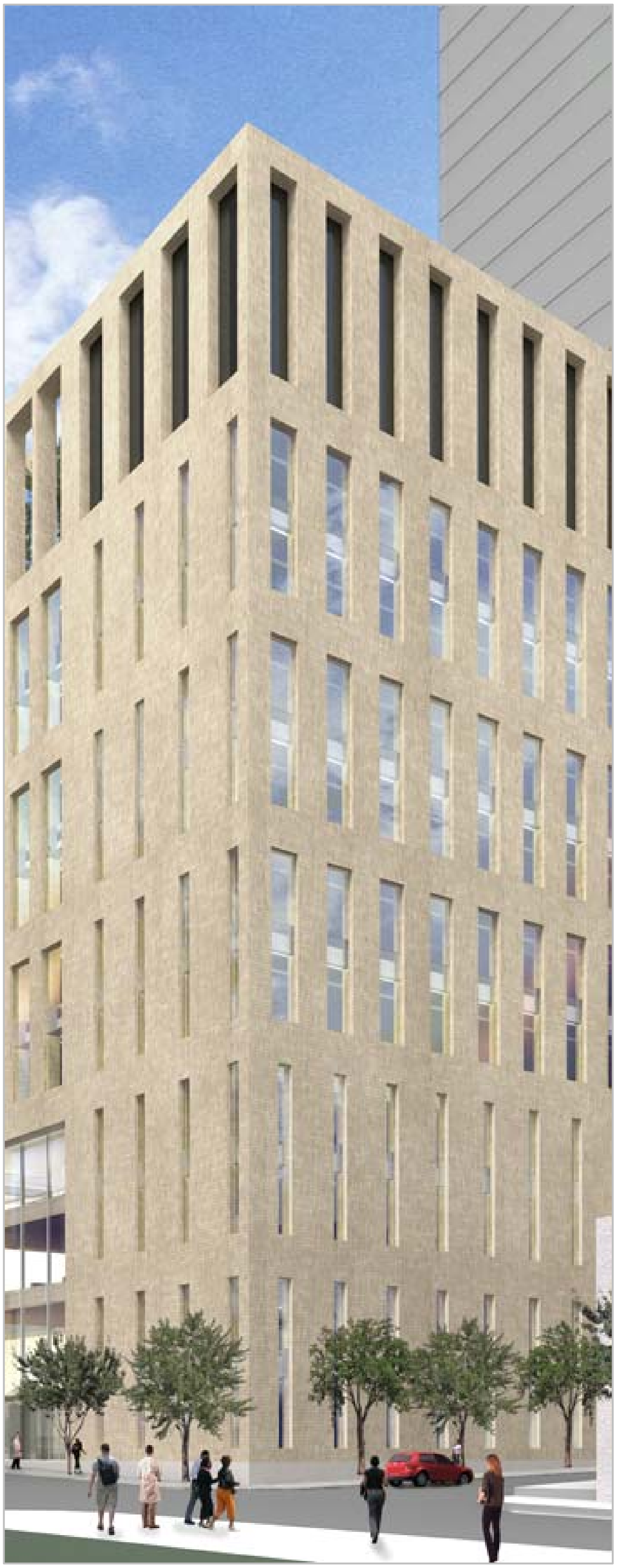
# center for science & medicine

## new york, ny



Optimization of Building
Systems & Processes

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# center for science & medicine
## new york, ny

### project team

architect / structural engineer: SOM

mep engineer: Jaros Baum & Bolles

civil engineer: Langan Engineering

lighting consultant: SBLD Studio

general contractor: Bovis Lend Lease

### statistics

location: manhattan's upper east side

levels: 11 stories above, 2 stories below grade

size: 443, 291 sq ft

construction dates: april 2008 – july 2011

delivery method: design-bid-build

project phase: 50% construction documents

### architecture

4-story atrium rises from Madison Avenue entrance

6 floors of wet lab research space;1 ½ floors of clinical trial area

green roof & rooftop terrace

facade comprised of brick-faced precast concrete & vertical window walls

a 40-story residential tower will rise on the site adjacent to the lab building

project is striving for LEED certification (gold)

all trades are coordinating with 4-D design techniques (BIM)

### structural

structural steel framing with composite metal deck / nwc topping

reinforced concrete spread footings at a maximum depth of 49'-0" below grade

typical floor heights are 15' above grade & 24' below grade

typical beams range from W18 to W30 in size, spaced at 10'-6" on center

typical columns range from W14 to W24 sections, spaced in 21'-0" bays

lateral resistance provided by a combination of braced and moment resisting structural steel frames

floor systems designed to meet stringent vibration criteria in laboratories / imaging rooms (2,000 – 8,000 micro-inches/sec)

### mechanical

laboratories, vivariums, and imaging spaces designed to use "once through" supply and exhaust systems, 100% outdoor air (12 systems total)

atrium, conference, and amenity spaces designed to use supply and return systems (3 systems total)

much of CSM's complex mechanical equipment will be located in the adjacent residential tower below a height of 160 ft, minimizing the need for additional height and/or excavation

remaining equipment to be housed in CSM's 11th flr penthouse

### lighting/electrical

power distributed by three 5 kV feeders

277/480 V, 3 phase, 4 wire system stepping down to 120/208 V for receptacles and incandescent lighting

laboratory floors will be served by an enclosed plug-in busway system, with a minimum of 3 takeoff positions per floor, run vertically through building

lighting fixtures are fluorescent, high intensity discharge lamps (277V) and incandescent lamps (120 V)

an on-site emergency generator plant utilizing two 1,200 kW diesel engine-generator sets will be provided

http://www.engr.psu.edu/ae/thesis/portfolios/2008/alb381/

ashley bradford | structural

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## EXECUTIVE SUMMARY

The Center for Science & Medicine is an 11-story research laboratory located in New York City's Upper Manhattan. Situated within the building is a spacious lobby area, 6 floors of wet lab research space, 1½ floors of clinical space, a clinical trial area, and space for research imaging. The building stands a total of 184'-0" above grade, with a typical floor to floor height of 15'-0." It is a steel structure designed with a core of braced frames in the center of the building and moment frames around the perimeter. The footprint of CSM is approximately 172 feet by 200 feet.

*Courtesy of Skidmore, Owings and Merrill, LLP.*

The primary goal of this report is optimization of existing design. Several building systems and processes will be evaluated and redesigned with efficiency as a driving factor. Specifically, the optimization of the following items will be addressed:

- Lateral load resisting system
- Construction means & methods of this system
- The design and coordination process
- A typical laboratory lighting system

### Depth Study: Lateral System Re-Design

At its current phase of design, the Center for Science and Medicine has been planned to utilize a combination of perimeter moment frames and core braced frames to resist lateral loads. After careful study of this system, it has been determined that moment frames are not significantly stiff, due to their double-heighted configuration, and braced frames pose coordination headaches as well as constructability issues. Therefore, an alternative system will be proposed to in an attempt to eliminate these inefficiencies.

The lateral system re-design consists of a core-only system of coupled shear walls which replace the braced frames currently existing at the building core. These shear walls are designed to resist 100% of the lateral load in both directions, therefore also eliminating the need for perimeter moment frames. It has been determined that the proposed core-only system provides more stiffness than the current dual system, provides added resistance to uplift, and presents a more efficient means of lateral force resistance. Moreover, the proposed design is expected to require less construction time while saving cost in the elimination of expensive moment connections and heavy framing members.

### Breadth Study 1: Construction Management & Building Information Modeling (BIM)

One of the unique aspects of the Center for Science & Medicine is that it has been designed in 3D, utilizing BIM (building information modeling) technology. Since this is a relatively new design tool in an industry based on historically-rooted standards and practices, it is a question as to whether this cutting-edge design method will truly pay off. This breadth study investigates the BIM implementation techniques used on this project by Skidmore, Owings and Merrill, evaluates the advantages and disadvantages of the technology, and identifies lessons learned by the project team. From the research conducted, it was determined the BIM is, indeed, a valuable and pertinent design tool with its potential benefits far greater than its shortcomings. Building information modeling is the future of the AEC industry, and the successful implementation of the technology by SOM can serve as an example to other firms adopting the software.

### Breadth Study 2: Laboratory Lighting Redesign

Lighting can be critical in laboratory spaces, where important procedures are carried out and visibility is critical. There are 6 typical "wet" laboratory spaces in the Center for Science and Medicine, and investigation has determined that the lighting systems of these spaces have actually been overdesigned, almost with too much care. Illuminance levels on the work plane exceed IES target ranges for laboratories, and lighting power density exceeds the limit set forth in ASHRAE Standard 90.1-2007. This breadth study proposes an alternative design to the existing lighting system in an attempt to reduce illuminance levels and LPD. The redesign will consider efficiency, aesthetics, and environment in the selection of new luminaires. Final design of the space successfully achieved target illuminance levels and lighting power density. Thus, this optimized system can be put in place throughout the building at every wet lab location to reduce energy use and better comply with industry standards.

## 1.0 INTRODUCTION

The Center for Science & Medicine is a research laboratory designed for scientific investigation, discovery, and treatment. Located in New York City's Upper Manhattan, the building is organized and shaped by its architectural program. On the north and south edges of the site, two linear lab bars encompass a core of support spaces. The building's east edge links the inside to the outside with a window-covered, multi-story atrium. Situated within the building are 6 additional floors of wet lab research space, 1½ floors of clinical space, a clinical trial area, and space for research imaging. The building is 11 stories above grade with a typical floor to floor height of 15'-0", giving a total building height of 184'-0." A 40-story residential tower will also rise on the site adjacent to the lab, but the buildings are clearly defined as two separate entities. Below is a site plan showing the CSM research center, the adjacent residential tower, outdoor service areas, and surrounding buildings.

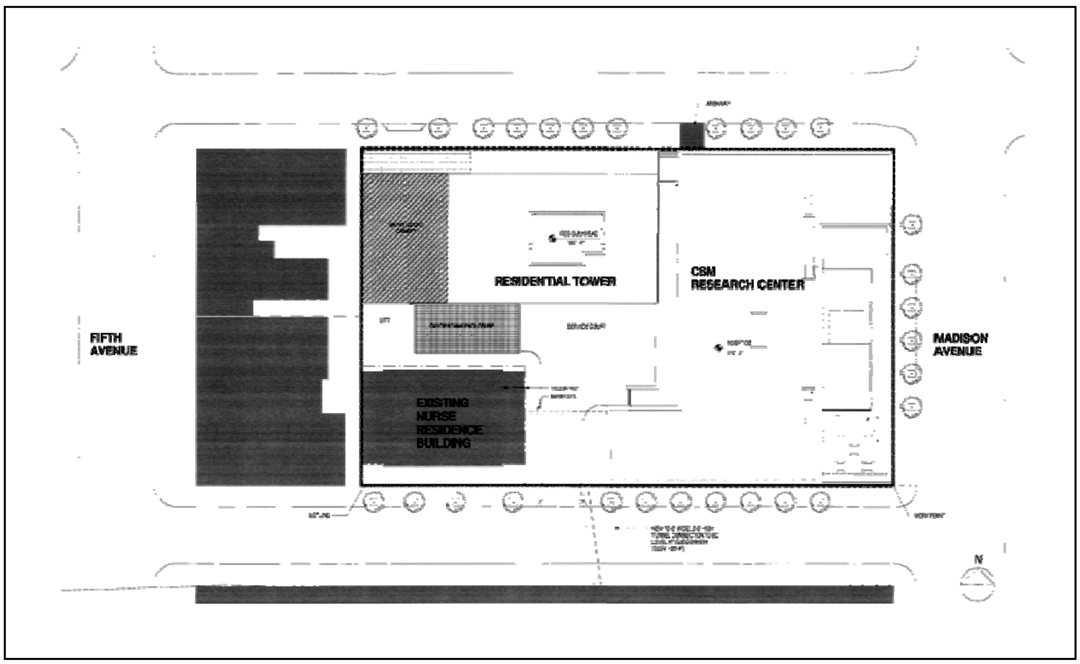



*Figure 1.1: Site Plan*

It is important to note that the Center for Science & Medicine, or the CSM, is only at the 50% construction document (CD) phase. Thus, all interpretations and calculations made within this report are based on information that has not been finalized or made absolute.

# 2.0 EXISTING STRUCTURAL SYSTEM

## 2.1 Foundation

The foundation consists of reinforced concrete spread footings ranging from 4'x4'x2' to 8'x8'x4' (l x w x h) in size, with a concrete compressive strength of $f'_c = 5000$ psi. Maximum footing depth is 49'-0" below grade, and all footings bear on sound bedrock (Class 2-65 rock with bearing capacity 40TSF or Class 1-65 rock with bearing capacity 60TSF, according to New York City Building Code). Seven of the total forty-three footings have been designed to support columns from both the research center and the residential tower, as dictated by their location at the CSM / tower interface. Foundation loads vary from 400 to 3200 kips.

Below grade perimeter walls consist of cast-in-place, reinforced concrete ($f'_c = 5000$ psi) braced by the below-grade floor slabs. The walls stand 48 ft in height (equivalent to 4 basement levels). These walls have been designed to resist lateral loads from soil and surcharge in addition to the vertical loads transferred from perimeter columns above. On the north and south perimeter walls, reinforced concrete pilasters support perimeter columns above. A continuous grade beam ($f'_c = 5000$ psi) supports these perimeter basement walls.

The lowest level basement floor is an 8" concrete slab on grade with a compressive strength of $f'_c = 4000$ psi, typically reinforced with #5 bars@12" each way. At typical columns, additional slab reinforcement is provided with (4)#4 bars around the column base. At lateral columns located around the building core, the slab is reinforced with (12)#5 bars with additional longitudinal bars arranged in a grid pattern around the column base.

## 2.2 Floor Framing System

The CSM's existing floor system uses composite metal deck. The floor slabs typically consist of 3" metal deck with 4 ¾" normal-weight concrete topping, giving a total slab depth of 7 ¾". Thicker, normal-weight concrete slabs will be provided in spaces such as mechanical floors to meet acoustic and vibration criteria. These thickened slabs will be designed with 3" metal deck and 8" NWT concrete topping with reinforcement, giving a total slab depth of 11". Full composite action is created by 6" long, ¾" diameter shear studs, and concrete compressive strength is $f'_c = 4000$ psi. The composite metal deck is supported by wide flange steel beams ranging from W12x14 to W36x150 in size and spaced approximately 10'-6" on center.

There are two typical bay sizes used throughout the building, 21'-0"x 21'-0" and 43'-8" x 21'-0." Square bays typically occur within the building core, and rectangular, longer span bays typically occur around the building perimeter where research labs and clinical spaces are located. All floor framing has been designed to meet stringent vibration limits, due to the sensitivity of laboratory equipment located throughout the building, and these requirements are outlined further into the body of this report.

## 2.3 Lateral System

Lateral resistance to wind and seismic loads is provided by a combination of braced and moment resisting steel frames. Refer to the plan on the right for the location of each lateral element and its label. Braced frames are shown in red, and moment frames are shown in blue.

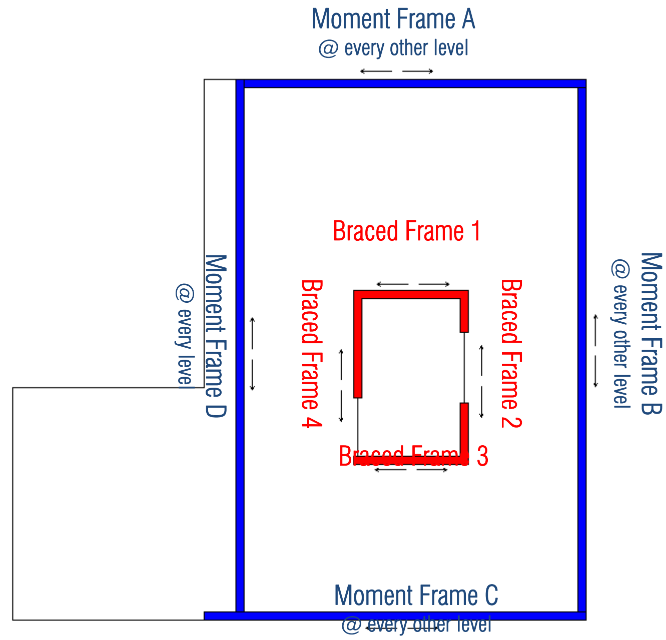



*Figure 2.1: Lateral Framing*

Braced Frames. In both the North-South and East-West directions, lateral loads are resisted by diagonally-braced frames located around the building core. The majority of the braced frames are braced concentrically, but some of the frames are eccentrically braced due to architectural needs (space for doors, etc.). The core is made up of (6) column bays spaced at approximately 20'x20' and using W14 column sections. Heavy double tee sections serve as diagonal braces at the core and vary from WT6x39.5 to WT6x68 in size.

<u>North-South Direction</u>

Braced Frame 2

Braced Frame 4

<u>East-West Direction</u>

Braced Frame 1

Braced Frame 3

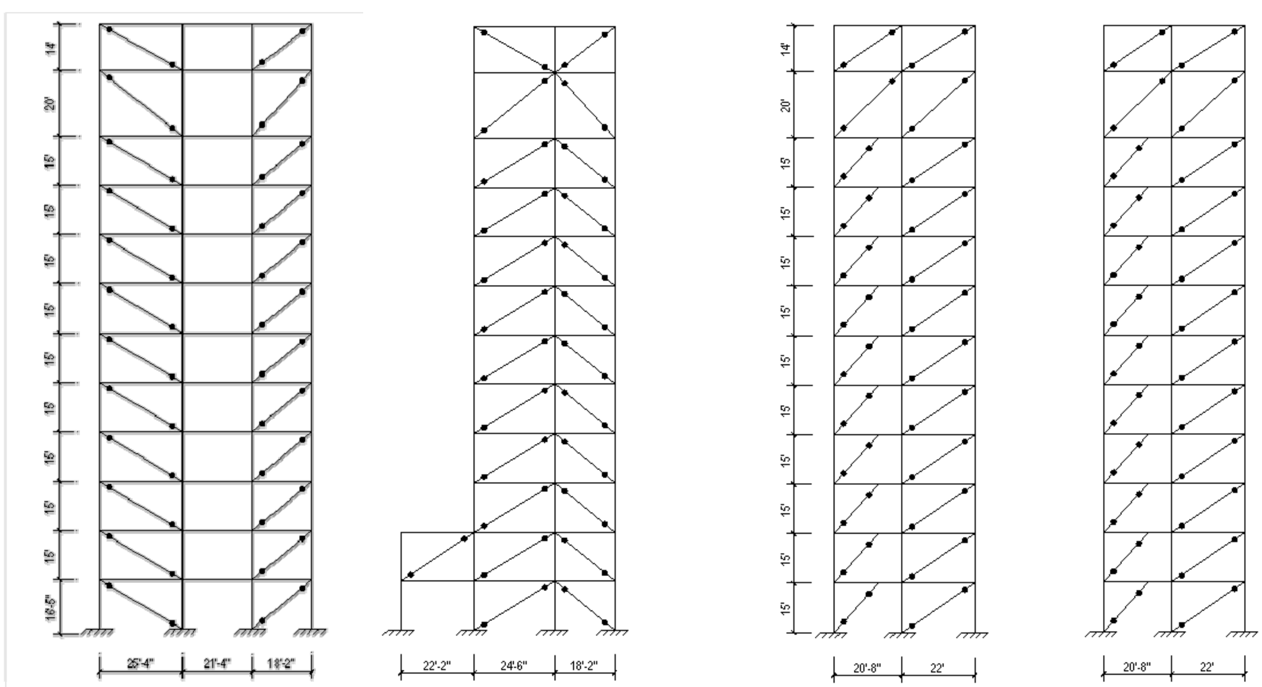


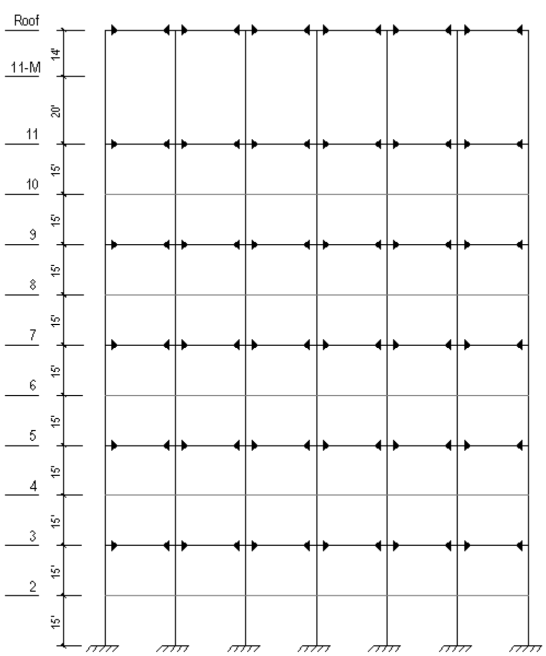

*Figure 2.2: Braced Frames (Note: sub-grade levels not shown)*

**Moment Frames.** In both the North-South and East-West directions, remaining lateral loads not resisted by braced frames are taken by a system of beam/column moment frames located at the perimeter of the building (or just inside of it, see Moment Frame D). These moment frames have been designed to use W14 or W24 column sections spaced approximately 21'-0" on center and W30 and W24 wide flange beams. What makes these frames unique is their double-heighted configuration. The first moment connections occur on the third level and then occur only on alternating levels up through the building's roof (a total of six floors with moment connections). Thus, instead of each moment frame being 15'-0" in height (as they would have been if occurring at each floor), the moment frames are actually 30'-0" in height. Shear connections occur on even-numbered levels where spandrel breams are set back (framing into girders), thus providing no contribution to lateral resistance at these locations.

Such a double-heighted frame configuration was necessary for CSM because of architectural design. The exterior cladding is a "perforated" system, meaning that the aesthetic pattern spans the height of two floors and the framing of every other level is visible through the windows. In other words, the exterior appears to be punched, or perforated, by alternating floor levels. For this reason, moment connections had to be placed at every other level, with intermediate levels framed by spandrel beams set back from the frame. Although this is not a desirable design from a structural point of view, it seemed to be the best solution that would satisfy both the structural integrity and the aesthetic appeal of the building. The diagrams below depict moment frames with dark lines and arrow heads, while intermediate levels (without moment connections) are grayed.

