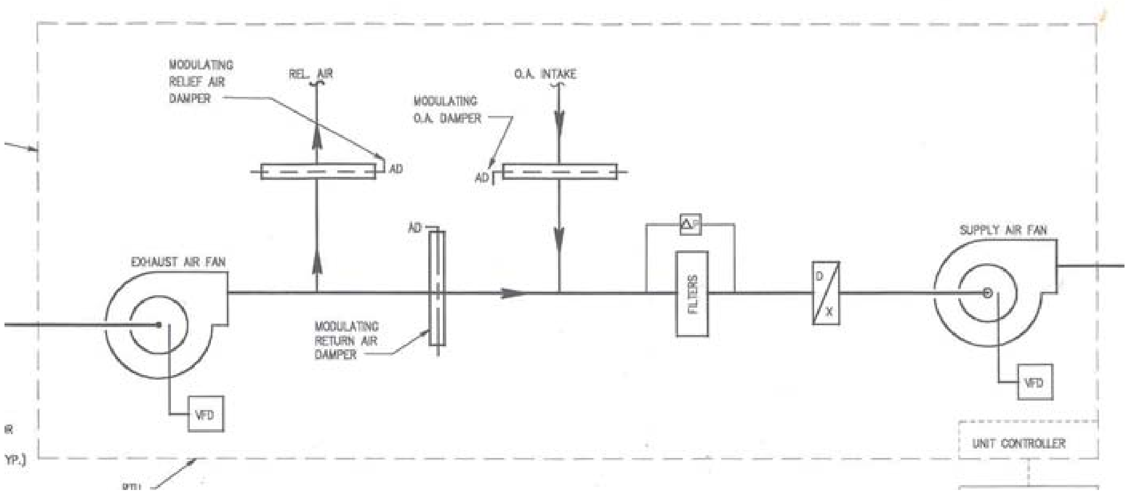
Mechanical

In consideration to the mechanical design, two studies are proposed. First option is to evaluate the impact of using enthalpy wheels. Upon investigation, minimum outside air did not appear to be too significant to have large energy savings from enthalpy wheels. In an attempt to verify the outside air requirement, minimum outside air was calculated. The below table was formed using the formula $V = R_p P_z + R_a A_z$.

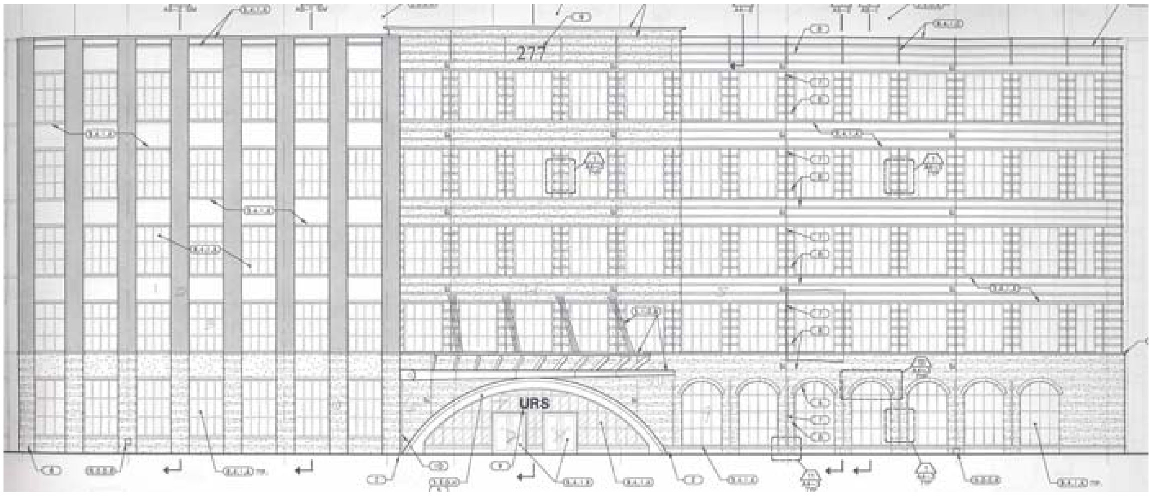
$$V_{oa} = (\text{cfm/person})*(\#\text{ of people}) + (\text{cfm/sf})*(\text{sf})$$

USE	Net S.F.	Assumed Occupants per SF	Rp (cfm)	Ra (cfm/sf)	Outside Air Req	Outside Air Supplied
Future Tenant	20000					
Office	20000	1/100	5	0.06	2200	3000
Office	20000	1/100	5	0.06	2200	3000
Office	20000	1/100	5	0.06	2200	3000
Office	20000	1/100	5	0.06	2200	3000
				total	8800	12000

8800 CFM of outside air is required to enter URS Office Building until a tenant moves into the ground floor. The actual outside air supplied is 12000 CFM meeting minimum requirement. Looking at the calculated required outside air which is about 10%, enthalpy wheel will not be pursued.