<u>East-West Direction</u>

Moment Frame A




Moment Frame C

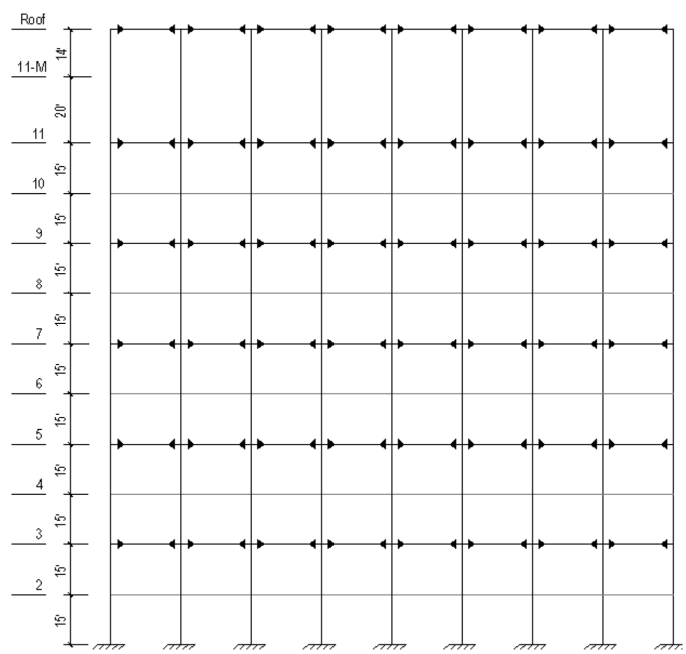



*Figure 2.4a: Moment Frames*

North-South Direction

Moment Frame B

Moment Frame D

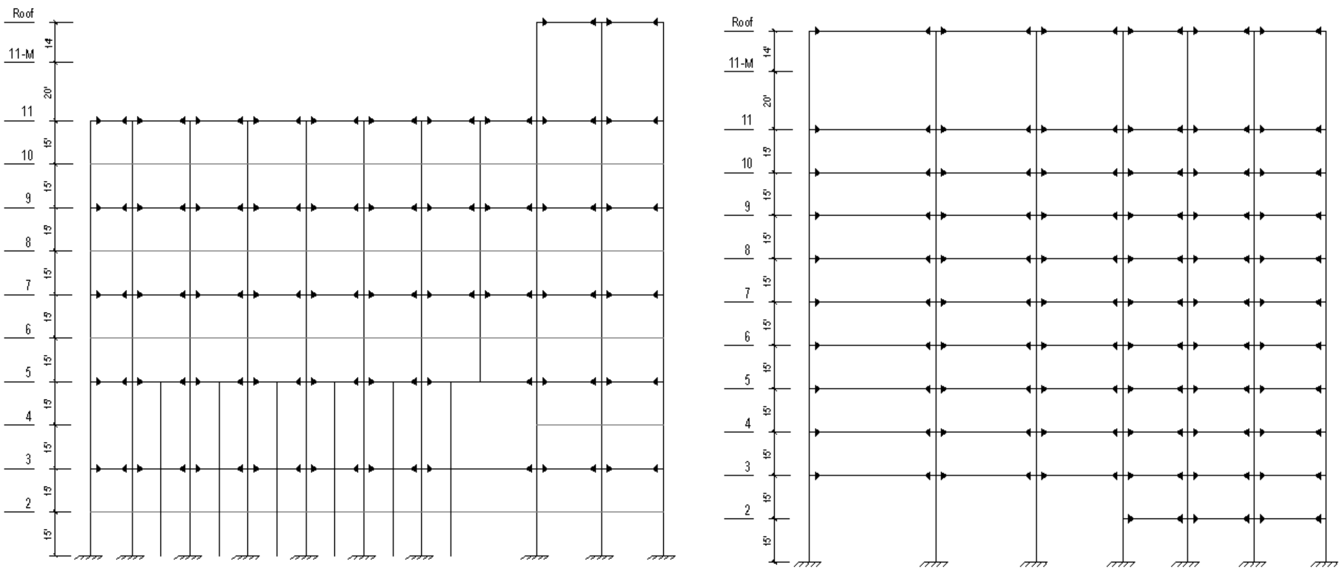



*Figure 2.4b: Moment Frames*

### 2.4 Roof System

The flat roof system is similar to a typical floor slab, consisting of 3" metal roof deck with 4 ¾" normal weight reinforced concrete topping and 6"x ¾" shear studs. Supporting this deck are wide flange steel beams ranging from W12x14 to W36x150 in size and spaced approximately 10'-6" on center. It is also important to note that a portion of the roof will be a green roof, requiring a significantly larger superimposed dead load to account for at this location.

## 2.5 Typical Floor Plans

### 2.5.1 Architectural

Below is the architectural floor plan for the first level of CSM. Colored zones indicate the functions of each area. The building footprint changes at Level 3, where it becomes more rectangular with the reduction in footprint at the southwest corner of the building.

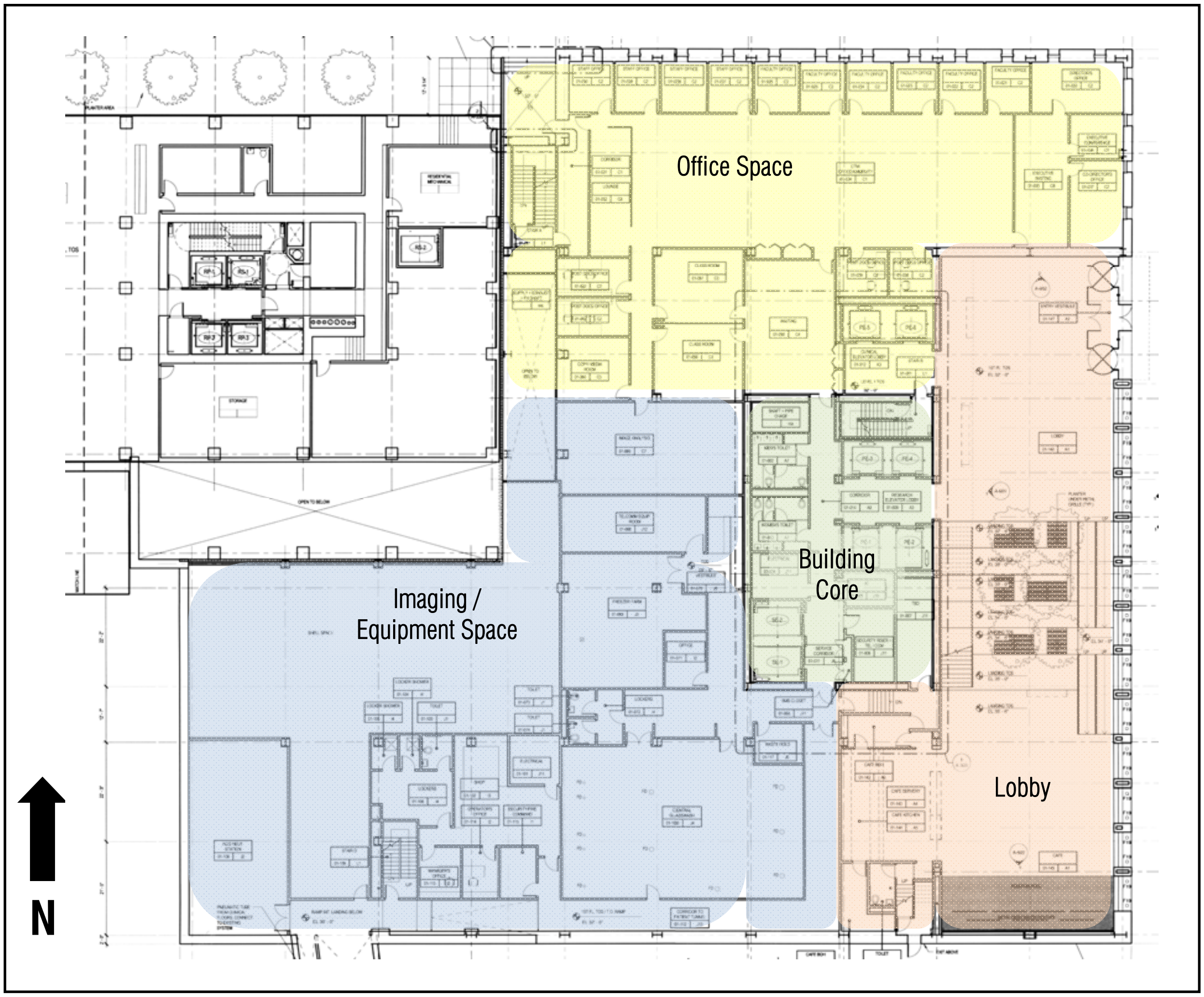



*Figure 2.5: Level 1, Architectural Plan*

### 2.5.2 Framing

Typical floor framing is shown in the figure below (laboratory floor). Composite metal deck spans the floor in the east-west direction in most areas and in the north-south direction above the atrium. Perimeter columns are spaced approximately 20'-0" to 22'-3" on center, and the longest span is 43'-8" (located on the north side of the building). A typical bay is noted with a dashed line and enlarged below. On this and other odd-numbered levels, moment connections exist at perimeter frames.

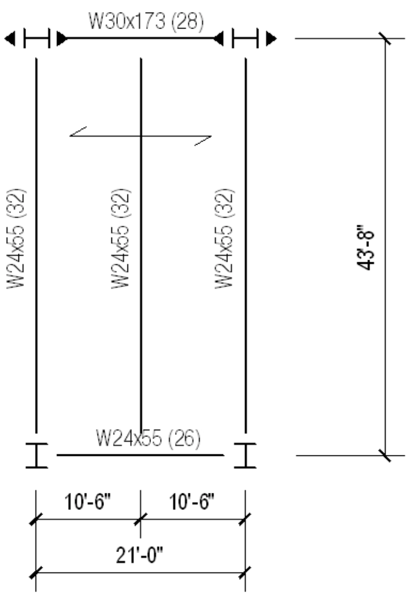



*Figure 2.6: Typical Bay*

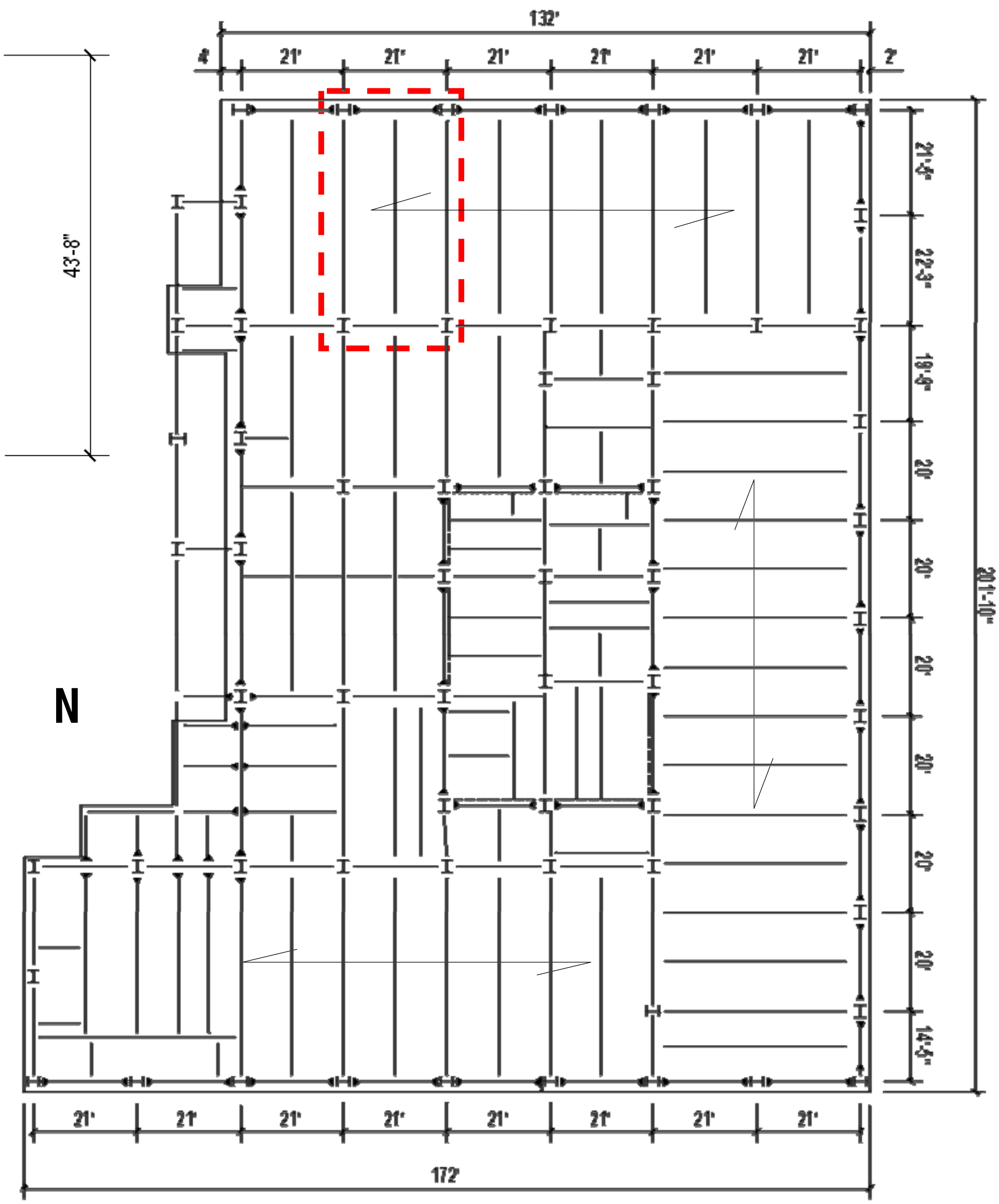



*Figure 2.7: Level 5, Floor Framing Plan*

# 3.0 DEPTH – Lateral System Redesign

## 3.1 Proposal

Problem Statement

In its current phase of design, the Center for Science and Medicine has been planned to utilize a combination of perimeter moment frames and core braced frames to resist lateral loads. Three of the four perimeter moment frames are two stories in height (Frames A, B, and C), due to restrictions imposed by the exterior cladding system, which makes them highly inefficient in terms of stiffness. Previous study indicates that each of these double-heighted frames resist only about 15%-20% of the lateral load in each direction, in some cases even less (Frame B, which resists 3%-8% at each level). One must question if it is even worthwhile to spend the time and money on constructing these frames, when they are not even playing a crucial role in the resistance of lateral loads. Moreover, braced frames at the core have been designed to utilize two heavy double-tee shapes for each brace. This may be difficult in terms of constructability. Braces also pose coordination issues at every level; openings are needed around the core, but awkward braces stand in the way. Thus, they must be shifted as needed, which slows the design process and increases the chance for coordination errors. Neither the braced frame system nor the moment frame system seems to be all-around ideal, and combining the two does nothing to improve their shortcomings. Therefore, an alternative system will be proposed in an attempt to eliminate the issues outlined above.

Proposed Solution

To improve the lateral system of the Center for Science and Medicine, a core of reinforced concrete shear walls will be proposed to replace the existing system of braced frames. Shear walls will stiffen the structure at the core in both directions, therefore eliminating the need of additional moment frames on the perimeter of the building. Also, a concrete core will alleviate coordination issues of proper brace placement, as openings can simply be punched where needed in each wall. Moreover, a concrete system has the potential to be more economic than a combined braced frame / moment frame system, as expensive moment connections will no longer be necessary in the proposed solution. The proposed shear walls will encompass the building core, which is 64'-10" long in the North-South direction and 42'-8" long in the East-West direction. These shear walls will be designed to resist 100% of the lateral load in both directions, therefore totally eliminating the need for perimeter moment frames.

Implications of Redesign

The building's effective seismic weight will likely increase due to the addition of heavy shear walls (although only by a small percentage), which will consequently increase the seismic loads to be resisted. Such a design would also change the response modification factor, R, used in seismic design calculations to a value of 5 (a value of 7 had been used in previous calculations, assuming a dual system), thus increasing the seismic loads to be resisted. Seismic loads will be re-evaluated and compared to wind loads to determine the governing case later in the body of this report. Also, the elimination of moment frames at the perimeter will allow for a redesign of these framing members as gravity-only. It is expected that girders will decrease in size and, as a result, reduce overall cost of steel, but they must also be checked for vibration where laboratory spaces are planned. Finally, spread footings below the core will likely require a re-design as a mat foundation in order to support the heavy distributed load from shear walls. These factors will be considered in addition to the redesign of the lateral system.

## 3.2 Codes and Design Requirements

### Applicable Codes

International Building Code 2006

New York City Building Code (referencing Uniform Building Code 1997)

AISC LRFD-2005, 13th Edition

ACI 318-05

ASCE 7-05

*The NYC Building Code was chosen as the design criteria for this report in order to maintain consistency with original design criteria.

### Deflection Criteria

**Floor Deflection**

| | |
|---|---|
| Typical live load deflection | L/360 |
| Typical total deflection | L/240 |

**Drift Limits**

| | |
|---|---|
| Allowable Building Drift | H/400 |
| Interstory Drift, Wind | h/400 to h/600 …… ASCE 7-05 (Section CC.1.2) |
| Interstory Drift, Seismic | 0.025h ……………. ASCE 7-05 (Table 12.12-1) |

### Load Combinations

The following load combinations should be considered when combining factored loads using strength design. In the case of gravity loads only, equation 2 usually governs. When both lateral and gravity loads are carried by a member, equations 4 or 5 may govern depending on the nature of the lateral load (wind vs. seismic).

**Basic Load Combinations (LRFD), UBC 1997** (referenced by New York City Building Code)

1.) $1.4D$

**2.) $1.2D + 1.6L + 0.5(L_r \text{ or } S)$**

3.) $1.2D + 1.6(Lr \text{ or } S) + (f_1L \text{ or } 0.8W)$

**4.) $1.2D + 1.3W + f_1L + 0.5(Lr \text{ or } S)$**

**5.) $1.2D + 1.0E + (f_1L + f_2S)$**

6.) $0.9D \pm (1.0\rho E_h \text{ or } 1.3W)$

## 3.3 Gravity Loads

Below is a table summarizing the gravity load values and vibration velocity limits as provided by the structural designer.

Table 3.1 Gravity Loads

| Floor / Description | | Design Dead Load | Design Live Load | Vibration Velocity |
|---|---|---|---|---|
| SC1 & SC 2 | | | | |
| · | Vivarium | 30 psf | 50 psf | 2000 $\mu$in/s |
| · | Stair | 5 psf | 100 psf | - |
| SC1 & SC2 Interstitial | | | | |
| · | Mechanical Service | 10 psf | 50 psf | - |
| · | Stair | 5 psf | 100 psf | - |
| Level 1 | | | | |
| · | Lobbies, Corridors | 110 psf | 100 psf | - |
| · | Office | 30 psf | 50 psf | 8000 $\mu$in/s |
| · | Glass Wash | 10 psf | 125 psf | 2000 $\mu$in/s |
| · | Stair | 5 psf | 100 psf | - |
| Level 2 | | | | |
| · | Wet Lab | 25 psf | 100 psf | 2000 $\mu$in/s |
| · | Loading Dock | 75 psf | 250 psf | - |
| · | Auditorium | 40 psf | 60 psf | - |
| · | Stair | 5 psf | 100 psf | - |
| Level 3 | | | | |
| · | Wet Lab | 25 psf | 100 psf | 2000 $\mu$in/s |
| · | Stair | 5 psf | 100 psf | - |
| Level 4 | | | | |
| · | Lobbies, Corridors | 110 psf | 100 psf | - |
| · | Office | 30 psf | 50 psf | 8000 $\mu$in/s |
| · | Stair | 5 psf | 100 psf | - |
| Levels 5 - 10 | | | | |
| · | Office | 30 psf | 50 psf | 8000 $\mu$in/s |
| · | Wet Lab | 25 psf | 100 psf | 2000 $\mu$in/s |
| · | Stair | 5 psf | 100 psf | - |
| Level 11 | | | | |
| · | Roof Terrace | 235 psf | 100 psf | - |
| · | Mechanical | 80 psf | 125 psf | - |
| · | Stair | 5 psf | 100 psf | - |
| Roof | | | | |
| · | Green Roof | 60 psf | 100 psf | - |
| · | Snow Load | - | 30 psf | - |
| Superimposed Loads | | | | |
| · | Partitions | 10-20 psf | - | - |
| · | CMEP | 10 psf | - | - |
| · | Finishes / Screed | 5-15 psf | - | - |
| · | Roofing Membrane / Insul. | 10 psf | - | - |

## 3.4 Lateral Loads

### 3.4.1 Seismic Loads

(Reference: NYC Building Code Article 5, RS 9-6)

Seismic loads were calculated in accordance with the New York City Building Code (2004). It was decided to use this code as the applicable standard in order to remain consistent with original design criteria. Since the Center for Science & Medicine has already been designed under the NYC Building Code, it would be inconsistent and inaccurate to compare a new system designed under a different building code with the existing system designed for New York City standards. Because the New York City Building Code is not a relatively well-known design standard, the seismic provisions are outlined in Appendix B. Below is a summary of the applicable design values and calculated seismic loads considered in the design of the Center for Science & Medicine.

Table 3.1 Seismic Design Values

| Seismic Design Values, NYC Building Code (references UBC 1997) | | |
|---|---|---|
| Occupancy | I | |
| Importance Factor | I = 1.25 | (Essential & Hazardous Facility) |
| Period, T | T = 1.89 sec | (from E-Tabs analysis) |
| S | 0.67 | (Rock, per Langan Report) |
| Z | 0.15 | (Zone 2A, per Langan Report) |
| $R_w$ | 8 | (Shear Walls) |
| Diaphragm | Rigid | |

The importance factor of I = 1.25 is applied because of the CSM's designation as an "essential and hazardous facility." The building period, T, was obtained from E-Tabs analysis. A site coefficient of S = 0.67 was obtained from the data within the geotechnical report, and it indicates the site's rocky soil profile. The seismic zone factor, Z, is designated as 0.15 for all buildings and structures in New York City (Zone 2A, according to UBC 1990). The value for $R_w$ is based on structural system type, and it is equal to 8 for a system of concrete shear walls.

Base shear was calculated using the following equations as defined in the New York City Building Code / UBC 1997:

- $V = \frac{Z\,I\,C\,W}{R_w}$ , where $C = \frac{1.25\,S}{T^{2/3}}$
- Story Force, $F_x = \frac{(V-F_t)w_x h_x}{\sum w_i h_i}$ , where $F_t = 0.07\,T\,V$
- Seismic Base Shear, $V = F_t + \sum F_i$

The following table summarizes the story force and total seismic base shear calculated by the methods outlined above.

Table 3.2 Calculation of Seismic Base Shear

| Level | Elevation | $w_i$ (given) | SW weight | $w_i$ (kips) | $w_ih_i$ (k-ft) | Fx (kips) |
|---|---|---|---|---|---|---|
| Roof | 184 | 4073 | 0 | 4073 | 749,432 | 109.8 |
| Level 11-M | 170 | 512 | 582 | 1094 | 185,959 | 27.2 |
| Level 11 | 150 | 6850 | 831 | 7681 | 1,152,188 | 168.8 |
| Level 10 | 135 | 3423 | 623 | 4046 | 546,269 | 80.0 |
| Level 9 | 120 | 4184 | 623 | 4807 | 576,893 | 84.5 |
| Level 8 | 105 | 3453 | 623 | 4076 | 428,026 | 62.7 |
| Level 7 | 90 | 4211 | 623 | 4834 | 435,099 | 63.8 |
| Level 6 | 75 | 3460 | 623 | 4083 | 306,258 | 44.9 |
| Level 5 | 60 | 4175 | 623 | 4798 | 287,906 | 42.2 |
| Level 4 | 45 | 3176 | 623 | 3799 | 170,975 | 25.1 |
| Level 3 | 30 | 4220 | 623 | 4843 | 145,303 | 21.3 |
| Level 2 | 15 | 4208 | 623 | 4831 | 72,472 | 10.6 |
| Level 1 | 0 | 5835 | 623 | 6458 | 0 | 0.0 |
| | | | | Seismic Base Shear (unfactored): | Σ = | 740.9 |

Calculated Base Shear, V = 740.9 kips

Factored Base Shear, (1.0) V = 740.9 kips

### 3.4.2 Wind Loads

(Reference: NYC Building Code Article 5, RS9-5)

Wind loads were calculated in accordance with the New York City Building Code (2004). As with seismic loads, it was decided to use this code as the applicable standard in order to remain consistent with original design criteria. Because the New York City Building Code is not a well-known code, the wind load provisions are outlined in Appendix B. Below is a summary of the design criteria and calculated wind loads considered in the design of the Center for Science & Medicine.

Table 3.3 Design Wind Pressures on Vertical Surfaces (NYC Building Code, Table RS9-5.1)

| Height Zone (ft) | Design Wind Pressure on Vertical Surface (psf) |
|---|---|
| 0-100 | 20 |
| 101-300 | 25 |
| 301-600 | 30 |
| 601-1000 | 35 |
| Over 1000 | 40 |

Interestingly, the New York City Building Code has a very simplified method of calculating wind pressures on vertical surfaces. As seen from the table above, the code requires the application of a constant pressure to surfaces increasing with height above ground. In the case of the 184-foot tall Center for Science & Medicine, the applied pressures are 20 psf and 25 psf (shown highlighted above). The application of these pressures in each direction generates the story forces, base shears, and overturning moments shown below.

Table 3.4 Design Wind Pressures, North-South (Y) Direction

| Story | Height (ft) | Height Above Grade (ft) | Tributary Height (ft) | Wind Pressure (psf) | N-S (Y) Width (ft) | Windward Y (kips) | Total Story Force (kips) | Overturning Moment Y (ft-k) |
|---|---|---|---|---|---|---|---|---|
| Roof | 0 | 184 | 7 | 25 | 172 | 30.1 | 30.1 | 5538.4 |
| 11-M | 14 | 164 | 17 | 25 | 172 | 73.1 | 73.1 | 11988.4 |
| 11 | 20 | 150 | 17.5 | 25 | 172 | 75.3 | 75.3 | 11287.5 |
| 10 | 15 | 135 | 15 | 25 | 172 | 64.5 | 64.5 | 8707.5 |
| 9 | 15 | 120 | 15 | 25 | 172 | 64.5 | 64.5 | 7740.0 |
| 8 | 15 | 105 | 15 | 25 | 172 | 64.5 | 64.5 | 6772.5 |
| 7 | 15 | 90 | 15 | 20 | 172 | 51.6 | 51.6 | 4644.0 |
| 6 | 15 | 75 | 15 | 20 | 172 | 51.6 | 51.6 | 3870.0 |
| 5 | 15 | 60 | 15 | 20 | 172 | 51.6 | 51.6 | 3096.0 |
| 4 | 15 | 45 | 15 | 20 | 172 | 51.6 | 51.6 | 2322.0 |
| 3 | 15 | 30 | 15 | 20 | 172 | 51.6 | 51.6 | 1548.0 |
| 2 | 15 | 15 | 15 | 20 | 172 | 51.6 | 51.6 | 774.0 |
| 1 | 15 | 0 | 7.5 | 20 | 172 | 25.8 | 25.8 | 0.0 |
| | | | | | | Base Shear Y: $\sum =$ 707.4 | | OTM Y: $\sum =$ 68,288.3 |

Table 3.5 Design Wind Pressures, East-West (X) Direction

| Story | Height (ft) | Height Above Grade (ft) | Tributary Height (ft) | Windward Pressure (psf) | E-W (X) Width (ft) | Windward X (kips) | Total Story Force (kips) | Overturning Moment X (ft-k) |
|---|---|---|---|---|---|---|---|---|
| Roof | 0 | 184 | 7 | 25 | 202 | 35.4 | 35.4 | 6,504.4 |
| 11-M | 14 | 164 | 17 | 25 | 202 | 85.9 | 85.9 | 14,079.4 |
| 11 | 20 | 150 | 17.5 | 25 | 202 | 88.4 | 88.4 | 13,256.3 |
| 10 | 15 | 135 | 15 | 25 | 202 | 75.8 | 75.8 | 10,226.3 |
| 9 | 15 | 120 | 15 | 25 | 202 | 75.8 | 75.8 | 9,090.0 |
| 8 | 15 | 105 | 15 | 25 | 202 | 75.8 | 75.8 | 7,953.8 |
| 7 | 15 | 90 | 15 | 20 | 202 | 60.6 | 60.6 | 5,454.0 |
| 6 | 15 | 75 | 15 | 20 | 202 | 60.6 | 60.6 | 4,545.0 |
| 5 | 15 | 60 | 15 | 20 | 202 | 60.6 | 60.6 | 3,636.0 |
| 4 | 15 | 45 | 15 | 20 | 202 | 60.6 | 60.6 | 2,727.0 |
| 3 | 15 | 30 | 15 | 20 | 202 | 60.6 | 60.6 | 1,818.0 |
| 2 | 15 | 15 | 15 | 20 | 202 | 60.6 | 60.6 | 909.0 |
| 1 | 15 | 0 | 7.5 | 20 | 202 | 30.3 | 30.3 | 0.0 |
| | | | | | | Base Shear X: Σ = 830.7 | | OTM X: Σ = 80,199.1 |

Calculated N-S Base Shear, V = 707.4 kips

**Factored N-S Base Shear, (1.3) V = 919.1 kips**

Calculated E-W Base Shear, V = 830.7 kips

**Factored E-W Base Shear, V = 1,080 kips**

## Conclusion

By inspection, the factored wind base shears in both directions are greater than the factored seismic base shear. Thus, wind controls in both directions, and the proposed lateral system will be designed to withstand these loads.

$$V_{seismic} = 740.9 \text{ k} < V_{wind,N\text{-}S} = 919.1 \text{ kips} < V_{wind,E\text{-}W} = 1{,}080 \text{ kips}$$

## 3.5 Preliminary Design

### 3.5.1 Design Criteria

Shear walls, boundary elements, and coupling beams were designed according to the provisions of ACI 318-05 Building Code Requirements for Structural Concrete. There are two chapters within this code that outline provisions for structural walls:

- Chapter 11, Section 11.10 (Shear and Torsion, Special Provisions for Walls)
- Chapter 21, Section 21.7 (Special Provisions for Seismic Design, Special Reinforced Concrete Structural Walls and Coupling Beams)

The proposed lateral system is designed for wind forces rather than seismic, which would lead one to choose Chapter 11 as the applicable design criteria. However, each shear wall is punched with openings and will thus act as two or more wall piers linked by coupling beams. Since coupling beam design is outlined only in Chapter 21, there was question as to which criteria should be used for design. After comparing both sets of provisions, it was decided to design all components of the lateral force resisting system in accordance with Chapter 21 (Special Provisions for Seismic Design). This set of criteria was selected for consistency, since coupling beams would be designed in Chapter 21 regardless, and because it would likely provide a more conservative design.

Summarized below are some of the basic design provisions for shear walls as defined in ACI 318-05, Chapter 21:

- (21.7.2.1) The distributed web reinforcement ratio shall not be less than 0.0025 in both transverse and longitudinal directions.
- (21.7.2.1) Maximum spacing of reinforcement is 18" each way.
- (21.7.4.1) The nominal shear strength (Vn) for structural walls shall not exceed:

$$V_n = A_{cv}\,(\alpha_c\sqrt{f'_c} + \rho_n f_y)$$

- (21.7.4.4) Nominal shear strength of all wall piers sharing a common lateral force shall not exceed:

$$8A_{cv}\sqrt{f'_c}$$

And nominal shear strength of any one pier shall not exceed:

$$10A_{cp}\sqrt{f'_c}$$

- (21.7.4.5) Nominal shear strength of horizontal wall segments and coupling beams shall not exceed:

$$10A_{cp}\sqrt{f'_c}$$

- (21.7.5.1) Structural walls shall be subject to combined flexure and axial loads. Effective flange widths, boundary elements, and effects of openings shall be considered.

### 3.5.2 Evolution of Design

To begin the lateral system redesign process, existing braced frames and moment frames were removed from the core and perimeter. Four shear walls were then placed around the core (two in each direction) to resist all lateral loads. A typical framing plan is shown below, locating the shear walls at the core.

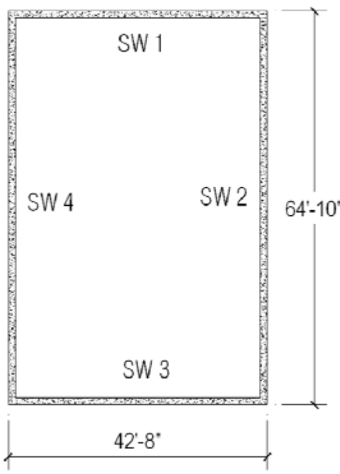



*Figure3.1: Shear Walls at Core, enlarged plan*

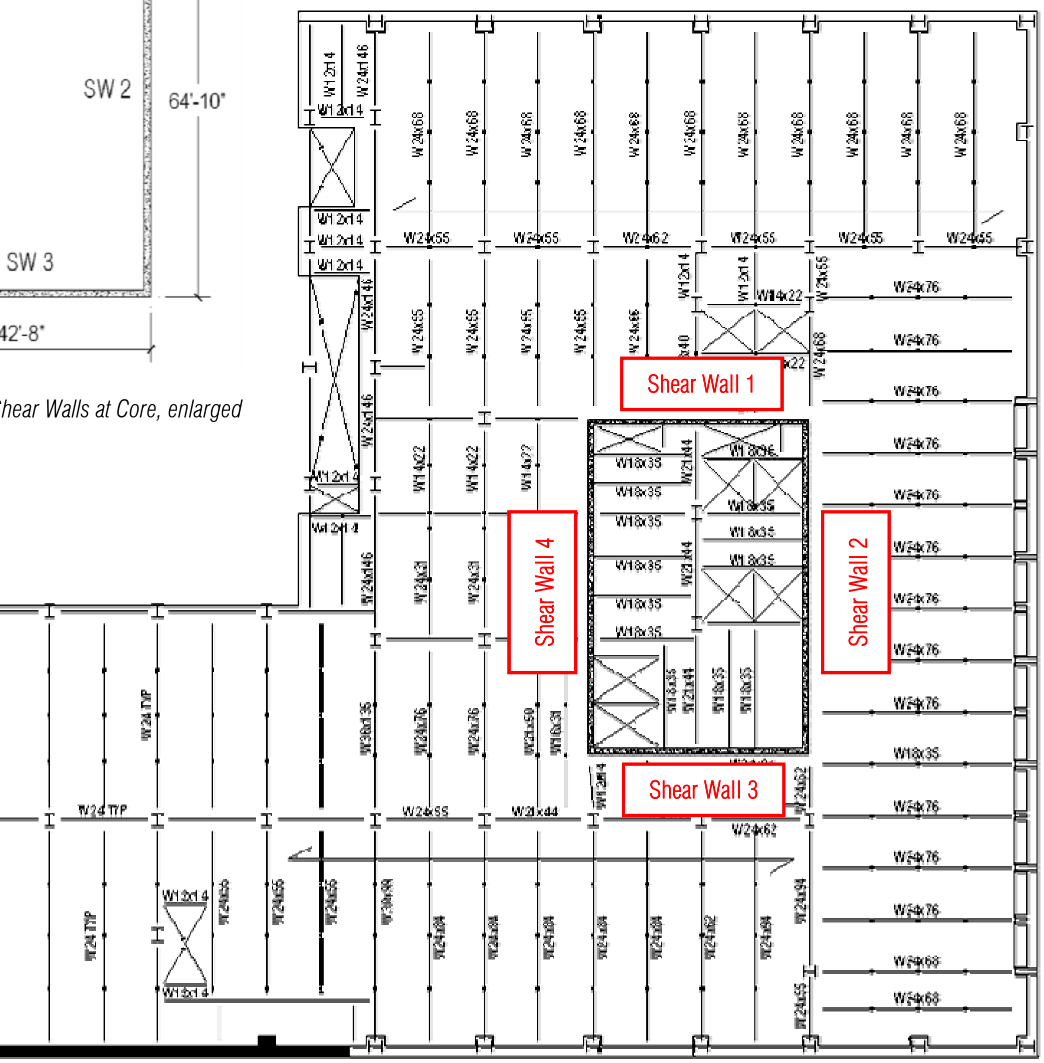



*Figure 3.2: First Floor Framing with Shear Walls*

Architectural floor plans were studied to determine exact placement of openings in each shear wall. The figures below illustrate the required configuration of openings in each shear wall and the proposed configuration of openings in each shear wall, respectively.

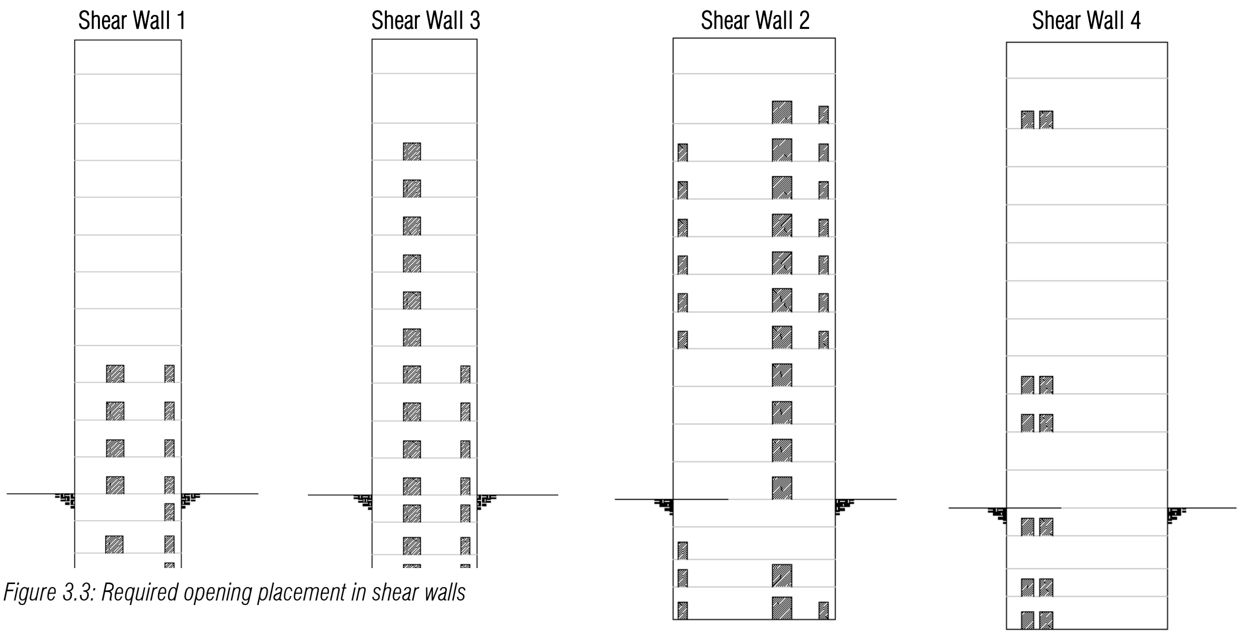



*Figure 3.3: Required opening placement in shear walls*

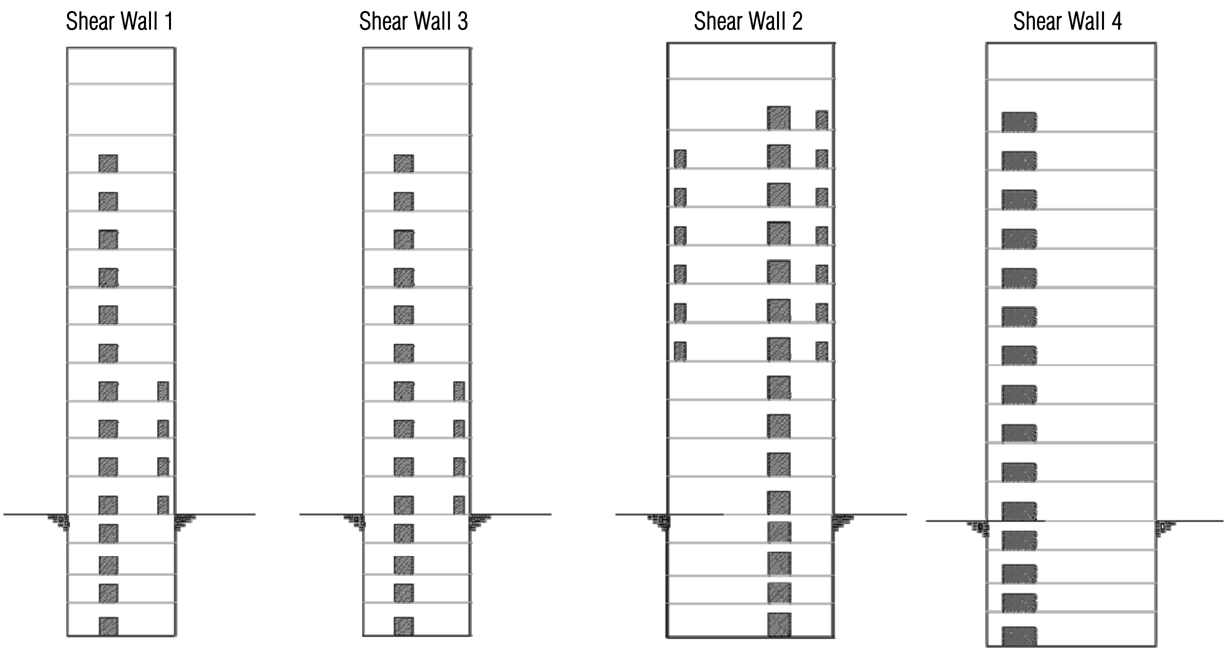



*Figure 3.4: Proposed opening placement in shear walls*

As seen in Figure 3.3, placing wall openings only where absolutely necessary would yield a very asymmetric configuration of wall piers. It is important that walls resisting load in the same direction are relatively similar in shape and size because this will allow for an even distribution of direct shear and thus minimal torsion. In order for Walls 1 & 3 to behave alike, additional openings were added to Shear Wall 1 to create a mirror image of Shear Wall 3. Similarly, in order for Walls 2 & 4 to behave alike, additional openings were added to Shear Wall 4. Not only does the proposed configuration of openings force similar behavior of shear walls, but it also simplifies their construction, allowing formwork to be used and re-used from floor to floor and from wall to wall.

It is important to note that the additional openings punched in Shear Walls 1 & 4 do not have negative architectural effects. The original openings in Wall 4 are for the service elevator which runs through the core from top to bottom. Providing an opening for the elevator doors at every level will allow for more flexibility and better use of space; the elevator can now service every floor of the CSM if the owner so desires. The additional openings in Wall 1 provide additional means of circulation through the core. If the owner decides these openings are not needed, they can be covered with drywall.

After deciding upon the configuration of openings for each shear wall, an arbitrary trial thickness of 20" was assigned to each, and concrete compressive strength was chosen to be 4,000 psi to match that of concrete already incorporated elsewhere in the building's design. This preliminary selection was checked against ACI 318-05 provisions to resist 100% of the lateral wind loads in the E-W and N-S directions. Hand calculations for shear strength, reinforcement, and combined bending/axial are shown in Appendix F. It was found that 20" thick shear walls provided more than enough capacity.

Next, an ETABS model was created to simulate the behavior of the core-only shear wall system, and lateral loads were applied as determined by the New York City Building Code. With the trial 20" thickness, overall building deflection was less than 0.5." Since this deflection is very small compared to the acceptable limit (H/400 = 5.52"), the thickness of the walls was able to be reduced. Still considering shear strength, combined axial and flexural strength, and the need for adequate thickness for coupling beams (complicated reinforcement layout), a final thickness of 16" was chosen for the design. Preliminary design for coupling beams assumed a 3 foot depth and a width equal to that of the shear wall thickness.

## 3.6 Computer Analysis

### 3.6.1 The Modeling Process

After completing a preliminary design, the proposed lateral system was modeled for analysis in ETabs. This model had 2 purposes:

1.) To determine how lateral load is distributed to each wall, depending on relative stiffness and inherent torsion,

2.) And to check serviceability drift limits.

The following outline details the step-by-step modeling procedure and input used in the final design of the shear wall system. All assigned values and properties are listed, as well as any assumptions made during the modeling process.

Software: ETabs Nonlinear Version 9.2 (Extended 3D Analysis of Building Systems)

**Story Data**

12 stories above ground
4 stories below ground
15' floor to floor, typical

**Definition of Materials**

Name: STEEL
Isotropic
Mass: 7.324E-07
Weight: 2.830E-04 $k/in^3$
Modulus of Elasticity: 29,000 ksi
Poisson's Ratio: 0.3
Thermal Coefficient: 6.5E-06
Fy: 50 ksi
Fu: 65 ksi

Name: CONC
Isotropic
Mass: 2.25E-07
Weight: 8.68E-05 $k/in^3$
Modulus of Elasticity: 4,000 ksi
Poisson's Ratio: 0.2
Thermal Coefficient: 5.5E-06
f'c: 4 ksi
Bending Reinforcement, fy: 60 ksi
Shear Reinforcement, fy: 60 ksi

**Definition of Wall Section**

Name: SW16
Material: CONC
Membrane: 16"
Bending: 16"
Type: Membrane
Property Modifier: f22 = 0.5 to account for effects of cracked sections (ACI 318-05 allows the use of 50% of stiffness values based on gross section properties).

### Wall Geometry and Assignments

**Shear Wall L6.9 and Shear Wall L9**
Wall Section: SW16
Total height: 232'
Total length: 42'-8"
Area Object Mesh: 24" x 24"
Restraints at Base: Fixed

**Shear Wall LE and Shear Wall LH**
Wall Section: SW16
Total height: 232'
Total length: 64'-10"
Area Object Mesh: 24" x 24"
Restraints at base: Fixed

### Coupling Beam Geometry and Assignments

**All Coupling Beams**
Beam Section: CB
Material: CONC
Property Modifiers: f22 = 0.5
I = 1,000 at ends of member (to simulate rigidity where plastic hinges form)

Width: 16"
Depth: 36"

### Diaphragm Definitions and Assignments

- All area objects defined with property "none."
- Assign each area object the D1 diaphragm property.
- Diaphragms defined as rigid since it is a concrete slab on metal deck.
- Restraints at Level 1 & below ground: Area springs were applied at each diaphragm's center of mass. Springs were assigned in all 3 directions (X,Y,Z) and given values of 1E20 to simulate infinite stiffness (these levels are restrained by the earth).