In redesigning the steel frame building into a post-tensioned concrete structure the floor depth was reduced significantly. Primary goal in this breadth study is to reduce energy cost. In reducing the floor depth building volume can be also reduced. Floor depth decreased from more than 25” to 8.5”. In an attempt to reduce building volume 12” per floor of wall will be shaved off. More of the wall could be removed to utilize the entire floor depth saving however to maintain the proportion of the building, only 12” height per floor of wall will be removed. Since the wall assembly is not shrunk uniformly, U-Value for the building must be calculated.



Calculate Original U Value of the building

$Q = U * A * \Delta T$

2576000 BTU/Hr $= U * 83200$ SF $* 25^{\circ}$F

$U = 0.619$

Determine the U Value of the wall

$U = \Sigma(Ui * A) / \Sigma A$

$U_{wall} = 0.038$

Determine the U Value of the building after 5' height of wall is removed

$U = \Sigma(Ui * A_{new}) / \Sigma A_{new}$

$U = 0.520$

$Q = 0.520 * (83200 - 5*600) * 25^{\circ}F = 1042600$ BTU/Hr

$Q = 1.08 * CFM * \Delta T$

$CFM = Q / (1.08 * \Delta T) = 1042600 / (1.08 * 25) = 38615$ CFM

Determine the ratio between old and new CFM and apply fan affinity law

$HP_1/HP_2 = (Q_1/Q_2)^3$

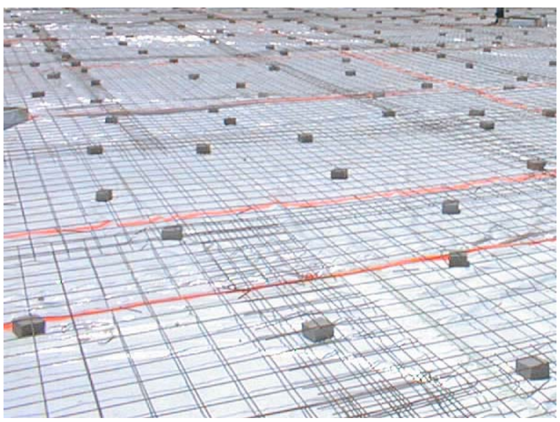
$160/HP_2 = (2576000/1042600)^3$

$HP_2 = 10.61$ HP

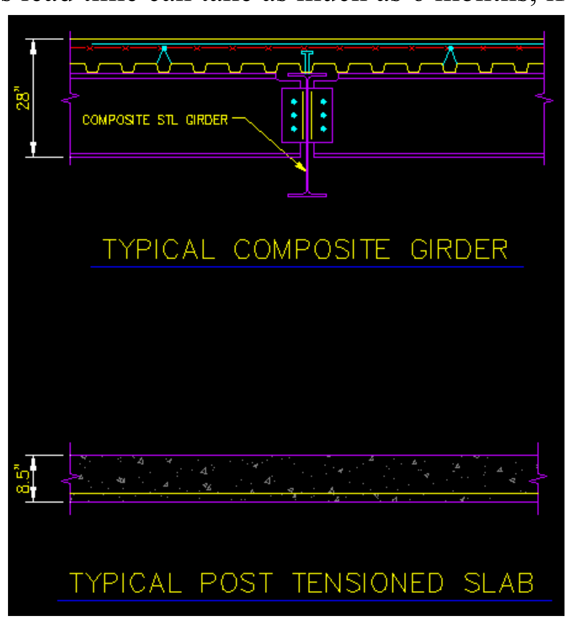
Construction Management

Originally the construction breadth was proposed to compare cost and schedule comparison between the existing steel and proposed concrete structure. Unable to receive the original cost and schedule, this breadth is comparison between the estimated steel construction cost and estimated concrete construction cost.

Primarily RS Means Building Construction Cost Data 2001 and RS Means Assemblies Cost Data 2001 were used to estimate cost and production. RS Means 2001 was used so accurate material cost for 2001 will be reflected. Also in using 2001 Edition, time factor need not be applied.