### Mass Definitions and Assignments

Each diaphragm was assigned a mass at its center:

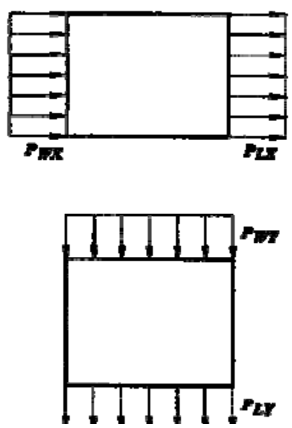
| Level | Area Type | Mass Per Area ($ksf/32.2/12^3$) |
|---|---|---|
| Roof | Null | $3.0 \times 10^{-6}$ |
| 11-M | Null | $2.0 \times 10^{-6}$ |
| 11 | Null | $2.0 \times 10^{-6}$ |
| 10 | Null | $1.0 \times 10^{-6}$ |
| 9 | Null | $1.0 \times 10^{-6}$ |
| 8 | Null | $1.0 \times 10^{-6}$ |
| 7 | Null | $1.0 \times 10^{-6}$ |
| 6 | Null | $1.0 \times 10^{-6}$ |
| 5 | Null | $1.0 \times 10^{-6}$ |
| 4 | Null | $1.0 \times 10^{-6}$ |
| 3 | Null | $1.0 \times 10^{-6}$ |
| 2 | Null | $2.0 \times 10^{-6}$ |
| 1 | Null | $2.0 \times 10^{-6}$ |
| SC1-M | Null | $2.0 \times 10^{-6}$ |
| SC1 | Null | $2.0 \times 10^{-6}$ |
| SC2-M | Null | $2.0 \times 10^{-6}$ |
| SC2 | Null | (base) |

(See Appendix B for effective weight calculations.)

Mass Source defined from self and specified mass. Include lateral mass only. Lump lateral mass at story levels.

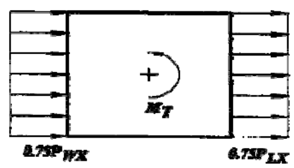
Static Load Cases, ASCE7-05
Wind Controls.

Case 1
From Figure 6-9,

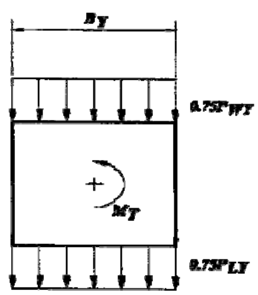



(See Appendix B for numerical values for each load case.)

Case 2
From Figure 6-9,

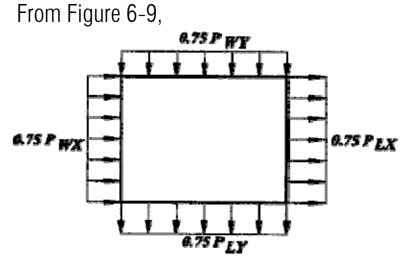



$M_T = 0.75\,(P_{WX}+P_{LX})B_X e_X$
$e_X = \pm\, 0.15\, B_X$

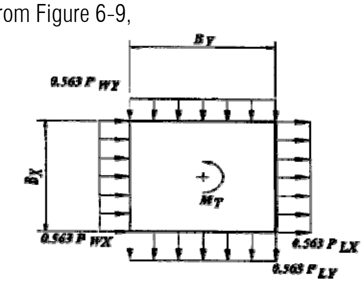



$M_T = 0.75\,(P_{WY}+P_{LY})B_Y e_Y$ $M_T$
$e_Y = \pm\, 0.15\, B_Y$

Case 3
From Figure 6-9,




Case 4
From Figure 6-9,




$= 0.563\,(P_{WX}+P_{LX})B_X e_X + 0.563\,(P_{WY}+P_{LY})B_Y e_Y$
$e_X = \pm\, 0.15\, B_X$ $e_Y = \pm\, 0.15\, B_Y$

*It is important to note that although the NYC Building Code (referencing the UBC) is the basis of design, the wind load cases above were taken from ASCE7-05. The UBC references ASCE7 for wind cases.

Static Load Combinations,
UBC 1997

Combination 1: 1.4D

Combination 2: 1.2D ± 0.8W

Combination 3: 1.2D ± 1.3W + L

Combination 4: 0.9D ± 1.3W

After inputting the load cases defined above into each of the 4 combinations to the left, 72 load combinations resulted and were checked by ETabs. These combinations are listed in Appendix B.

**Analysis Options**

All DOFs selected
Dynamic Analysis
Include P-Delta effects (non-iterative based on mass)

**Output: Calculated Building Periods**

$T_1$ = 1.881 sec
$T_2$ = 1.798 sec
$T_3$ = 1.255 sec
$T_4$ = 0.419 sec
$T_5$ = 0.361 sec
$T_6$ = 0.296 sec
$T_7$ = 0.239 sec
$T_8$ = 0.197 sec
$T_9$ = 0.175 sec
$T_{10}$ = 0.146 sec
$T_{11}$ = 0.139 sec
$T_{12}$ = 0.136 sec

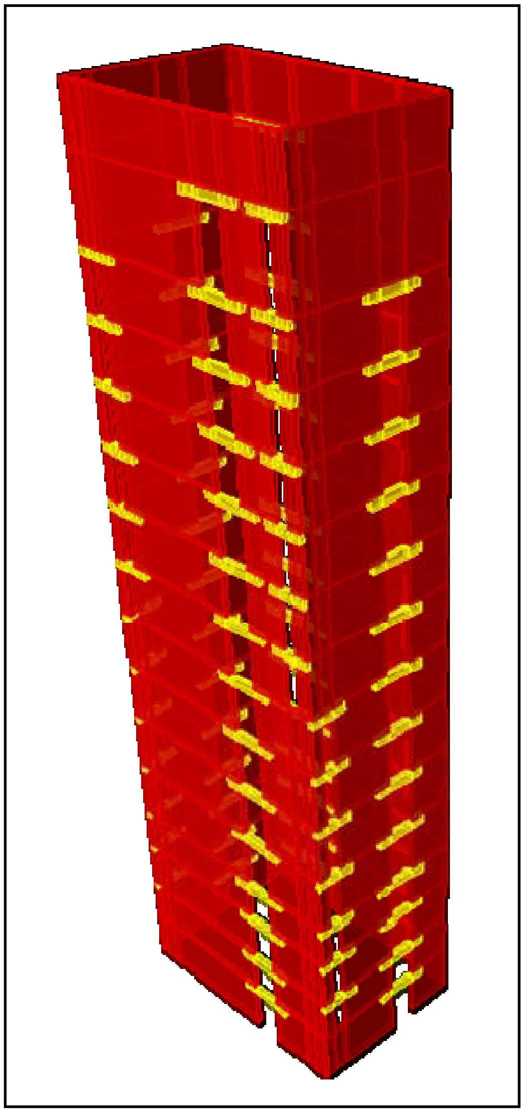


*Figure 3.5: Shear walls and coupling beams modeled in ETabs*

### 3.6.2 Distribution of Direct Shear

It is worthwhile to investigate the distribution of lateral load to each shear wall to determine relative stiffness and the effects of torsion. To determine the distribution of direct shear to each shear wall, a 1000 kip load was applied to the top of the building (at its center of pressure) in each direction. The model was run after deactivating the $R_z$ degree of freedom, which allows for neglect of torsion. Story shears were read from ETabs output, and the distribution of shear to each shear wall was found to be within a reasonable range (within 10% in both directions). Thus, the proposed configuration of openings is symmetric enough to allow a reasonable distribution of direct shear. In other words, each wall is has a similar stiffness to its counterpart. A summary of direct shear distribution (determined by the 1,000 kip load) is shown below.

Table 3.6 Direct Shear Distribution (due to 1,000 kip load)

| 1,000 kip load in East-West Direction (X) | | | | |
|---|---|---|---|---|
| | Shear Wall 1 | | Shear Wall 3 | |
| Level | Direct Shear (kips) | % of Total | Direct Shear (kips) | % of Total |
| Roof | 538 | 53.6 % | 465 | 46.4 % |
| 11-M | 598 | 59.6 % | 405 | 40.4 % |
| 11 | 524 | 52.0 % | 483 | 48.0 % |
| 10 | 541 | 53.7 % | 467 | 46.3 % |
| 9 | 558 | 55.3 % | 451 | 44.7 % |
| 8 | 561 | 55.5 % | 450 | 44.5 % |
| 7 | 562 | 55.6 % | 449 | 44.4 % |
| 6 | 583 | 56.4 % | 451 | 43.6 % |
| 5 | 557 | 55.0 % | 456 | 45.0 % |
| 4 | 553 | 54.5 % | 461 | 45.5 % |
| 3 | 546 | 53.9 % | 467 | 46.1 % |
| 2 | 515 | 50.8 % | 499 | 49.2 % |
| 1 | 531 | 53.1 % | 469 | 46.9 % |
| | Average: | 54.5 % | | 45.5 % |

| 1,000 kip load in North-South Direction (Y) | | | | |
|---|---|---|---|---|
| | Shear Wall 2 | | Shear Wall 4 | |
| Level | Direct Shear (kips) | % of Total | Direct Shear (kips) | % of Total |
| Roof | 454 | 45.4 % | 547 | 54.6 % |
| 11-M | 636 | 63.5 % | 366 | 36.5 % |
| 11 | 645 | 64.3 % | 358 | 35.7 % |
| 10 | 522 | 51.9 % | 483 | 48.1 % |
| 9 | 498 | 49.6 % | 507 | 50.4 % |
| 8 | 496 | 49.3 % | 510 | 50.7 % |
| 7 | 503 | 50.0 % | 504 | 50.0 % |
| 6 | 532 | 52.8 % | 475 | 47.2 % |
| 5 | 686 | 68.1 % | 322 | 31.9 % |
| 4 | 588 | 58.3 % | 420 | 41.7 % |
| 3 | 545 | 54.1 % | 463 | 45.9 % |
| 2 | 568 | 56.4 % | 439 | 43.6 % |
| 1 | 451 | 44.7 % | 557 | 55.3 % |
| | Average: | 54.5 % | | 45.5 % |

### 3.6.3 Actual Distribution of Shear (Including Torsion)

Under realistic conditions, inherent torsion must be accounted for in determining the distribution of shear to each lateral load resisting element. To do this, the same ETabs model was run with the $R_z$ degree of freedom activated. The distribution of load was found to be significantly different with torsion accounted for. A summary of actual shear distribution is shown below, and the average percentage of shear taken by each wall can be considered as its relative stiffness.

Table 3.7 Actual Shear Distribution (due to 1,000 kip load)

| 1,000 kip load in East-West Direction (X) | | | | |
|---|---|---|---|---|
| | Shear Wall 1 | | Shear Wall 3 | |
| Level | Shear (kips) | % of Total | Shear (kips) | % of Total |
| Roof | 658 | 65.6 % | 345 | 34.4 % |
| 11-M | 602 | 60.0 % | 402 | 40.0 % |
| 11 | 603 | 59.8 % | 405 | 40.2 % |
| 10 | 573 | 56.8 % | 436 | 43.2 % |
| 9 | 575 | 57.0 % | 434 | 43.0 % |
| 8 | 580 | 57.4 % | 431 | 42.6 % |
| 7 | 588 | 58.1 % | 424 | 41.9 % |
| 6 | 602 | 59.4 % | 411 | 40.6 % |
| 5 | 570 | 56.3 % | 443 | 43.7 % |
| 4 | 576 | 56.8 % | 438 | 43.2 % |
| 3 | 564 | 55.6 % | 450 | 44.4 % |
| 2 | 545 | 53.7 % | 469 | 46.3 % |
| 1 | 601 | 59.2 % | 414 | 40.8 % |
| | Average: | 58.1 % | | 41.9 % |

| 1,000 kip load in North-South Direction (Y) | | | | |
|---|---|---|---|---|
| | Shear Wall 2 | | Shear Wall 4 | |
| Level | Shear (kips) | % of Total | Shear (kips) | % of Total |
| Roof | 19.4 | 1.9 % | 983 | 98.1 % |
| 11-M | 485.9 | 48.5 % | 516 | 51.5 % |
| 11 | 309 | 30.8 % | 695 | 69.2 % |
| 10 | 244 | 24.3 % | 761 | 75.7 % |
| 9 | 246 | 24.5 % | 760 | 75.5 % |
| 8 | 255 | 25.3 % | 751 | 74.7 % |
| 7 | 261 | 25.9 % | 746 | 74.1 % |
| 6 | 263 | 26.1 % | 744 | 73.9 % |
| 5 | 266 | 26.4 % | 742 | 73.6 % |
| 4 | 243 | 24.1 % | 765 | 75.9 % |
| 3 | 232 | 23.0 % | 776 | 77.0 % |
| 2 | 292 | 25.7 % | 842 | 74.3 % |
| 1 | 167 | 17.7 % | 775 | 82.3 % |
| | Average: | 24.9 % | | 75.1 % |

It is clear that when torsion is neglected, lateral load is shared almost evenly between each wall in both directions. This indicates that despite the asymmetry between Shear Walls 2 & 4 (Shear Wall 2 has more openings but of smaller size), the walls have similar stiffness. However, when torsion is accounted for (as it would be in a realistic design approach), load sharing changes significantly. Loads are split 60/40 between Shear Walls 1 & 3, and loads are split 25/75 between Shear Walls 2 & 4. The imbalance of load sharing can be explained by considering the location of the applied load in relation to the shear walls. Shear Wall 4 is located exactly at the building's center of pressure (COP) in the E-W direction (see next page for illustration). Thus, it will see much more load than its counterpart, Shear Wall 2, which is 43 feet away from the COP. The distribution of load between Walls 1 & 3 is more equal, but the imbalance that does exist can again be explained by their positions relative to the COP. Shear Wall 1 is closer to the COP, and thus it carries more of the applied lateral load.

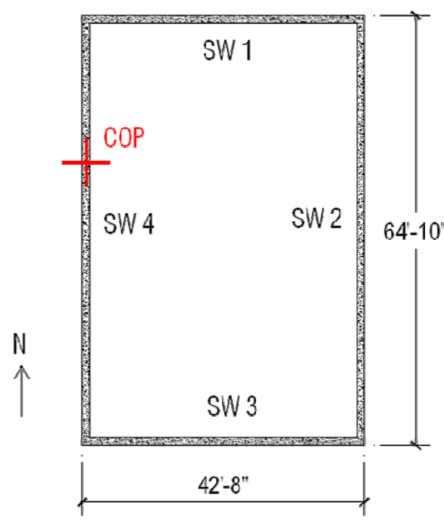



*Figure 3.6: Location of center of pressure, enlarged plan*

While it would be ideal for the walls to share the loads equally, such an imbalance of load sharing is not necessarily a bad thing. Simply put, each wall must be designed to carry the load it receives. So, Shear Walls 1 & 4, which take more load than their counterparts, will need to be designed for this extra load. This will be considered when final shear wall design is performed.

### 3.6.4 Building Deflection and Interstory Drift

**Total Building Drift**

Total building drift is taken as the maximum deflection at the top of the lateral force resisting system in each direction, as calculated by the ETabs analysis. These deflections are compared to an industry standard drift limitation of H/400 for both wind and seismic loads. Since drift is a serviceability check, no load factors need to be applied to lateral loads. Total deflections, for both wind and seismic load cases, are recorded in the table below. They are much less than the standard H/400 (where H = 184'or 2208") and are therefore acceptable. Very small drift was expected, as the core of shear walls provides a significant amount of stiffness.

Table 3.8 Total Building Drift, Wind and Seismic

| H/400 | $\Delta_{top}$ E-W | | $\Delta_{top}$ N-S | |
|---|---|---|---|---|
| | Seismic | Wind | Seismic | Wind |
| 5.52" | 1.45" | 2.37" | 0.876" | 1.26" |

**Interstory Drift**

Interstory drift was also calculated by ETabs analysis. Drift between stories was checked for both wind and seismic load cases, and they were then compared to ASCE 7-05 standards for wind interstory drift (h/400 to h/600) and seismic interstory drift (0.02h), where h is the story height. Total interstory drifts, recorded in the tables below, are significantly less than the allowable limits for both loading types.

Table 3.9 Interstory Drift, Wind

| Wind X-direction (E-W): Diaphragm Drift | | Limit, h/400 | Wind Y-direction (N-S): Diaphragm Drift | Limit, h/400 |
|---|---|---|---|---|
| Level 12 | 0.133 " | 0.42 " | 0.222 " | 0.42 " |
| Level 11-M | 0.180 " | 0.60 " | 0.038 " | 0.60 " |
| Level 11 | 0.159 " | 0.45 " | 0.086 " | 0.45 " |
| Level 10 | 0.177 " | 0.45 " | 0.094 " | 0.45 " |
| Level 9 | 0.191 " | 0.45 " | 0.100 " | 0.45 " |
| Level 8 | 0.200 " | 0.45 " | 0.104 " | 0.45 " |
| Level 7 | 0.204 " | 0.45 " | 0.106 " | 0.45 " |
| Level 6 | 0.200 " | 0.45 " | 0.105 " | 0.45 " |
| Level 5 | 0.208 " | 0.45 " | 0.098 " | 0.45 " |
| Level 4 | 0.219 " | 0.45 " | 0.092 " | 0.45 " |
| Level 3 | 0.204 " | 0.45 " | 0.053 " | 0.45 " |
| Level 2 | 0.152 " | 0.45 " | 0.084 " | 0.45 " |
| Level 1 (ground) | 0.049 " | 0.45 " | 0.024 " | 0.45 " |

Table 3.10: Interstory Drift, Seismic

| Seismic X-direction (E-W): Diaphragm Drift | | x 3.6 amp | Limit, $0.02h_{sx}$ | Seismic Y-direction (N-S): Diaphragm Drift | | x 3.6 amp | Limit $0.02h_{sx}$ |
|---|---|---|---|---|---|---|---|
| Level 12 | 0.080 " | 0.288 " | 3.36 " | Level 12 | 0.132 " | 0.475 " | 3.36 " |
| Level 11-M | 0.118 " | 0.425 " | 4.8 " | Level 11-M | 0.042 " | 0.151 " | 4.8 " |
| Level 11 | 0.104 " | 0.373 " | 3.6 " | Level 11 | 0.065 " | 0.234 " | 3.6 " |
| Level 10 | 0.116 " | 0.417 " | 3.6 " | Level 10 | 0.070 " | 0.253 " | 3.6 " |
| Level 9 | 0.124 " | 0.446 " | 3.6 " | Level 9 | 0.074 " | 0.267 " | 3.6 " |
| Level 8 | 0.128 " | 0.461 " | 3.6 " | Level 8 | 0.083 " | 0.299 " | 3.6 " |
| Level 7 | 0.129 " | 0.463 " | 3.6 " | Level 7 | 0.076 " | 0.274 " | 3.6 " |
| Level 6 | 0.124 " | 0.446 " | 3.6 " | Level 6 | 0.074 " | 0.267 " | 3.6 " |
| Level 5 | 0.130 " | 0.468 " | 3.6 " | Level 5 | 0.068 " | 0.245 " | 3.6 " |
| Level 4 | 0.127 " | 0.457 " | 3.6 " | Level 4 | 0.062 " | 0.224 " | 3.6 " |
| Level 3 | 0.113 " | 0.407 " | 3.6 " | Level 3 | 0.043 " | 0.153 " | 3.6 " |
| Level 2 | 0.083 " | 0.299 " | 3.6 " | Level 2 | 0.048 " | 0.174 " | 3.6 " |
| Level 1 (ground) | 0.028 " | 0.099 " | 3.6 " | Level 1 (ground) | 0.015 " | 0.054 " | 3.6 " |

### 3.7 Shear Wall Design

Shear walls were designed as coupled walls, linked by rigidly-connected beams at floor levels where openings occur. Not only do coupled walls allow for large openings within the core shear walls, but they also provide an efficient means of energy dissipation when subject to lateral loads. Coupling beams experience large, inelastic rotations at their ends as shear walls receive lateral load. As a result, plastic hinges occur at these locations rather than at the bases of the walls, thus maintaining the overall integrity of the structure. However, because coupling beams are subject to such conditions, their detailing and shear reinforcement must be designed sufficiently to prevent shear failure, ensure ductility, and allow for proper energy dissipation. This was considered in design and will be described in more detail in the following pages.

#### 3.7.1 Pier Design

Analysis by ETabs was used to find the maximum pier and beam forces generated by applied wind loads. A complete table of these values is available upon request. For the purposes of this report, a condensed table of maximum forces was assembled and used for pier design. Specifically, the maximum shear forces, axial loads (both tension and compression considered) and moments were read from ETabs output at three locations per wall:

- at the base of the walls (48 feet underground), where forces are the largest,
- at Ground Level, where base shear first reaches its maximum,
- and at Level 5 (halfway to the top of the building), where forces may be low enough to reduce the amount of reinforcement required for adequate strength.

The pier forces for Shear Wall 1 are shown in the table below. Negative axial loads indicate uplift, while negative moments and shears indicate direction (left vs. right). Pier forces for Walls 2, 3, and 4 can be found in Appendix F.

Table 3.11 Maximum Pier Forces for Shear Wall 1

SHEAR WALL 1 (16"): E-Tabs Output

| | Flexural Design | | | | | | Shear Design | |
|---|---|---|---|---|---|---|---|---|
| Level | Pier ID | Combo | Pu (k) | Ag ($ft^2$) | Pu + self + SDL + LL | M3u (ft-k) | Combo | Vu |
| Base | W1P1 | 416 | -237 | 16.888 | 1,871 | 1307 | 320 | 158 |
| Base | W1P2 | 424 | 623 | 30.666 | 4,450 | 2815 | 37 | 574 |
| | | | | | | | | |
| Level 1 | W1P1 | 424 | -10 | 16.88 | 1,877 | 432 | 38 | 254 |
| Level 1 | W1P2 | 424 | 412 | 21.999 | 2,871 | 1207 | 32 | 309 |
| Level 1 | W1P3 | 42 | -117 | 4 | 330 | 187 | 37 | 49 |
| | | | | | | | | |
| Level 5 | W1P1 | 424 | 175 | 16.888 | 1,423 | -133 | 320 | 127 |
| Level 5 | W1P2 | 424 | 459 | 30.666 | 2,725 | 454 | 37 | 387 |

Once maximum pier forces were determined, walls were designed for shear, flexure, and combined loading at each of the 3 levels under consideration. As stated in Section 3.5, the provisions of ACI 318-05 Chapter 21 were used to design and detail shear walls.

For shear, maximum forces given by ETabs were conservative to use as design values, since each wall was modeled as a membrane and thus took all shear applied in its direction (no out-of-plane shear). To check combined bending and axial load, PCA Column was used to analyze pier sections for an applied moment and axial load (maximum forces from ETabs) on a specified cross sectional area with a specified reinforcement ratio. Effective flange widths were accounted for in this check. Sample hand calculations for Wall1-Pier1 (W1P1) and Wall1-Pier2 (W1P2) are shown in Section 2.7.3. Calculations for all other wall piers are included in Appendix F or are available upon request.

Below is a wall pier schedule indicating web length (total pier length minus boundary element lengths) and the selected reinforcement. The majority of wall piers required minimum reinforcement in both flexure and shear ($\rho = 0.25\%$). No. 5 bars @ 12" on both faces and in both directions were chosen to achieve this required ratio.

Table 3.12 Wall Pier Schedule

| | Pier # | Web Length | Web Reinforcement | | Required $\rho$ | Provided $\rho$ |
|---|---|---|---|---|---|---|
| Base | W1P1 146" | 90" | Flexural | (2 curtains) #5 @12 = (16) #12 | 0.0025 | 0.0034 |
| | | | Shear | #5 @ 12 | 0.0025 | 0.0032 |
| | W1P2 186" | 130" | Flexural | (2 curtains) #5 @12 = (22) #12 | 0.0025 | 0.0033 |
| | | | Shear | #5 @ 12 | 0.0025 | 0.0032 |
| | W2P1 474" | 362" | Flexural | (2 curtains) #5 @12 = (64) #12 | 0.0025 | 0.0034 |
| | | | Shear | #5 @ 12 | 0.0025 | 0.0032 |
| | W2P2 208" | 136" | Flexural | (2 curtains) #5 @12 = (24) #12 | 0.0025 | 0.0034 |
| | | | Shear | #5 @ 12 | 0.0025 | 0.0032 |
| | W3P1 146" | 90" | Flexural | (2 curtains) #5 @12 = (16) #12 | 0.0025 | 0.0034 |
| | | | Shear | #5 @ 12 | 0.0025 | 0.0032 |
| | W3P2 186" | 130" | Flexural | (2 curtains) #5 @12 = (22) #12 | 0.0025 | 0.0033 |
| | | | Shear | #5 @ 12 | 0.0025 | 0.0032 |
| | W4P2 562" | 450" | Flexural | (2 curtains) #5 @12 = (76) #12 | 0.0031 | 0.0033 |
| | | | Shear | #5 @ 12 | 0.0025 | 0.0032 |

### 3.7.2 Boundary Element Design

There are two approaches for determining the need for boundary elements in shear walls according to ACI 318-05 Chapter 21. The method chosen for this design is based on compressive stress limits. Specifically, this method requires that

- Boundary elements will be designed for compression zones when the maximum extreme fiber stress exceeds 0.2f'c
- Boundary elements can be discontinued where compressive stress is less than 0.15f'c

Where boundary elements are required, they must extend horizontally into the "web" of the wall the larger distance of:

- c – 0.1lw, OR c/2

Vertically, boundary elements must extend from the critical section a distance greater than or equal to the larger of:

- lw, OR Mu / 4Vu, to ensure that elements extend beyond the zone over which concrete spalling would occur.

Boundary element requirements were checked at every base-level pier in each wall. In other words, each corner of the core was checked for compressive stress as well as each end of individual piers adjacent to openings in the walls. Sample calculations for the boundary elements required in Wall 1-Pier 1 (W1P1) and Wall 1-Pier 2 (W1P2) are shown in Section 3.7.3. Calculations for other wall piers are included in Appendix F or are available upon request. The element's length, longitudinal reinforcement, and transverse reinforcement were designed by hand. The section was then checked in PCA Column for combined bending/axial strength, using the controlling load combination (as determined by ETabs) and user-defined reinforcement.

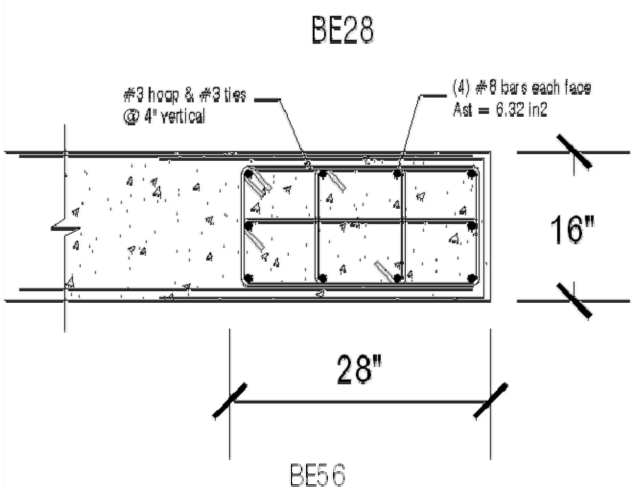
To the right is a schedule of final boundary element design and detailing. All boundary elements utilize #8 bars for longitudinal reinforcement ($\rho \geq 1\%$) and #3 bars for transverse reinforcement (As $\geq$ 0.44 in$^2$ per 4"). There are 7 typical sections: 4 L-shaped sections at wall corners and 3 rectangular sections adjacent to wall openings, shown on the following page.

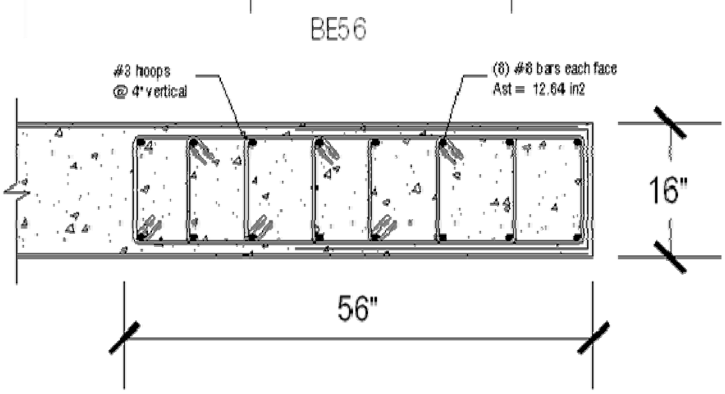
BOUNDARY ELEMENT REINFORCEMENT

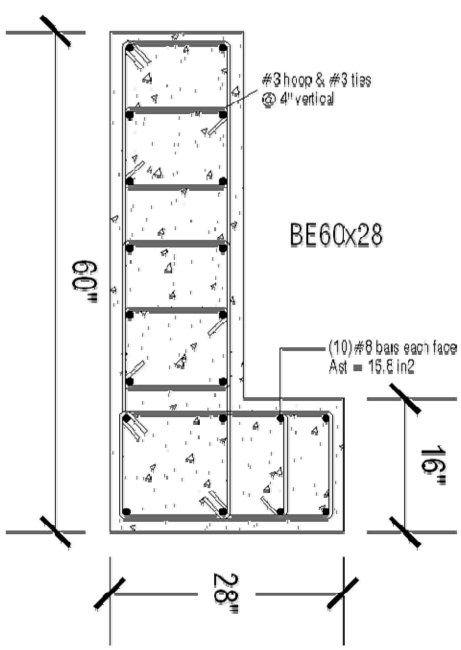
Table 3.13 – Boundary Element Schedule

| BE # | BE length | BE reinforcement | | Steel Required | Steel Provided |
|---|---|---|---|---|---|
| BE28 | 28" | Flexural | (2 curtains) # 8 bars = (8) #8 bars | $\rho = 0.01$ | $\rho = 0.033$ |
| | | Shear | #3 hoops and ties @ 4" vertical | $A_{st} = 0.3$ in$^2$ | $A_{st} = 0.44$ in$^2$ |
| BE 36 | 36" | Flexural | (2 curtains) # 8 bars = (10) #8 bars | $\rho = 0.01$ | $\rho = 0.014$ |
| | | Shear | #3 hoops and ties @ 4" vertical | $A_{st} = 0.3$ in$^2$ | $A_{st} = 0.44$ in$^2$ |
| BE56 | 56" | Flexural | (2 curtains) # 8 bars = (16) #8 bars | $\rho = 0.01$ | $\rho = 0.014$ |
| | | Shear | #3 hoops and ties @ 4" vertical | $A_{st} = 0.3$ in$^2$ | $A_{st} = 0.44$ in$^2$ |
| BE28x56 | 28"x56" (corner) | Flexural | (2 curtains) # 8 bars = (24) #8 bars | $\rho = 0.01$ | $\rho = 0.017$ |
| | | Shear | #3 hoops and ties @ 4" vertical | $A_{st} = 0.3$ in$^2$ | $A_{st} = 0.44$ in$^2$ |
| BE36x36 | 36"x36" (corner) | Flexural | (2 curtains) # 8 bars = (21) #8 bars | $\rho = 0.01$ | $\rho = 0.023$ |
| | | Shear | #3 hoops and ties @ 4" vertical | $A_{st} = 0.3$ in$^2$ | $A_{st} = 0.44$ in$^2$ |
| BE36x56 | 36"x56" (corner) | Flexural | (2 curtains) # 8 bars = (28) #8 bars | $\rho = 0.01$ | $\rho = 0.018$ |
| | | Shear | #3 hoops and ties @ 4" vertical | $A_{st} = 0.3$ in$^2$ | $A_{st} = 0.44$ in$^2$ |
| BE60x28 | 60"x28" (corner) | Flexural | (2 curtains) # 8 bars = (20) #8 bars | $\rho = 0.01$ | $\rho = 0.014$ |
| | | Shear | #3 hoops and ties @ 4" vertical | $A_{st} = 0.3$ in$^2$ | $A_{st} = 0.44$ in$^2$ |

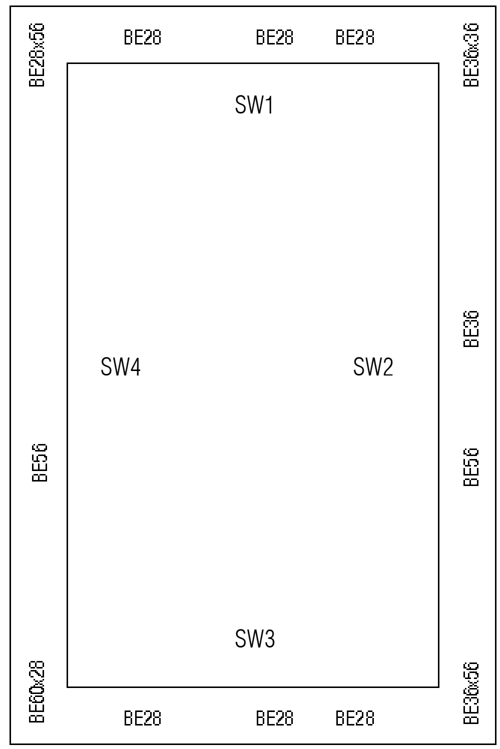
Boundary Element Detail (Sample Sizes)




Plan View of Base Level BE locations.



### 3.7.3 Sample Hand Calculations

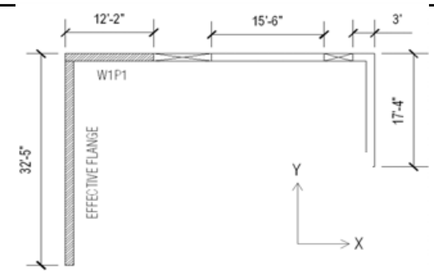
SHEAR WALL 1 / PIER 1 / BASE

| L (in) | Thickness (in) | Height (ft) | $A_g$ (in$^2$) | $A_n$ (in$^2$) | f'c | fy |
|---|---|---|---|---|---|---|
| 512 | 16 | 232 | 8,192 | 5,404 | 4,000 | 60,000 |

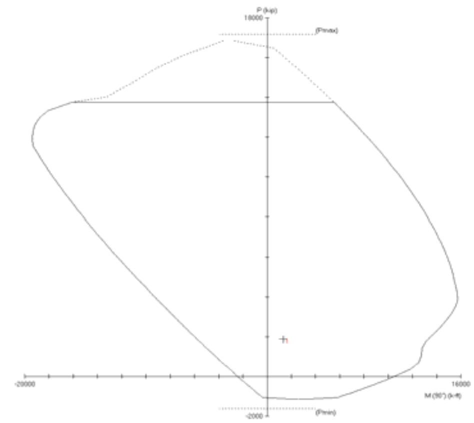
BASE, PIER 1: Length = 146", Web = 90" (length without BE's)

| Pier Label | $A_{cp}$ (in$^2$) | Vu (kips) | $Mu_y$ (ft-k) | Pu (kips) |
|---|---|---|---|---|
| W1P1 | 2,336 | 158 | 1,307 | 1,871 |

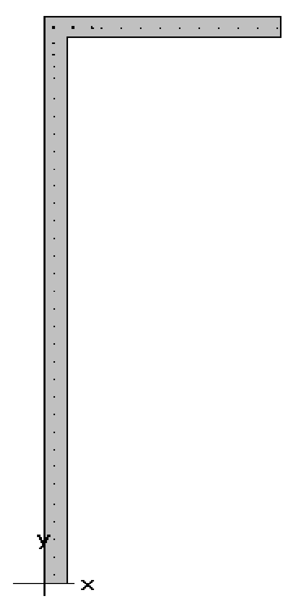
PLAN VIEW




INTERACTION DIAGRAM



CROSS SECTION OF DISTRIBUTED REINFORCEMENT




SHEAR WALL 1 / PIER 1 / BASE

**FLEXURAL DESIGN: ACI 318, Sec 21.7.5**

1.) Effective flange width (in) = 389

| Smaller of: | 1/2 distance OR to next opening: | OR 25% total wall height: |
|---|---|---|
| | 389 | 1,392 |

2.) Design axial load strength shall not exceed: 0.8ϕ[0.85f'c(Ag-As) + fyAs]

ϕ = 0.75

ϕPn, max (k) = 5,039 > 1,871 OK

3.) Check interaction of axial and bending with PCA Column: OK, within I-D.

| | | | |
|---|---|---|---|
| Max spacing each way = | 18 in | Selected Reinf: | (2 curtains) #5 @ 12" |
| Min. reinforcement ratio, $\rho$ = | 0.0025 | # bars provided: | 16 bars |
| Requ'd reinforcement ratio, $\rho$ = | 0.0025 | $\rho$ provided = | 0.0034 OK |

**SHEAR DESIGN: ACI 318, Sec 21.7.4**

1.) Check maximum shear strength of Pier 1 permitted: Vn = $10A_{cp}\sqrt{f'c}$

ϕVn, max (k) = 1,108 > 158 OK

| | | | |
|---|---|---|---|
| Max spacing each way = | 18 in | Selected Reinf: | (2 curtains) #5 @ 12" |
| Min. reinforcement ratio, $\rho$ = | 0.0025 | # bars provided: | 2 bars / ft |
| Requ'd reinforcement = | 0.48 in$^2$ / ft | area per foot= | 0.6200 OK |

**BOUNDARY ELEMENT DESIGN, ACI 318, Sec 21.7.6**

1.) BE required if fc > 0.2f'c

fc = Mc/I + P/A = 1,077 c = 73

0.2f'c = 800 I = 4,149,515

BE required.

2.) Determine BE length.

BE length = $c-0.1l_w$ = -7.02 c = 7.6

c/2 = 3.79

BE length ≥ 3.79

**BE length = 28"** Approximately 10% of pier length plus 1'.

3.) Longitudinal BE reinforcement

Determine average compressive and tensile stresses on pier. $A_{BE}$ = 448

Max Tension Combo: Pu = -237k, Mu = 1307 'k $A_{pier}$ = 2336

Max Compression Combo: Pu = 1547k, Mu = -1448 'k

\+ DL, Pu = 2108 + 1547 = 3655k

Apply $P_{uBE} = Pu/A*A_{BE} + Mu/I$

Tension: $P_{uBE}$ = 152.9

Compression: $P_{uBE}$ = 701.79 <-- Controls.

Apply ϕPn,max = $P_{uBE,max}$ = 0.8ϕ[0.85f'c(Ag - Ast) + fyAst] to find Ast.

701.8 = 0.8(0.75)[0.85*4000*(448-Ast) + fyAst]/1000

Ast = 4.65 in$^2$

Try (6) #8 bars, Ast = 4.74 in$^2$ > 4.65 OK

The spacing of 3 bars each face will exceed 8" --> Use (8) #8 bars

**Use (4) #8 bars each face, Ast = 6.32in2 > 4.65 OK**

4.) Transverse BE reinforcement: Sec 21.4.4

Assume #3 ties.

$h_x$ = 8.00 s ≤ 0.25(16") = 4.00

$h_c$ = 12.25 $6*d_b$ = 5.25

$4+(14-h_x)/3$ = 6.00

AND 4 ≤ s ≤6

$A_{sh} \geq 0.09*sh_cf'_c/f_{yh}$ Therefore, s = 4.00

**$A_{sh}$ ≥ 0.294** (vertical spacing of ties)

**$A_{sh,provided}$ = 4 x 0.11 = 0.44 OK**

**Use #3 hoops and #3 ties spaced at 4" vertical.**

## SHEAR WALL 1 / PIER 2 / BASE

| L (in) | Thickness (in) | Height (ft) | $A_g$ (in$^2$) | $A_n$ (in$^2$) | f'c | fy |
|---|---|---|---|---|---|---|
| 512 | 16 | 232 | 8,192 | 5,404 | 4,000 | 60,000 |

BASE, PIER 2: Length = 186", Web = 130" (length without BE's)

| Pier Label | $A_{cp}$ (in$^2$) | Vu (kips) | $Mu_y$ (ft-k) | Pu (kips) |
|---|---|---|---|---|
| W1P2 | 2,976 | 574 | 2,815 | 4,450 |

SHEAR WALL 1 / PIER 2 / BASE

**FLEXURAL DESIGN: ACI 318, Sec 21.7.5**

1.) Effective flange width (in) = N/A

| Smaller of: | 1/2 distance OR to next opening: | OR 25% total wall height: |
|---|---|---|
| | N/A | N/A |

2.) Design axial load strength shall not exceed: 0.8ϕ[0.85f'c(Ag-As) + fyAs]

ϕ = 0.75

ϕPn, max (k) = 6,402 > 4,450 OK

3.) Check interaction of axial and bending with PCA Column: OK, within I-D.

| | | | |
|---|---|---|---|
| Max spacing each way = | 18 in | Selected Reinf: | (2 curtains) #5 @ 12" |
| Min. reinforcement ratio, ρ = | 0.0025 | # bars provided: | 22 bars |
| Requ'd reinforcement ratio, ρ = | 0.0025 | ρ provided = | 0.0033 OK |

**SHEAR DESIGN: ACI 318, Sec 21.7.4**

1.) Check maximum shear strength of Pier 1 permitted: $V_n = 10A_{cp}\sqrt{f'c}$

ϕVn, max (k) = 1,412 > 574 OK

| | | | |
|---|---|---|---|
| Max spacing each way = | 18 in | Selected Reinf: | (2 curtains) #5 @ 12" |
| Min. reinforcement ratio, ρ = | 0.0025 | # bars provided: | 2 bars / ft |
| Requ'd reinforcement = | 0.48 in$^2$ / ft | area per foot= | 0.6200 OK |

SHEAR WALL 1 / PIER 2 / BASE

**BOUNDARY ELEMENT DESIGN, ACI 318, Sec 21.7.6**

1.) BE required if fc > 0.2f'c

fc = Mc/I + P/A = 1,861 — c = 93

0.2f'c = 800 — I = 8,579,808

BE required.

2.) Determine BE length.

BE length = $c - 0.1l_w$ = -4.95 — c = 9.7

c/2 = 4.83

BE length ≥ 4.83

BE length = 28" — Approximately 10% of pier length plus 1'.

3.) Longitudinal BE reinforcement

Determine average compressive and tensile stresses on pier. — $A_{BE}$ = 448

Max Tension Combo: Pu = 371k, Mu = -7305 'k — $A_{pier}$ = 2976

Max Compression Combo: Pu = 1850k, Mu = 6776'k

\+ DL, Pu = 1850 + 3827 = 5677k

Apply $P_{uBE} = Pu/A*A_{BE} + Mu/I$

Tension: $P_{uBE}$ = 527.1

Compression: $P_{uBE}$ = 857.64 <-- Controls.

Apply $\phi P_{n,max} = P_{uBE,max}$ = 0.8ϕ[0.85f'c(Ag - Ast) + fyAst] to find Ast.

857.6 = 0.8(0.75)[0.85*4000*(448-Ast) + fyAst]/1000

Ast = 1.7 in$^2$

Try (6) #8 bars, Ast = 4.74 in$^2$ > 1.7 OK

The spacing of 3 bars each face will exceed 8" --> Use (8) #8 bars

Use (4) #8 bars each face, Ast = 6.32in2 > 1.7 OK

4.) Transverse BE reinforcement: Sec 21.4.4

Assume #3 ties.

$h_x$ = 8.00 — s ≤ 0.25(16") = 4.00

$h_c$ = 12.25 — $6*d_b$ = 5.25

$4+(14-h_x)/3$ = 6.00

AND 4 ≤ s ≤6

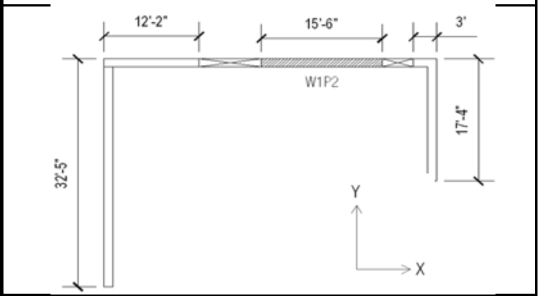
$A_{sh} \geq 0.09*sh_cf'_c/f_{yh}$ — Therefore, s = 4.00

$A_{sh}$ ≥ 0.294 — (vertical spacing of ties)

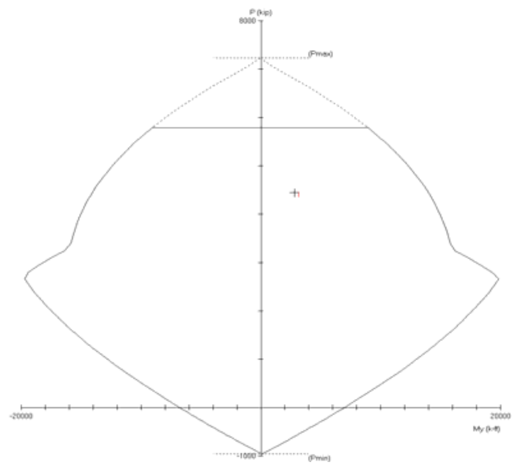
$A_{sh,provided}$ = 4 x 0.11 = 0.44 OK

Use (1) #3 hoop and (2) #3 ties spaced at 4" vertically to confine longitudinal reinf.