Both the steel and concrete structure has slab on grade. Therefore no advantage can be stated for either systems here. Producing at 92 CY per day, slab on grade can be finished in 6 day. For steel construction, steel erection took 27 work days to complete. This estimates about 3 pieces per hour and according to RSMeans Assemblies Cost Data, steel construction will cost $15.58 per SF. Also eliminating moment frames will reduce labor and equipment cost of welding steel. On the other hand concrete structure will cost $16.80 per SF and duration of 53 work days will complete the concrete framing. Although concrete frame take almost twice as long as steel frame to construct, lead time for steel must be accounted for. As lead time can take as much as 6 months, if the project needs to start concrete is a good option. Also with concrete design changes can be better handled than steel projects where structural steel member are made elsewhere and shipped over to the job site. As previously seen, concrete construction reduce floor thickness from more than 28” deep to 8.5” deep. This in turn decreases the volume of the building. The variable frequency drive fan, when reduced to pushing less air through the duct can save on energy costs.



There are so many variables and not having the original schedule and costs, it is very difficult to determine for certain which system will end up more cost efficient.

Conclusion and Recommendation

URS Office Building was analyzed both as steel and concrete structure. Throughout the analysis cost, time, constructability along with structural integrity was given utmost consideration. In laying out the lateral load resisting members architectural consequences were kept in mind and the explicit architectural constraints such as long spans and street facade free of obstruction were strictly followed. Breadth studies focused on cost savings from material, labor, equipment, and energy savings. Mechanical study was promising in that 160 horsepower motor can run at 11 horsepower if floor depth is reduced.

Of the three options existing structure, alternate steel lateral framing, and concrete design I would recommend the alternate steel lateral framing. Concrete design has its advantages in smaller floor depth, no lead time, and flexibility in the field. The existing structure is designed by experienced engineers working for the largest global engineering firm and its structural integrity and building performance is not questioned. Recommendation provided in this document is based on the calculated structural performance and information gathered while preparing the construction breadth. In the construction breadth steel was the cheaper and quicker solution. By eliminating moment frames, steel would see greater benefits in labor and equipment savings from bolting instead of welding. Also the rearrangement of the lateral members reduces eccentricity and torsion.

While the systems mentioned seem viable and good alternatives to each other, the variable factors such as market conditions, price fluctuation, and availability must be factored into the final decision. Without the actual schedule and cost, the comparison made in this document between the steel and concrete options are not exact

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Naaman, Antoine E. Prestressed Concrete Analysis and Design, Second edition.

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PCI Design Handbook 5th Edition

RSMeans Assemblies Cost Data 2001

RSMeans Building Construction Cost Data 2001

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I am so grateful for everyone who has helped me through the thesis process. I know without your assistance this project would not have happened. Once again I thank you for your instruction, patience, support and most of all your time.

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Professor Jelena Srebric

Professor John Messner

AE Students

Sean Flynn

Brian Ault

A special thanks to friends and family that's been the source of encouragement and strength.

APPENDICES

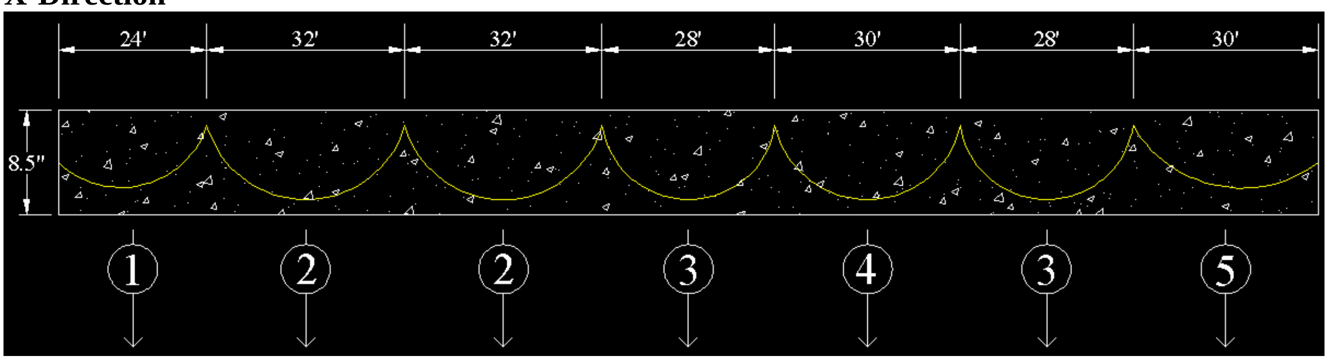
POST-TENSIONED 2 WAY FLAT SLAB

Suggested Thickness = $\frac{span}{45} = \frac{33'*12}{45}$ = 8.8” → try **8.5” slab**