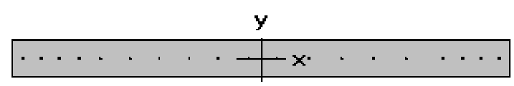
PLAN VIEW




INTERACTION DIAGRAM




CROSS SECTION OF DISTRIBUTED REINFORCEMENT



### 3.7.4 Coupling Beam Design

According to ACI 318-05 Chapter 21, the design of a coupling beam is largely dependent on its aspect ratio, $l_n/h$. Where $l_n/h \geq 4$, the coupled beam must be designed to satisfy the requirements specified for flexural members of a special moment frame. Where $l_n/h < 4$, the beam should be reinforced with two intersecting groups of diagonal bars centered about midspan. In the proposed core-only design for the CSM, there are three different coupling beam spans, all at 36" deep as an original assumption. The coupling beams are defined below, along with a summary of their final design. Condensed hand calculations are shown on the following page, and expanded calculations are included in Appendix G.

Table 3.14 Coupling Beam Properties

| | 4' span | 8' span | 13' span |
|---|---|---|---|
| f'c | 4,000 | 4,000 | 4,000 |
| Length (in) | 48 | 96 | 156 |
| Depth (in) | 36 | 36 | 36 |
| Width (in) | 16 | 16 | 16 |
| Acp ($in^2$) | 768 | 1536 | 2496 |
| Aspect Ratio, l/h | 1.33 | 2.67 | 4.33 |
| Reinforcement | Diagonals required. | Diagonals permitted. | Treat as flexural member of special moment frame. |
| $Vu_{max}$ | 30.2 k | 49.4 k | 41.5 k |

Table 3.15 Coupling Beam Schedule

| | 4' span | 8' span | 13' span |
|---|---|---|---|
| Transverse Reinforcement | #3 hoops @ 5" | #3 hoops @ 5" | (2) #3 legs @ 8" |
| Longitudinal Reinforcement | (2) #3 bars @ 6" | (2) #3 bars @ 6" | (5) #8 bars, top & bot |
| Diagonal Reinforcement | 2 diagonals of (4) #5 bars, $\alpha = 25°$ | 2 diagonals of (4) #7 bars, $\alpha = 17°$ | N/A |
| Diagonal Confinement | #4 hoops @ 6" | #4 hoops @ 6" | N/A |

## Condensed Hand Calculations

### 4' Span:

Check if Vu,max > or < 4√(f'c)Acp

Vu, max = 30.2

4√(f'c)Acp = 0

**Transverse Reinforcement: ACI 11.8.4**

$Av \geq 0.0025b_ws$

where s ≤ d/5 = 34.875/5 = 6.975"

Try s = 5" -->

Av ≥ 0.0025(16)(5) = 0.20 $in^2$ per 5"

Use #3 hoops @ 5" --> Av = 2(0.11) = 0.22 $in^2$

**Longitudinal Reinforcement: ACI 11.8.5**

$Avn \geq 0.0015b_ws_2$

where $s_2 \leq d/5$ = 34.875/5 = 6.975"

Try s = 6" -->

Avn ≥ 0.0015(16)(6) = 0.14 $in^2$

Use (2) #3 bars @ 6" --> Avn = 2(0.11) = 0.22 $in^2$

**Diagonal Reinforcement**

$Vn = 2A_{vd}f_y\sin\alpha \leq 10\sqrt{(f'c)}A_{cw}$

Using (4) #5 bars,

øVn = 0.75(2)(4 x 0.31)(60)sin(25°) = 47.2 k

47.2 k > 30.2 k --> OK

Use (4) #5 bars in each diagonal, at 25°

**To confine diagonals:**

Ash ≥ 0.09shcf'c/fy

Ash ≥ 0.09(6)(13)(4000/60,000)

Ash ≥ 0.39 $in^2$

Use #4 hoops @ 6" -->

Ash = 2(0.2) = 0.2 $in^2$ > 0.39 $in^2$ OK

### 8' Span:

Check if Vu,max > or < 4√(f'c)Acp

Vu, max = 49.4

4√(f'c)Acp = 0

**Transverse Reinforcement: ACI 11.8.4**

$Av \geq 0.0025b_ws$

where s ≤ d/5 = 34.875/5 = 6.975"

Try s = 5" -->

Av ≥ 0.0025(16)(5) = 0.20 $in^2$ per 5"

Use #3 hoops @ 5" --> Av = 2(0.11) = 0.22 $in^2$

**Longitudinal Reinforcement: ACI 11.8.5**

$Avn \geq 0.0015b_ws_2$

where $s_2 \leq d/5$ = 34.875/5 = 6.975"

Try s = 6" -->

Avn ≥ 0.0015(16)(6) = 0.14 $in^2$

Use (2) #3 bars @ 6" --> Avn = 2(0.11) = 0.22 $in^2$

**Diagonal Reinforcement**

$Vn = 2A_{vd}f_y\sin\alpha \leq 10\sqrt{(f'c)}A_{cw}$

Using (4) #7 bars,

øVn = 0.75(2)(4 x 0.6)(60)sin(17°) = 63.2 k

63.2 k > 49.4 k --> OK

Use (4) #7 bars in each diagonal, at 17°

**To confine diagonals:**

Ash ≥ 0.09shcf'c/fy

Ash ≥ 0.09(6)(13)(4000/60,000)

Ash ≥ 0.39 $in^2$

Use #4 hoops @ 6" -->

Ash = 2(0.2) = 0.2 $in^2$ > 0.39 $in^2$ OK

### 13' Span:

**Transverse Reinforcement: ACI 11.8.4**

Maximum spacing of hoops =

d/4 = 8.6"

8*(1") = 8" ← Controls.

24*(0.0375") = 9"

12 in

ØVs = ØAvfyd/s

41.5 k = $0.75A_v(60)(33)/8$

$A_v \leq 0.22$ $in^2$ per 8"

Use (2) #3 legs @ 8"

$A_{s,\,provided} = 0.44\ in^2 \geq 0.224\ in^2$ OK

**Longitudinal Reinforcement: ACI 21.3.3**

Minimum reinforcement not less than:

$3\sqrt{(f'c)}b_wd/f_y = 3\sqrt{(4000)}(16)(33)/60000 = 1.67\ in^2$

OR

$200b_wd/f_y = 200(16")(33")/60000 = 1.76\ in^2$

Maximum reinforcement ratio, $\rho_{max}$

$\rho_{max} = 0.85\beta f'c/f_y[\varepsilon_u/(\varepsilon_u + 0.005)]$

$\rho_{max} = 0.85(0.85)(4/60)[0.003/0.008] = 0.0181$

$ØM_n = 0.9(A_sf_y)(d-a/2)$, where $a = A_sf_y/(0.85f'_cb)$

With $A_s = 3.3\ in^2$, $ØM_n$ =463 ft-k

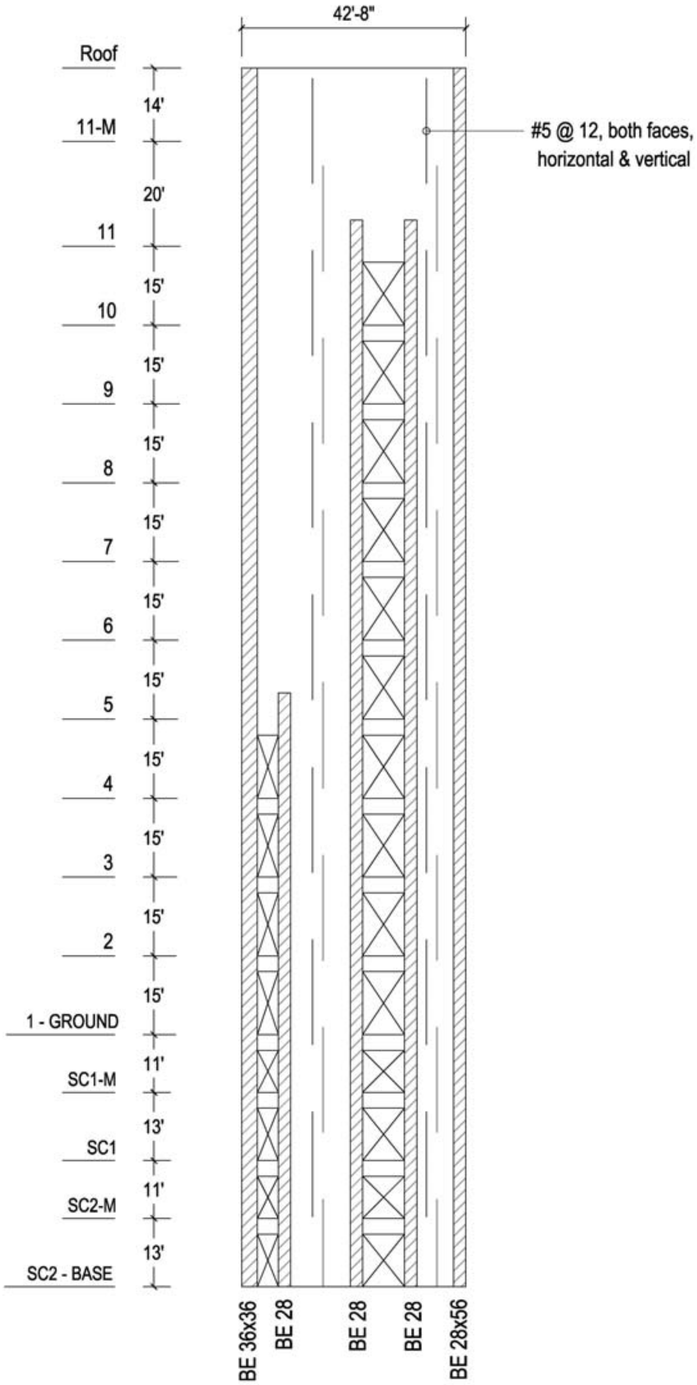
Use (5) #8 top and bottom

As = 3.95 in2 > As, req'd = 3.3 in2

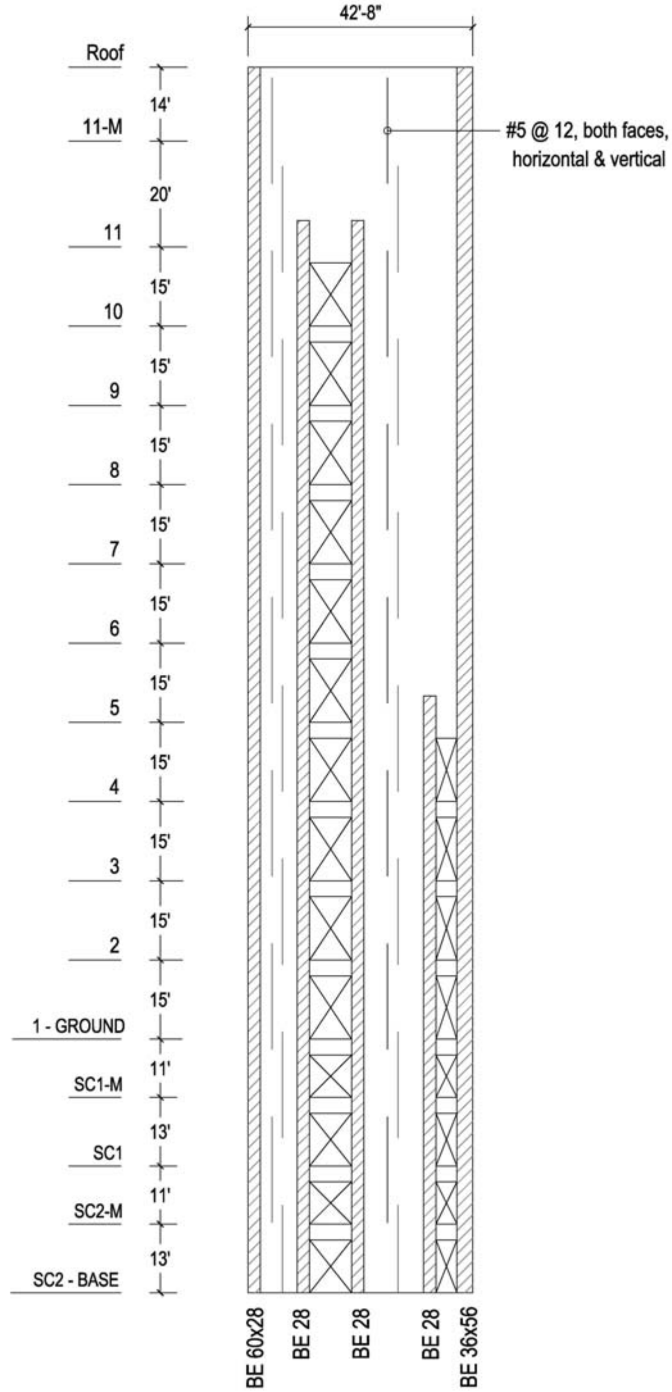
> As, min = 1.76 in2 OK

### 3.7.5 Summary: Final Shear Wall Design and Detailing

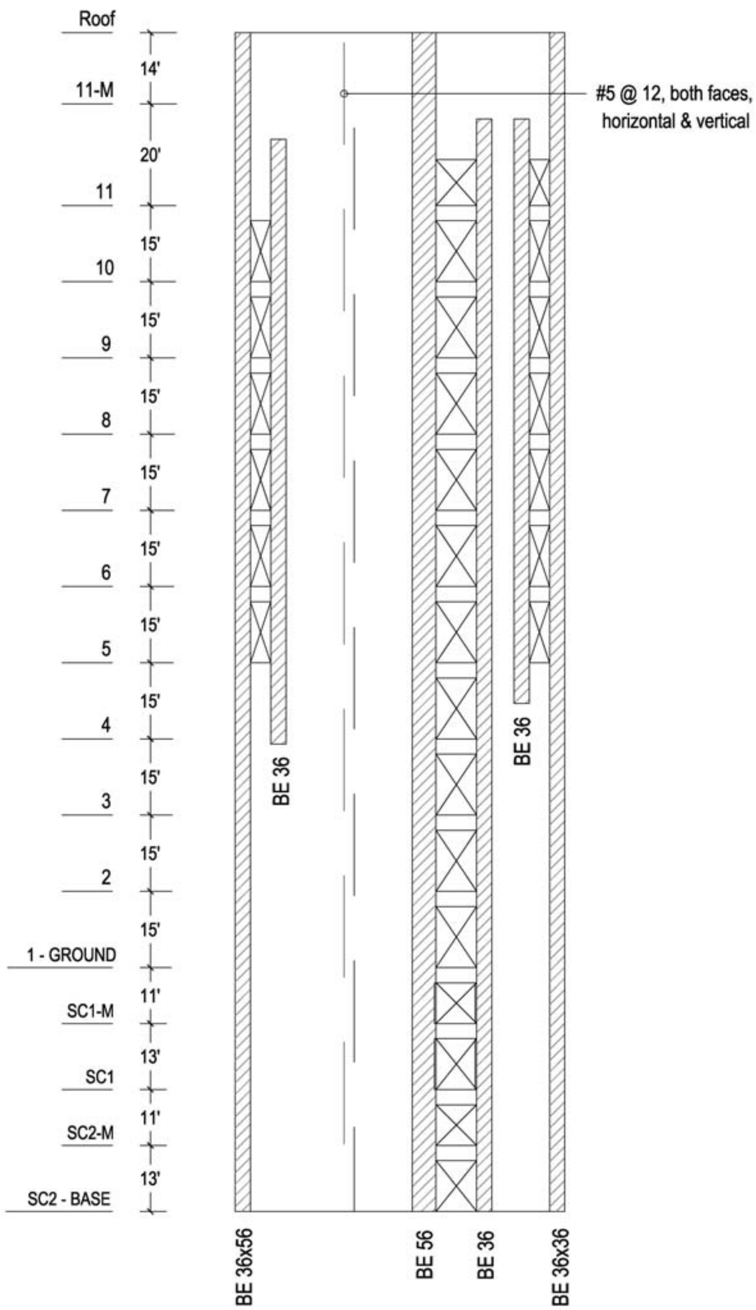
*SHEAR WALL 1*



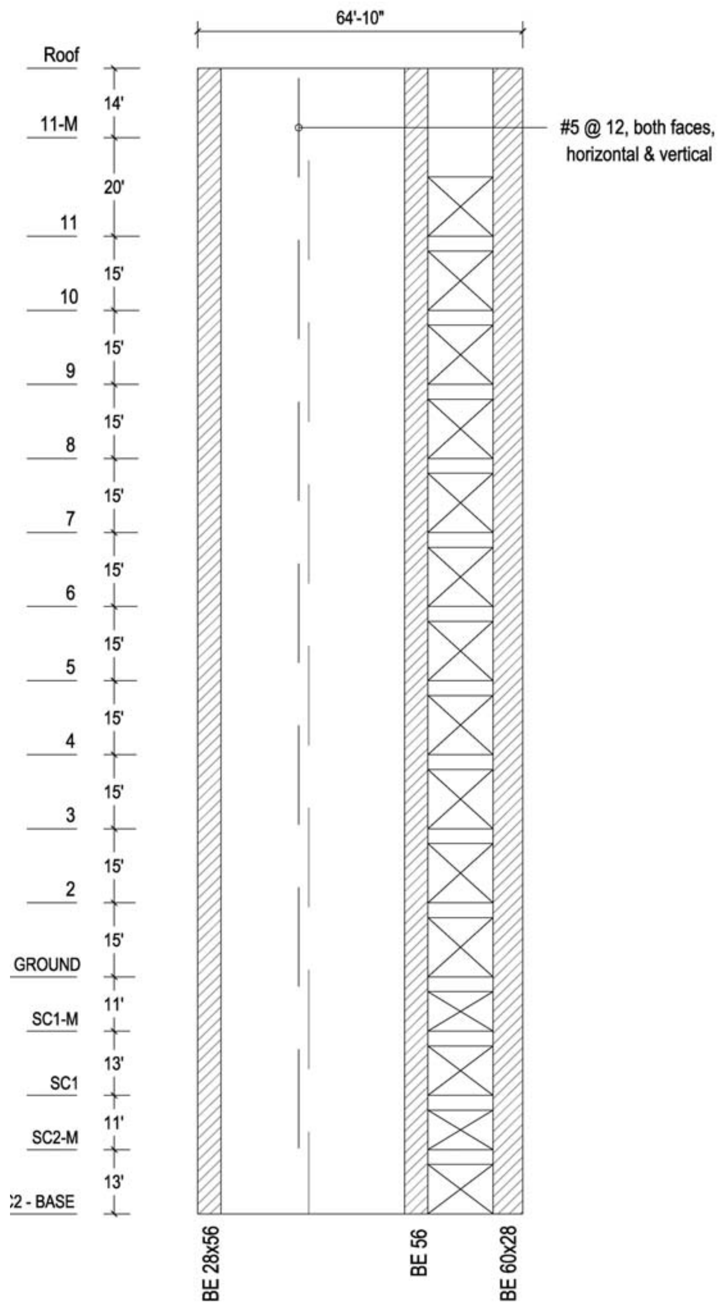

*SHEAR WALL 3*



*SHEAR WALL 2*




*SHEAR WALL 4*



### 3.8 Implications of Lateral System Redesign

#### 3.8.1 Gravity System Modifications

Removal of the four perimeter moment frames allows for a redesign of the structural members at these locations. Specifically, since these members at the perimeter are no longer resisting lateral load, they can be resized for gravity loads only. As a preliminary redesign, the girders were resized from W30 shapes to W24x55 shapes. This size was initially selected because it is the same shape as the girders on intermediate levels of the frame (only every other level of the perimeter frames were designed as lateral load-resisting). A typical moment frame is shown in comparison with a proposed gravity-only frame below.

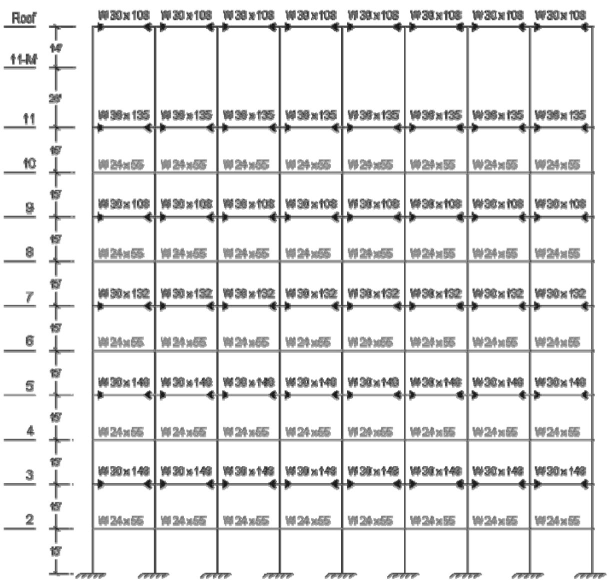



*Figure 3.11: Typical Moment Frame (existing design)*

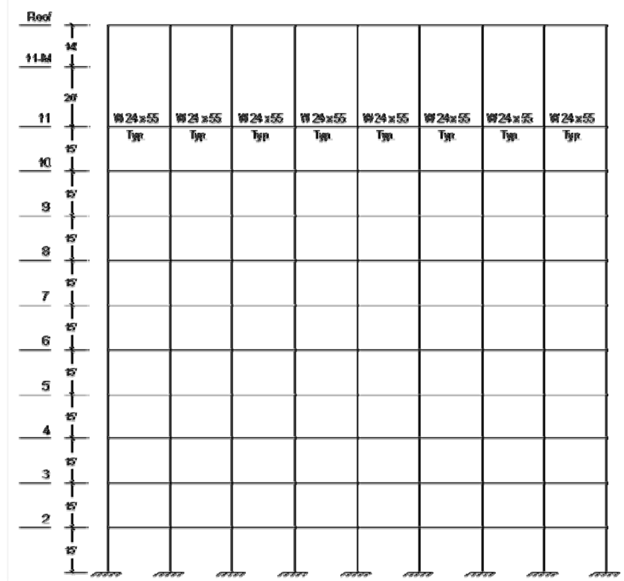



*Figure 3.12: Redesigned Gravity-Only Frame (proposed design)*

A typical level of the building was then modeled in RAM Structural System to check these perimeter members for strength and serviceability. Although other floors of the same loading conditions were designed with these exact member sizes, the strength and serviceability checks were performed for the sake of thoroughness. Results are summarized in the following pages and are shown in detail in Appendix H.

### Strength

From RAM output, the maximum shear and moment for a typical, redesigned exterior spandrel (W24x55) are shown below.

- Shear: Vu =22.5 k → ØVn = 0.75(251) = 188.3 k > 22.5 k OK
- Flexure: Mu = 231.8 ft-k → ØMn = 0.9(503) = 452.7 ft-k > 231.8 ft-k OK

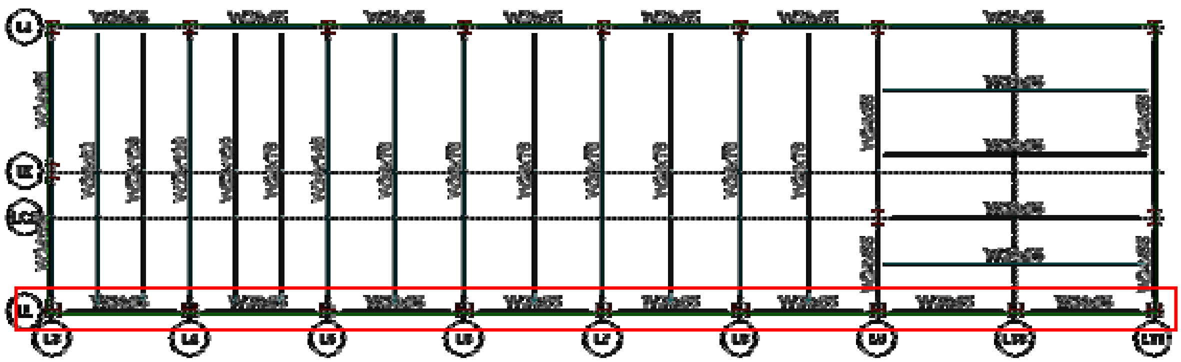
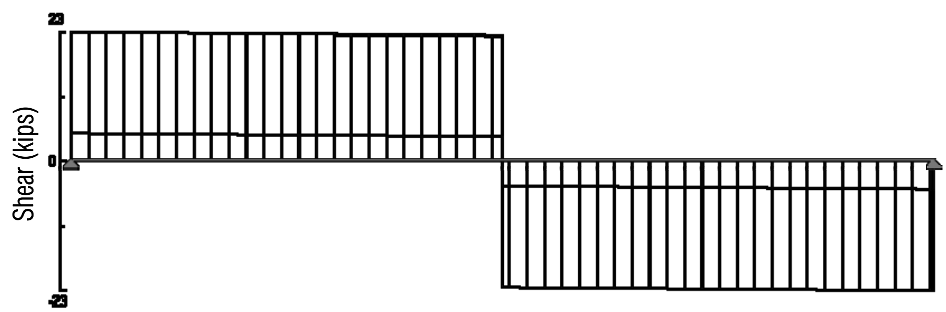
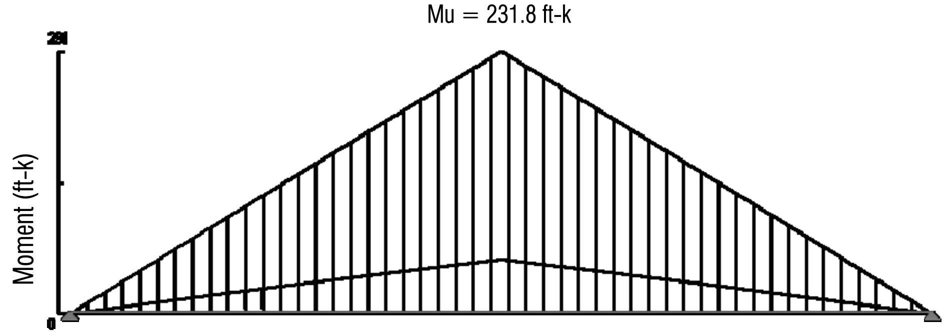


*Figure 3.13: RAM Model, floor plan of typical framing at redesigned girders (shown in red)*

Vu = 22.5 k




Mu = 231.8 ft-k




Therefore, W24x55 shapes are sufficient to resist gravity loads applied to these perimeter frames.

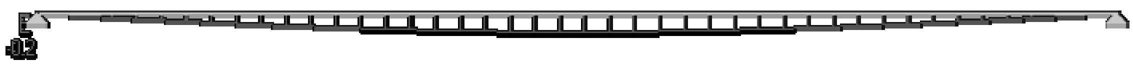
### Serviceability

It is clear from the previous strength check that gravity members for the CSM are over-sized in terms of capacity. However, it is important to note the stringent vibration criteria placed on the facility. Due to the sensitive equipment housed in many of the building's lab spaces, most laboratory areas are limited to a vibration velocity of 2,000 $\mu$in/s. Thus, members must be large enough to keep deflection to a minimum. The re-sized W24x55 girders (reduced from the original W30 shape) support wet laboratory spaces, so they must meet this criterion in addition to general deflection limits. Below, floor deflection will first be checked against general ASCE 7 code limitations and then against more stringent AISC Design Guide 11 vibration criteria for sensitive equipment.

### Allowable Total Load Deflection

ASCE 7-05 limits total load deflection to $\Delta \leq L/240$. Checking RAM's deflection output against this criterion gives:

$$\Delta max = 0.134" < (21' \times 12)/240 = 1.05" \rightarrow \quad OK$$

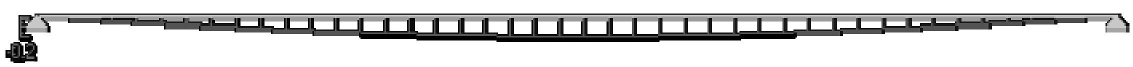


$\Delta = 0.134"$

### Allowable Live Load Deflection

ASCE 7-05 limits live load deflection to $\Delta \leq L/360$. Checking RAM's deflection output against this criterion gives:

$$\Delta max = 0.11" < (21' \times 12)/360 = 0.7" \rightarrow \quad OK$$



$\Delta = 0.11"$

### Vibration Criteria

AISC Design Guide 11 (Chapter 6) outlines specific criteria for evaluating a floor's vibration performance for sensitive equipment. Theoretically, the vibration performance of the W24x55 spandrels should be acceptable, since they are already used in the existing design (on every other level at the perimeter), assuming the members were designed appropriately. However, a check will be carried out to verify the adequacy of the member size. See Appendix H for complete calculations.

<u>Typical Wet Lab Floor Plan</u> (superimposed onto framing plan):


| <u>Beam Properties:</u> | <u>Girder Properties:</u> | <u>Slab:</u> |
|---|---|---|
| W24x76 | W24x55 | 4.75" topping on 3" metal deck |
| $A = 22.4 in^2$ | $A = 16.2 in^2$ | NWC, f'c = 4,000 psi |
| $I = 2100 in^4$ | $I = 1350 in^4$ | $E_c = 33(145)^{1.5}\sqrt{(4000)} = 3,644$ psi |
| d = 23.9 in | d = 23.6 in | n = 29000/(1.35*3644) = 5.9 |
| Span = 43'-8" | Span = 21'-0" | |
| Spacing = 10'-6" = 126" | | |

- From RAM analysis:
  $\Delta$total = 0.134"
- Natural frequency:
  $f_n = 0.18\sqrt{[g / (\Delta total)]} = 0.18\sqrt{[386.4 / (0.134l)]} = 9.64$ Hz
- Evaluation of predicted velocity:

  $V = U_v \Delta p / f_n$

  Since laboratory layout is not completely open (only one corridor along the edge), assume moderate walking speed.

For moderate walking,

$U_v = 5{,}500$

$V_p = 5{,}500(3.58 \times 10\text{-}6) / 9.64$

$V_p = 2{,}034\ \mu\text{in/sec} > 2{,}000\ \mu\text{in/sec limit}$ → No good?

- Conclusion:

  The vibration velocity limit for this laboratory is set at 2,000 $\mu$in/sec due to the high power microscopes and other sensitive equipment operated in the room. It appears that the predicted velocity is slightly high and that, consequently, the floor system does not satisfy the design limit. However, the above evaluation method does not take into account:

  - Interior partitions: There are two, full-height partitions in each laboratory (see plan). Partitions help in reducing floor vibration, which is not taken into account in the above calculation.

  - Location of walking path: The main corridor in the laboratory is along the edge of the room, away from lab benches and closer to columns supporting the bay. This configuration will also help in keeping under control any vibration from walking excitation, but was not able to be accounted for in calculation.

Using the justification above, the redesigned floor system with smaller W24x55 girders is acceptable for sensitive laboratory equipment.

### 3.8.2 Effects on Foundation

Replacing a core of steel framing with a core of concrete shear walls will certainly have an impact on foundations. The proposed system of shear walls is about 10 times heavier than the original steel braced frames. Also, rather than column point loads bearing down on foundations, long stretches of distributed loads will need to be transferred into the foundation. These differences will require a redesign of the foundation from existing spread footings to a mat foundation or strip footings with greater strength.

In addition to increased gravity loads, overturning moment must be accounted for as well. From the shear distribution found in Section 2.6.3, it is possible to determine the amount of wind load seen by each shear wall and thus the overturning moment created at the base of each.

TOTAL WIND, E-W: 830.7 kips (unfactored)

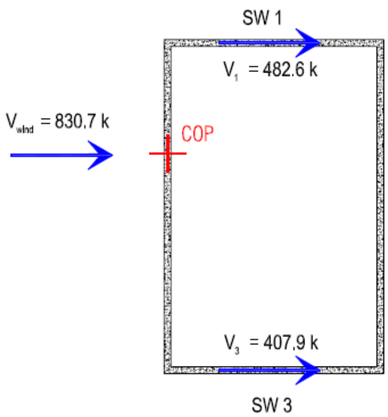
Shear Wall 1 receives 58.1% = 482.6 kips

Shear Wall 3 receives 41.9% = 407.9 kips

TOTAL WIND, N-S: 707.4 kips (unfactored)

Shear Wall 2 receives 24.9% = 176.1 kips

Shear Wall 4 receives 75.1% = 531.3 kips



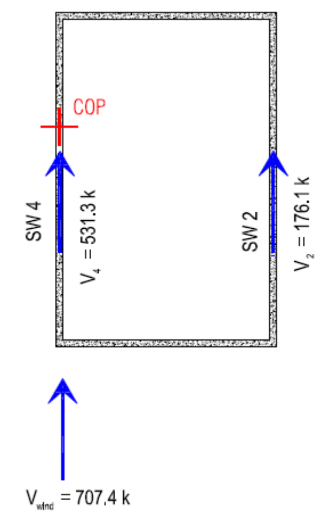




Table 3.16 Resisting Moment vs. Overturning Moment

| | Shear Wall 1 | Shear Wall 2 | Shear Wall 3 | Shear Wall 4 |
|---|---|---|---|---|
| *Height* | 232 ft | 232 ft | 232 ft | 232 ft |
| *Length* | 42'-8" | 64'-10" | 42'-8" | 64'-10" |
| *Applied Wind Load* | 482.6 k | 176.1 k | 407.9 k | 531.3 k |
| *Overturning Moment* | 111,963 ft-k | 40,855 ft-k | 94,633 ft-k | 123,192 ft-k |
| *Resisting Dead Load* | 4533 k | 8186 k | 3716 k | 8636 k |
| *Resisting Moment* | 96,712 ft-k | 264,779 ft-k | 79,275 ft-k | 184,241 ft-k |
| | $M_R < M_{OT}$ | $M_R > M_{OT}$ | $M_R < M_{OT}$ | $M_R > M_{OT}$ |

As shown in the table to the left, applied wind loads create an overturning moment at the base of each shear wall. Wall self-weight and any additional dead load they carry creates an opposing moment to resist this overturning. Shear Walls 2 & 4 carry enough dead load to resist uplift from wind loads. However, Shear Walls 1 & 3 are unable to resist the overturning moment on their own. This uplift would need to be considered when redesigning the foundation.

3.8.3 Construction Method

The location of the Center for Science & Medicine creates a problem when it comes to construction. Typically, a building with a concrete core and surrounding steel framing is built by placing the core first and allowing steel to follow in erection sequence. This gives concrete time to cure and develop its specified compressive strength before other members must frame into it. In New York City, however, the steel union does not allow any other trade to work above their employees on a construction site. So, because the CSM is located in New York City, this conflict arises; the building's steel frame must be erected before the proposed concrete shear walls. This makes construction of the walls very difficult, as typical methods of placing concrete walls can no longer be used (ie, flying concrete forms). Instead, slip forming or some other similar construction method must be employed, and temporary bracing must be installed to support the steel structure while walls rise to the proper height and cure to the specified strength.

There is a good solution to this problem, although potentially a costly one. PERI Automatic Climbing System (ACS) is an automatic self-climbing formwork system used in many steel-first construction jobs. This formwork is hydraulically operated and is raised without the use of a crane, as it is connected to the structure at all times and literally moves itself from pour to pour. This system is quite efficient, as there is no need for workers to ride the formwork as it is raised, thus eliminating the slow and unsafe crane time used in typical concrete placing methods. The PERI ACS has been used on more than 100 different projects around the world, including Seven World Trade Center, the Petronas Towers, and the Trump World Tower. In the case of Seven World Trade Center, ACS formwork was employed to allow for steel to rise before concrete. The strategy proved challenging but successful, as a constant four-day-per-floor cycle allowed the project to top out a month before scheduled completion.

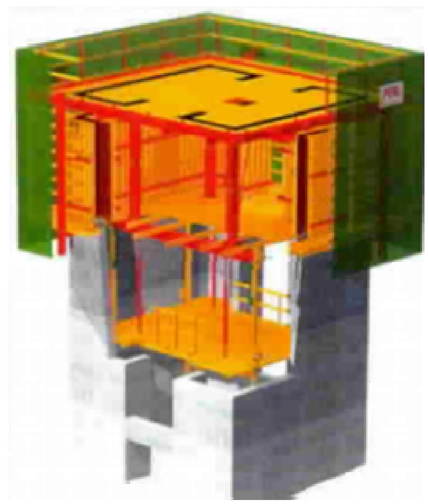


*Figure3.14: Self-climbing formwork*

The proposed lateral system for the Center for Science & Medicine would not pose a problem if the building's location were different. Because of its New York City address, however, ACS formwork would be required to construct the core (following steel erection in sequence). The implications of this construction method cannot be quantified. While it seems that self-climbing formwork is more efficient, faster, and safer, the cost is unknown (and pricing information could not be obtained). In speaking to a representative from PERI, the formwork vendor, it was determined that the added cost of specialty formwork would be offset by the labor cost saved if one were to use this system. However, this information could be biased, so a firm conclusion cannot be reached on this matter. Regardless, the issue must be taken into consideration in the final evaluation of the proposed lateral redesign for the Center for Science & Medicine.

### 3.9 Cost Analysis and Comparison

A rough cost estimate was performed to investigate any savings obtained by changing the CSM's lateral system from a dual braced frame / moment frame system to that of a core-only shear wall system. R.S. Means Cost Data from 2008 was used to estimate costs, and all data acquired is specific to New York, NY. First, cost incurred from shear wall construction was considered:

| CONCRETE SHEAR WALL COST DATA | | | | | | |
|---|---|---|---|---|---|---|
| Item | Unit Cost | | # Units | | Total Cost |
| Shear Wall: | | | | | |
| Materials | $117 | / CY | 2116 | CY | $247,572 |
| Placement | $33.65 | / CY | 2116 | CY | $71,203 |
| Reinforcement | $1,730.00 | / ton | 137.4 | tons | $237,702 |
| | | | TOTAL EXPENSE | | $556,477 |

Assumptions:

- Material includes aggregate, sand, Portland cement, and water. Excludes all additives and treatments.
- Placement (with pump) includes labor & equipment.
- Reinforcement includes material & labor costs.
- Formwork is not considered in this analysis, since pricing information could not be obtained for PERI Automatic Climbing System.
- As a "rule of thumb" used by local engineers, moment connections were assumed to be $1,000 per weld at a beam-to-column flange connection, which equals $2,000 per connection.
- All totals include 10% overhead & profit.

Next, savings were considered to account for the elimination of heavy W-shapes at the building core, the downsizing of large W-shapes at the perimeter, and the elimination of full penetration welds at original beam-to-column flange moment connections.

| STEEL & MOMENT CONNECTION COST DATA | | | | | | |
|---|---|---|---|---|---|---|
| Item | Convert to: | Unit Cost | | # Units | | Total Cost |
| Steel: | | | | | | |
| Core Columns W14, 232'-0" | (eliminate) | $267 | / LF | 2320 | LF | $618,280 |
| Core beams W 24 | (eliminate) | $147.91 | / LF | 3225 | LF | $477,009.75 |
| MF Beams W30x124 | W24x55 | $88.72 | / LF | 252 | LF | $22,357.44 |
| MF Beams W30x173 | W24x55 | $172.77 | / LF | 378 | LF | $65,307.06 |
| MF Beams W30x108 | W24x55 | $77.42 | / LF | 927 | LF | $71,768.34 |
| MF Beams W30x132 | W24x55 | $112.95 | / LF | 168 | LF | $18,975.60 |
| MF Beams W30x148 | W24x55 | $136.07 | / LF | 336 | LF | $45,719.52 |
| MF Beams W30x99 | W24x55 | $64.21 | / LF | 394 | LF | $25,298.74 |
| MF Columns W36, 184'-0" | W24 | $62.00 | / LF | 1288 | LF | $79,856.00 |
| MF Columns W24, 184'-0" | W14 | $21.00 | / LF | 2944 | LF | $61,824.00 |
| Moment Connections | (eliminate) | $1,000 | / weld | 1346 | welds | $1,346,000 |
| | | | | TOTAL SAVINGS | | $2,832,396 |

TOTAL SAVINGS - TOTAL EXPENSE = NET SAVINGS

$2,832,396 - $556,477 = + $2,275,919

Therefore, a net savings of almost $2.3 million was attained by switching to a core-only shear wall lateral system.

### 3.10 Conclusion and Recommendations

Overall, the redesign of the Center for Science & Medicine's lateral system proved to be an efficient and economical solution to the issues presented by the original design. Below is a summary comparing the advantages and disadvantages of each system.

Table 3.17 Lateral System Comparison

| | Braced / Moment Frame Design (Original) | Shear Wall Design (Proposed) | Conclusion |
|---|---|---|---|
| *Sitffness* | BF1: 34% of E-W load<br>BF3: 33% of E-W load<br>MFA: 19% of E-W load<br>MFC: 14% of E-W load<br><br>BF2: 35% of N-S load<br>BF4: 57% of N-S load<br>MFB: 3% of N-S load<br><br>MFD: 5% of N-S load | SW1: 58.1% of E-W load<br>SW3: 49.1% of E-W load<br><br>SW2: 24.9% of N-S load<br>SW4: 75.1% of N-S load | Moment frames in original design have very little stiffness due to their double-heighted configuration. Their inefficiency in resisting load is not worth the cost of welded connections.<br><br>Two shear walls are able to resist entire wind load in either direction, as opposed to 4 lateral load resisting elements having to work together to resist load in the original design.<br><br>BETTER SOLUTION: PROPOSED SYSTEM |
| *Coordination* | Potential issues with braced frame placement in relation to designed wall openings. Potential constructability issues with heavy double tee braces. | Potential issues with placement of reinforcement and complicated opening patterns. | In proposed design, reinforcement was kept as uniform as possible, and wall openings were placed as symmetrically as possible to combat coordination / constructability issues.<br><br>BETTER SOLUTION: INCONCLUSIVE |
| *Cost* | Moment connections are very costly (approximately $1,000 per connection), and heavy W-shapes incur a large expense as well. | Potential cost incurred by automatic self-climbing formwork, but this is unknown.Neglecting formwork, $2.3 million saved. | Materials needed for proposed design are less expensive than materials required for original. The cost of self-climbing formwork is unknown, but it likely will not exceed the amount saved from BF & MF elimination.<br><br>BETTER SOLUTION: (LIKELY) PROPOSED SYSTEM |
| *Schedule* | Welding of moment connections is likely on critical path for construction, which is undesirable. | As long as automatic climbing system is used for concrete pours, no negative scheduling effects are forseen. | Schedule's critical path would likely be shortened if moment connections were eliminated from design, thus reducing overall construction time. ACS formwork would allow core to be poured simultaneouly with steel erection, thus having no negative effect on schedule.<br><br>BETTER SOLUTION: PROPOSED SYSTEM |

Aside from a few unknown factors mentioned in the table above, it is fair to claim the shear wall system as a more efficient and less costly system than the original design. Thus, it is recommended to implement this solution for the Center for Science & Medicine in order to optimize lateral system behavior, ease coordination issues, reduce overall cost, and potentially shorten the construction schedule.

# 4.0 BREADTH # 1 – CM & Building Information Modeling

## 4.1 Background Information

Building information modeling (BIM) is defined as the ultimate compilation of construction and design information for a building, housed in a database and graphically represented through a computer software program. Within a building information model, relationships can be created between building elements that allow software to manage interactions, and in some instances, allow the objects to respond to changes in the design. The result is a single, multi-dimensional, "intelligent" design.

The use of this new technology in A & E design firms holds much promise. Designing in 3D could potentially reduce interdisciplinary conflicts within the office, allowing for easy and efficient coordination of drawing sets. BIM is likely to increase the productivity of a design team, improve the quality of work generated, and provide a better understanding of how a building works and fits together for the benefit of both designers and clients. Essentially, there is a good chance that BIM will eventually transform the way we design buildings, engineer systems, and interact with both coworkers and clients within the AEC community.

Interestingly, one of the unique aspects of the Center for Science & Medicine is that it is being designed with 3-D software, utilizing the latest BIM (building information modeling) technology. Since BIM is a relatively new design tool, it is a question as to whether this cutting-edge design method will truly pay off. The following breadth study will consider the positive and negative effects BIM has had on the design process of the Center for Science & Medicine thus far. While problems are posed by up-front software and training costs, time spent on implementing the system within the design team, and possible tension created by the new design tool between different generations of engineers, long-term benefits will be investigated and weighed against the negative. From here, conclusions can be drawn regarding the effectiveness of building information modeling, and recommendations can be made to the CSM's design team as well as the rest of the AEC community.

## 4.2 Method of Research

To investigate the use of 3D modeling for the Center for Science & Medicine project (currently on-going), interviews were conducted with Skidmore, Owings & Merrill professionals on the CSM design team. Specifically, the following project team members provided consultation:

- Project Structural Engineer
- Project Architect
- Digital Design Specialist
- Digital Design Coordinator / Structural Drafter

The main objectives of the interview process were:

- To understand how BIM techniques are implemented within Skidmore, Owings & Merrill as well as for this specific project
- To evaluate the advantages and disadvantages of the technology
- To identify lessons learned by the project team

An outline of the specific interview questions asked to each project team member can be found in Appendix K.