Loads → **Self Weight** = 150 PCF*8.5”/12= **106.25 PSF**
Super Imposed Dead Load = **20.00 PSF**
Live Load = **50.00 PSF**
Total Load = **176.25 PSF**

$\mathbf{W_{pre}}$ = 0.9*Self Weight = 0.9*106.25 = 95.625 PSF → USE **95.25 PSF**
$\mathbf{W_{net}}$ = Total Load - W_{pre} = 176.25–95.25 = **81 PSF**

X-Direction



SPAN 1
W = 81 PLF
M = 5.83 ft-k
a_{max} = 4.5”
F = 15.55 k/ft
F/A = 152 psi

SPAN 2
W = 81 PLF
M = 10.37 ft-k
a_{max} = 6”
F = 20.74 k/ft
F/A = 203 psi

SPAN 3
W = 81 PLF
M = 10.37 ft-k
a_{max} = 6”
F = 20.74 k/ft
F/A = 203 psi

SPAN 4
W = 81 PLF
M = 9.11 ft-k
a_{max} = 6”
F = 18.23 k/ft

SPAN 5
W = 81 PLF
M = 9.11 ft-k
a_{max} = 4.5”
F = 24.30 k/ft

F/A = 179 psi F/A = **238 psi**

Max Stress per tendon → Initial Stress – Losses = fpi – Losses = fpe
fpi = 0.74*fpu = 0.74*270 ksi = 200 ksi
Losses = 30 ksi ← suggested loss by prestressed concrete text
fpe = 200–30 = 170 ksi

Fe = fpe*As = 170000 psi * 0.153 in^2/tendon = 26010 lb/tendon

$$\frac{F}{Fe} = \frac{24276}{26010} = 0.93 \ \frac{tendon}{ft} \rightarrow \text{spacing} = 1.07 \frac{ft}{tendon}$$

$$f = -238 \text{ psi} \pm \frac{87550}{2*8.5*8.5} = -238 \pm 606 = \frac{424\,psi \le 6*\sqrt{f'c}}{844\,psi \le 0.45*f'c}$$

@ULTIMATE

$$Mcr = F(e_o - k_t) - f_r z_b = 24300*(3+1.42) + 7.5*\sqrt{5000}*144.5 = 184039 \text{ in-lb}$$

$$Mn = A_{ps}*f_{ps}*(d_p - \frac{a}{2}) = 0.142*262300*(7.25 - \frac{0.859}{2}) = 254040 \text{ in-lb}$$

$$f_{ps} = 270*(1 - \frac{0.28}{0.85}*0.0016*\frac{270}{5}) = 262.3$$

$$a = \frac{0.142*262.3}{0.85*5*12} = 0.73 \rightarrow c = \frac{a}{\beta} = \frac{0.73}{0.85} = 0.859$$

$$\frac{c}{d} = \frac{0.859}{7.25} = 0.119 < 0.375 \rightarrow \phi = 0.9$$

ϕMn = 0.9*254040 = 228636 in-lb = 19.05 ft-k
Mu = 87550 in-lb = 7.3 ft-k

2 CHECKS FOR ULTIMATE

1. **ϕMn = 19.05 ft-k > Mu = 7.3 ft-k**
2. **ϕMn = 19.05 ft-k > 1.2 Mcr = 1.2*15.34 = 18.41 ft-k**

SHEAR (20”x 20” columns)

$Vc = (3.5*\sqrt{f'c} + 0.3\sigma_G)*b_w*d_p + Vp$
$= (3.5*\sqrt{5000} + 0.3*238.24)*109*7.25 + 759 = 252817$ lb = 252.82 kip

$$\sigma_G = \frac{F}{Ac} = \frac{24300}{8.5*12} = 238.24 \text{ psi}$$

$$Vp = F*\sin\alpha = 24300*\frac{0.5}{16.01} = 759 \text{ lb}$$

$Vu = (1.2*126.25 + 1.6*50)*(32*29 - 2^2) = 213906$ lb = 214 kip

$$Vs = \frac{Vu - \phi Vc}{\phi} = \frac{214 - 0.75*252.82}{0.75} = 32.51 \text{ kip}$$

$$Vs = 32513 \text{ lb} = \frac{Av*fy*dp}{s} \rightarrow Av = \frac{32513*5}{7.25*60000} = 0.374 \text{ in}^2$$