### 4.3 Summary of Findings

Technology Adoption

- Autodesk's AutoCAD Revit 3D modeling program was selected by SOM for use on select projects. A recent initiative within the firm has pushed for BIM-use on at least one of every project type by the year 2009. The Center for Science & Medicine was the chosen laboratory project to be designed in 3D.
- Revit was selected over other 3D software because it seemed to be one of the easiest 3D design programs to learn and implement. Being an Autodesk product, the software has a large amount of support options and resources available to its users.
- Revit has been used on the CSM project since schematic design. Disciplines using Revit are: architecture, structural, and MEP. Furniture and equipment are also modeled to an extent.
- SOM offers a 4-day training course for employees assigned to Revit projects. Team members take the course together and work on their actual building model in the training course. These sessions are paid for by project budget. All interviewees agreed that training was essential, although most of the learning comes from experience with the software.

How it Works

- There is a company-wide graphical standard (similar to that of 2D drafting in AutoCAD) in developmental stages at SOM. This standard will eventually define detail libraries, component libraries, etc. A team of professionals is currently working on developing this standard. In the meantime, a set group of people reviews the designs graphically.
- A standard practice exists for information exchange between design disciplines, running in a weekly cycle:
  - There are 3 separate models for the CSM project.
  - Every Friday, architects post their updated architectural model to Buzzsaw for all engineers to access (but not change). This is called a static model.
  - The following Tuesday, engineering disciplines submit their specific models, and all are "linked."
  - On Wednesday, a coordination meeting is held to discuss any conflicts, problems, or major changes that were recognized after linking.

Organization of Staff

From the top down,

- Three partners oversee the entire design process: a management partner, a technical partner, and a design partner.
- Below the 3 partners are a senior technical coordinator, a senior designer, and a project manager.
- Below these coordinators are digital design specialists (architects acting as BIM managers) and senior level engineers.
- Junior project architects and junior engineers.
- Architectural drafters, technical drafters, and IT support.

Interviewees all agreed that this organization of staff was successful. As long as coordinators at upper levels worked well together, cooperation would trickle down to the rest of the project team.

Results

- The use of Revit creates noticeable improvements in project quality. Much more coordination takes place much earlier in the process. A level of detail in design is attained that was never even approached before. There are significantly less issues once CD phase is reached because so many have been resolved early-on.
- BIM has enabled the project team to design with more accuracy, as they must work in all three dimensions rather than just two.
- Disadvantages: 3D software is very taxing on computers. Hardware technology is not as advanced as the software technology, so time is lost waiting for computers to load models and run the program.
- Efficiency has neither increased nor decreased with 3D modeling. BIM is not about time savings, but rather the quality of the product. There is still the same number of people on a job with this software.
- Workers are generally more productive when using 3D software. Revit allows the project team to truly understand the building and how all systems work together. This understanding increases productivity in terms of coordination.
- The total number of design hours has not increased noticeably. More hours are budgeted for schematic design, but less for construction documents. Therefore, the total number of design hours is generally the same in 3D and 2D.
- The learning curve: Most software-users start improving significantly after about one month. In 2-3 months, the average user in proficient in Revit.

The Next Step

- Once a design is completed, a copy of the 3D model is given to the contractor.
- The model is strictly for reference. It conveys the design intent, but not everything is detailed. Contractors must rely on 2D drawings for actual construction. (A disclaimer is attached to the model explaining this.)

Lesson Learned / General Recommendations

General Problems:

- Sometimes BIM expertise is limited, so it takes longer for software / modeling problems to be solved.
- In the case of the CSM, the structural analysis model was unable to load into the BIM model.
- Models become cumbersome and inefficient if too many components are modeled.

Advice for architects:

- Components available via download (ie furniture, people, etc.) make the model "heavy." Too many components causes the model to be too large in size and run too slowly. The model becomes cumbersome and inefficient.
- Be flexible in the way elements are represented graphically (may not be the same as 2D standard)

Advice for any user:

- Get familiar with the software as soon as possible.
- Have a mentor.
- Keep the model lightweight and efficient. Consider you intent to decide if any element should / should not be modeled.
- Patience

- 3D software causes the roles of the engineer and draftsperson to change considerably. Engineers are able to “draft” their designs in Revit, while drafters must have an understanding of the engineered aspects of design. Drafters do not simply follow instruction from the red pencil on drawing mark-ups.
- Remember that Autodesk is still working to improve their software. There are, inevitably, quirks in the system.
- Engineers need to be willing to design quicker up-front, but at the same time, architects must understand that these preliminary designs are subject to change since they were set so early on.

### 4.4 Conclusions and Recommendations

The insight provided by the design professionals on the CSM project team was invaluable. It was made clear that 3D modeling is the most efficient and effective tool for architectural design and engineering that can be utilized within the walls of an office. Buildings are designed faster and more thoroughly, coordination begins early-on, and communication between different designers on a project team is almost seamless. The final product of a project modeled in 3D well surpasses any 2D drawing effort of the past. Building information modeling, if implemented in an organized and careful way, has the potential to change the AEC community for the better, one design firm at a time. It’s use on the Center for Science & Medicine project will only benefit the design team, the contractor, and the occupant when all is said and done.

# 5.0 BREADTH # 2 – Laboratory Lighting Redesign

## 5.1 Background Information

The lighting system in a laboratory can play a critical role in the productivity and success of its occupants. It is important that these systems are designed with careful consideration in order to ensure a positive, productive, and safe work environment. For laboratories, direct lighting systems provide the best option for achieving the right illuminance levels. Another factor that must be considered when designing the lighting for a laboratory is the location of work benches. Light fixtures must be placed in line with lab benches so that the maximum amount of light is distributed onto the work plane, and so that anyone sitting at a lab bench does not cast a shadow onto the surface of interest. Luminaires must also be placed in continuous rows to produce the most desirable light distribution in the space.

## 5.2 Existing Conditions

There are four wet labs on each of the five typical laboratory floors of the Center for Science & Medicine, giving a total of twenty wet lab spaces. Lab modules are spaced at 11', and there are 6 modules per laboratory. Each lab bench is lit by surface-ceiling mounted fluorescent wraparound fixtures and by task lighting at the level of the work plane. The main circulation space lining the far edge of the laboratory is lit by recessed fluorescent fixtures. The existing design for these typical lab spaces is summarized below and shown in plan on the following page.

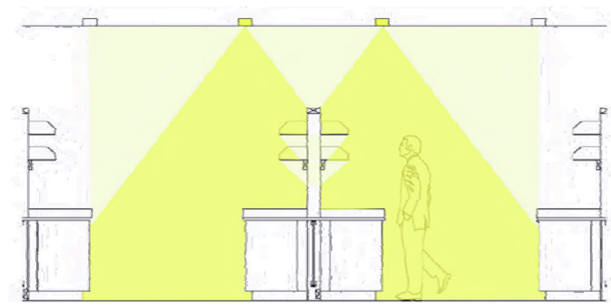


*Figure 5.1: Direct Lighting for a laboratory, courtesy of CUH2A, Inc.*

**LIGHTING DESIGN CRITERIA**

- ASHRAE limits lighting power density (LPD) to 1.4 W/ft$^2$ in a laboratory space.
- The laboratories were designed for a target illuminance level of:
  Ambient: 40-50 footcandles
  Workplane: 80 footcandles

**ROOM FINISHES**

- Walls: gypsum wall board
- Floor: vinyl tile
- Ceiling: acoustic tile and GWB

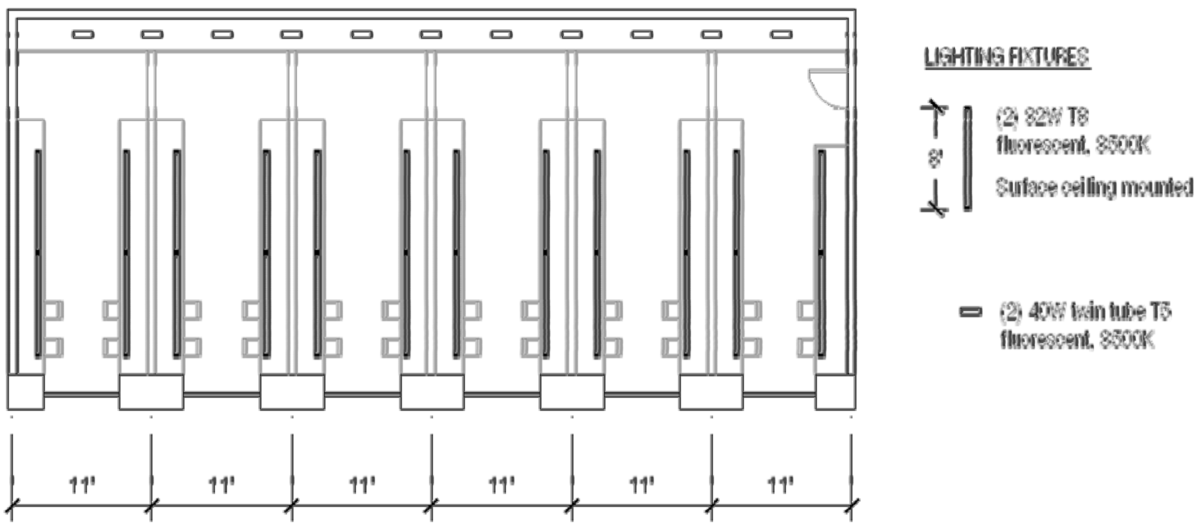
**DESIGN DESCRIPTION**

Lighting Design Description
Direct illumination; Ceiling at 9'-6"
High efficiency fluorescent fixtures
Task lighting integrated into laboratory benches
Luminaries sealed, gasketed, and rated for wet locations

Lighting Fixtures

- Manufacturer: National DTF/232RS/EG/T8
  Surface ceiling mounted
  (2) 32W T8 fluorescent, 3500K
- Manufacturer: Lightolier DPA/2/X/16/L/5/2/FT/X/3
  2'x2' recessed fluorescent fixture
  (2) 40W twin tube T5 fluorescent, 3500K

TYPICAL WET LAB LIGHTING PLAN




To evaluate existing conditions, the laboratory space was modeled in AGI32. Photometric data for the fixture specified in the existing design could not be obtained, so a similar fixture was chosen to model the conditions. Specifically, Prudential Lighting's P-1220-2T8-WA fixture was selected for modeling purposes. This is a wraparound, surface-mounted fluorescent fixture suitable for wet laboratory environments, which is what the existing design calls for. Below is a summary of the chosen fixture used to model existing conditions; an official cut sheet can be found in Appendix L.

Prudential Lighting
P-1220-2T8-WA

Description: Surface-mounted, fluorescent wraparound

Lamping: 2-F32T8 (48in) lamps

UL Listed for damp location, with a wet-listing option available.

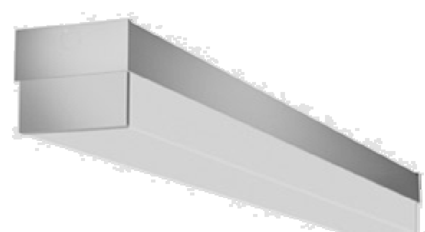


*Figure 5.2: Existing lighting fixture above lab benches*
*Courtesy of www.eLumit.com*

By modeling the space in AGI, the power density for the entire room could be determined, along with average illuminance levels on lab benches (at 37" above the floor). It was determined that the current design surpasses the power density limitation of 1.4 W/ft$^2$ for laboratory spaces, and illuminance levels were too high as well. It is clear from the findings shown below that the wet laboratory spaces were over-designed.

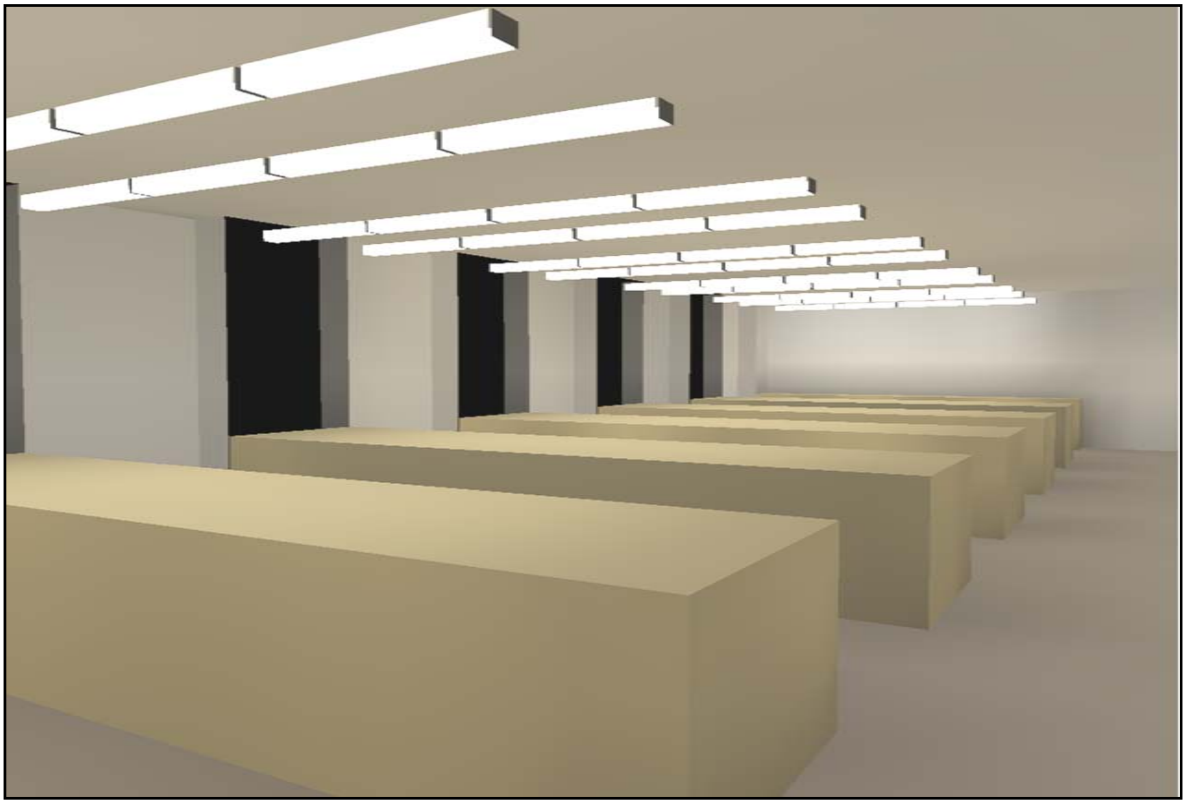


*Figure5.2: Basic rendering of typical wet laboratory, existing design (shown without partitions)*

Table 5.1 Existing Lighting & Power Levels

| | Calculated Value | | Limit | Reference |
|---|---|---|---|---|
| Power Density | 1.76 W/ft$^2$ | > TOO HIGH | 1.4 W/ft$^2$ maximum | ASHRAE Standard |
| Ambient Illuminance | 59.2 FC | > TOO HIGH | 40 - 50 FC target | Criteria provided by designer. |
| Work Plane Illuminance | 97.4 FC | > TOO HIGH | 70 - 80 FC target | Criteria provided by designer. |

Thus, the typical wet lab space is over-designed according to industry standards and original design criteria. To reduce power density and illuminance levels, a new lighting layout will be proposed in the following section.

### 5.3 Proposed Lighting Redesign

Design Goals:

- Decrease power density to an acceptable level (less than 1.4 W/ft$^2$)
- Decrease illuminance levels to achieve target illumination
- Increase the system's overall efficiency

Selected Luminaire (to replace existing 32W-T8 fluorescent fixture):

Corelite Class R2 –Shallow Recessed Fluorescent
R2-WL-1N5-1D-120-14-T1-LG

Description: 1'x4' shallow, recessed fixture

Lamping: 1-T5 (48in) lamp

Options: Lens gasketing (for damp locations)
Anti-microbial paint

*Figure 5.3: Corelite Class R2 Luminaire*
*Courtesy of www.cooperlighting.com*

In an attempt to T5 lamps of the Class R2 Series provide more lumens per watt in comparison to traditional T8 luminaires.

85% efficiency

Supports energy-saving ballasts and controls

The Class R Effect: The optics provided by this type of luminaire have two components. The direct component illuminates horizontal task surfaces, while the indirect component disperses a small amount of light to vertical surfaces. The combination of these distributions results in a low-energy space with excellent vertical visibility.

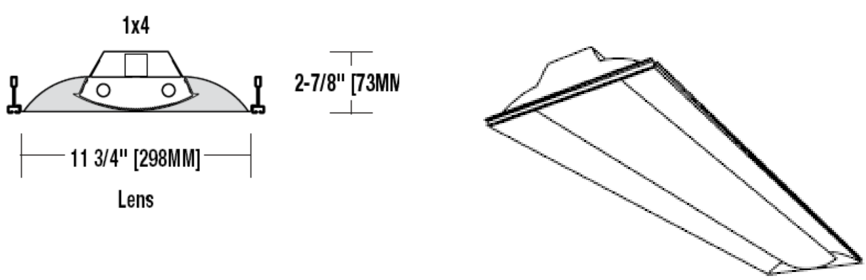
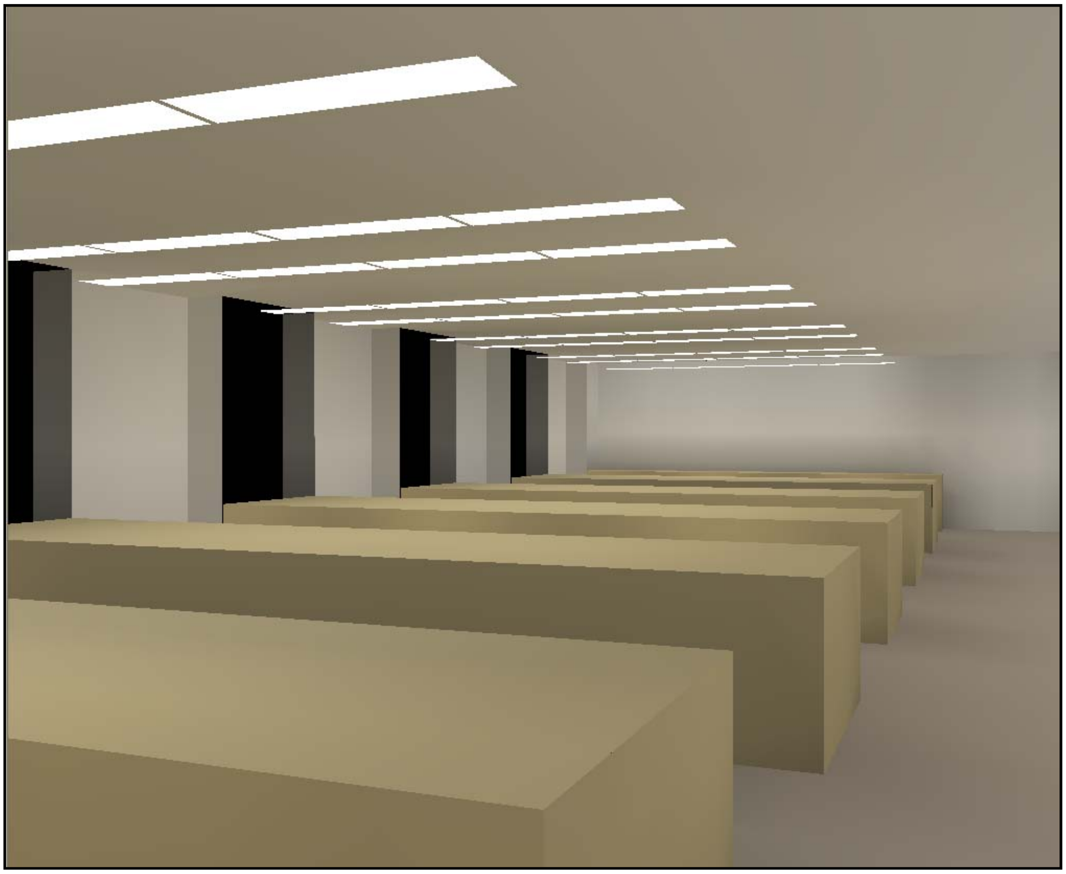



*Figure 5.4: Corelite Class R2 Luminaire, 1'x4'*
*Courtesy of www.cooperlighting.com*



| | Calculated Value | | Limit | Reference |
|---|---|---|---|---|
| Power Density | 1.02 W/ft$^2$ | < Acceptable | 1.4 W/ft$^2$ maximum | ASHRAE Standard |
| Ambient Illuminance | 51.1 FC | ≈ Acceptable | 40 - 50 FC target | Criteria provided by designer. |
| Work Plane Illuminance | 77.0 FC | = Acceptable | 70 - 80 FC target | Criteria provided by designer. |

PROPOSED LIGHTING PLAN

(Fixture locations are the same as existing layout, so a plan will not be shown).

### 5.4 Conclusion and Recommendations

The selected Class R2 fluorescent fixture proved to be a wise choice for the purposes of this space. The fixture not only provides a superior light distribution ideal for people working in a laboratory environment, but it is also more efficient than the traditional T8 lamps used in original design and it works with energy saving ballasts. Target illuminance levels were achieved for ambient conditions and for tasks performed on the work plane. Power density was reduced by about 30%, and the space now complied with ASHRAE Standard 91.1-2007. It is recommended that all 6 of these wet laboratory spaces within the Center for Science & Medicine be redesigned in accordance with this proposed system to save energy and achieve a better workplace for occupants.

## 6.0 SUMMARY & CONCLUSIONS

The primary goal of this report was optimization of existing design. Several building systems and processes were evaluated and redesigned with efficiency as a driving factor. Specifically, the optimization of the following items was addressed:

- Lateral load resisting system
- Construction means & methods of this system
- The design and coordination process
- A typical laboratory lighting system

The lateral system re-design consists of a core-only system of coupled shear walls which replace the braced frames currently existing at the building core. These shear walls are designed to resist 100% of the lateral load in both directions, therefore also eliminating the need for perimeter moment frames. It has been determined that the proposed core-only system provides more stiffness than the current dual system and therefore presents a more efficient means of lateral force resistance. Moreover, the proposed design is expected to require less construction time while saving cost in the elimination of expensive moment connections and heavy framing members.

The investigation of BIM implementation techniques used on this project by Skidmore, Owings and Merrill, evaluated the advantages and disadvantages of the technology and identified lessons learned by the project team. From the research conducted, it was determined the BIM is, indeed, a valuable and pertinent design tool with its potential benefits far greater than its shortcomings. Building information modeling is the future of the AEC industry, and the successful implementation of the technology by SOM can serve as an example to other firms adopting the software.

A final optimization was made by considering the lighting system of a typical laboratory. Lighting can be critical in laboratory spaces, where important procedures are carried out and visibility is crucial. There are 6 typical "wet" laboratory spaces in the Center for Science and Medicine, and investigation has determined that the lighting systems of these spaces have actually been overdesigned. An alternative design to the existing lighting system was proposed in an attempt to reduce illuminance levels and LPD. The redesign considered efficiency, aesthetics, and environment in the selection of new luminaires. Final design of the space successfully achieved target illuminance levels and lighting power density. Thus, this optimized system can be put in place throughout the building at every wet lab location to reduce energy use and better comply with industry standards.