$s \leq 0.75*d = 0.75*7.25 = 5.43$" → USE **5"**
fy = 60000 psi
$d_p = 7.25$"

*****PROVIDE SHEAR REINFORCEMENT*****

COLUMN SIZING

If $\frac{KL}{r} \leq 34\text{–}12 * \frac{M1}{M2}$ is true, slenderness effects need not be accounted

$$\frac{KL}{r} = \frac{1*(16*12)}{h} * \sqrt{12} \leq 34\text{–}12*1 = 22$$

$$h \geq \frac{1*(16*12)}{22} * \sqrt{12} = 30.23''$$

*****since column is 20"x 20" calculate slenderness effect*****

$Mc = \delta_{ns}*M2 = 1.69*M2$

$$\delta_{ns} = \frac{Cm}{1-\frac{Pu}{0.75*Pc}} = \frac{1}{1-\frac{1103}{0.75*3597}} = 1.69$$

Cm = 0.6+0.4(M1/M2)

$$Pc = \frac{\pi*\pi*EI}{KL*KL} = \frac{\pi*\pi*1.34*10\text{^}10}{16*12*16*12} = 3596872 \text{ lb} = 3597 \text{ kip}$$

$$EI = \frac{0.4*Ec*Ig}{1+\beta d} = \frac{0.4*4030509*13333}{1+0.6} = 1.34*10^{10}$$

$$Ec = 57000*\sqrt{f'c} = 57000*\sqrt{5000} = 4030509 \text{ psi}$$

$$Ig = \frac{20*20\text{^}3}{12} = 13333$$

LATERAL LOAD – CONCRETE STRUCTURE

h = 76.67 ft
Sa = 12%
S_1 = 4.6%
Class D
Building Category II
Seismic Use Group I
I = Importance Factor = 1.0
R = 5
Cd = 4.5
Ct = 0.02
x =0.75

$Ta = Ct*h^x = 0.02*(72)^{0.75} = 0.494$

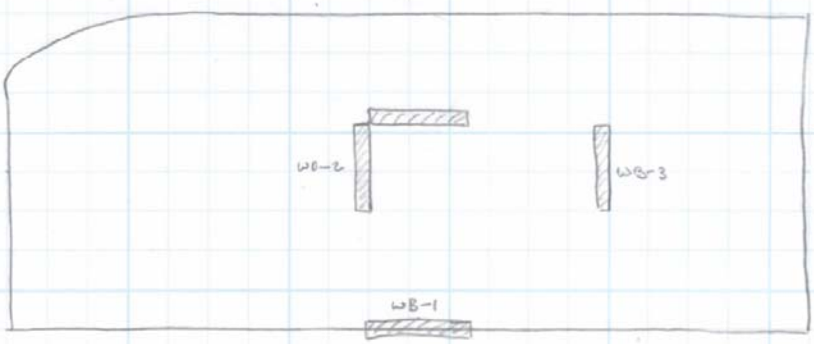
$$Cs = \frac{Sds}{R/I} = \frac{0.128}{5/1} = 0.0256$$

V = Cs*w = 0.0256*12625 = 323.2 kip

Floor	Load (PSF)	Area (SF)	w (kip)	h (ft)	w*h (ft-k)	Cvx	F (kip)	Story Shear (kip)
2	126.25	20000	2525	16	40400	0.072727	23.50545	323.2
3	126.25	20000	2525	30	75750	0.136364	44.07273	299.6945455
4	126.25	20000	2525	44	111100	0.2	64.64	255.6218182
5	126.25	20000	2525	58	146450	0.263636	85.20727	190.9818182
Roof	126.25	20000	2525	72	181800	0.327273	105.7745	105.7745455

*****BASE SHEAR = 323 kip*****

Alternate Steel Lateral Framing

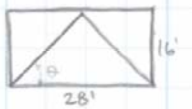


$K = \sum AE\cos^2\theta / L$

(*) Assume columns are infinitely stiff

WB-1

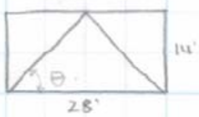
$K_1 = (2 \times 10.4\ in^2)(29000\ ksi)\cos^2(48.81°)/(21.26' \times 12) = 1025.43$



TS → 8×8×3/8
A → 10.4 in²
$\theta \to \tan^{-1}(\frac{16}{14}) = 48.81°$

$L \to \sqrt{14^2 + 16^2} = 21.26'$
E → 29000 ksi

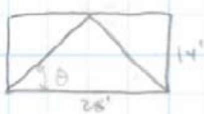
$K_2 = K_3 = (2 \times 7.58\ in^2)(29000\ ksi)\cos^2(45°)/(19.80' \times 12) = 925.17$



TS → 6×6×7/8
A → 7.58 in²
$\theta \to \tan^{-1}(\frac{14}{14}) = 45°$

$L \to \sqrt{14^2 + 14^2} = 19.80'$
E → 29000 ksi

$K_4 = K_5 = (2 \times 5.24\ in^2)(29000\ ksi)\cos^2(45)/(19.80' \times 12) = 639.56$



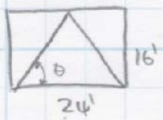
TS → 6×6×1/4
A → 5.24 in²
θ → 45°

L → 19.80'
E → 29000 ksi

$\Sigma K = 1025.43 + (2 \times 925.17) + 2(639.56) = 4154.89$

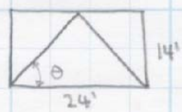
WB-2

$K_1 = (2 \times 10.4\ in^2)(29000\ ksi)\cos^2(53.13°)/(20' \times 12) = 904.80$



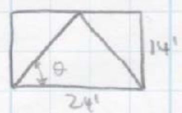
TS → 8×8×3/8
A → 10.4 in²
$\theta \to \tan^{-1}(\frac{16}{12}) = 53.13°$
$L \to (12^2 + 16^2)^{1/2} = 20'$
E → 29000 ksi

$K_2 = (2 \times 7.58\ in^2)(29000\ ksi)\cos^2(49.40°)/(18.44' \times 12) = 841.43$



TS → 6×6×3/8
A → 7.58 in²
$\theta \to \tan^{-1}(\frac{14}{12}) = 49.40°$
$L \to \sqrt{12^2 + 14^2} = 18.44'$
E → 29000 ksi

$K_3 = K_4 = K_5 = (2 \times 5.24\ in^2)(29000\ ksi)\cos^2(49.40°)/(18.44' \times 12) = 581.67$



TS → 6×6×1/4
A → 5.24 in²
θ → 49.40°
L → 18.44'
E → 29000 ksi

$\Sigma K = 904.80 + 841.43 + (3 \times 581.67) = 3491.24$

WB-3

$K_1 = (2 \times 7.58\ in^2)(29000\ ksi)\cos^2(53.13°)/(20' \times 12) = 659.46$

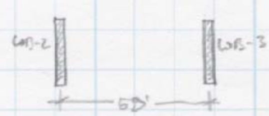
TS → 6×6×3/8
A → 7.58 in²
θ → 53.13°
L → 20'
E → 29000 ksi

$K_2 = K_3 = K_4 = K_5 = (2 \times 5.24\ in^2)(29000\ ksi)\cos^2(49.40°)/(18.44' \times 12) = 581.6$

TS → 6×6×1/4
A → 5.24 in²
θ → 49.40°
L → 18.44'
E → 29000 ksi

$\Sigma K = 659.46 + (4 \times 581.67) = 2986.14$

$$\frac{\Sigma K_{WB-2}}{\Sigma K_{WB-2} + \Sigma K_{WB-3}} = \frac{3491.24}{3491.24 + 2986.14} = 0.54$$



$0.54 \times 58' = 31.32'$

TORSION SEISMIC

$M_x = eP = 18.60' \times 169^k = 3143^{'k}$

$M_y = eP = 9.42' \times 169^k = 1592^{'k}$

ACCIDENTAL → $M_{at_x} = 0.05(114.67')(169^k) = 968.96^{'k}$

$M_{at_y} = 0.05(204.67')(169^k) = 1729.46^{'k}$

$T_x = 4112^{'k}$

$T_y = 3321^{'k}$

RELATIVE STIFFNESS → WB-1 = 1.39

WB-2 = 1.17

WB-3 = 1

WB-4 = 1.39

$J = 1.39(28.70')^2 + 1.17(31.32')^2 + 1(26.68)^2 + 1.39(28.70')^2 = 4149.38$

$F_{WB-1} = \frac{1.39(28.7)P}{4149.38} = 0.0096P = 1.62^k$

$F_{WB-2} = \frac{1.17(31.32)P}{4149.38} = 0.0088P = 1.49^k$

$F_{WB-3} = \frac{1(26.68)P}{4149.38} = 0.0064P = 1.09^k$

$F_{WB-4} = \frac{1.39(28.7)P}{4149.38} = 0.0096P = 1.62^k$