ECMC Skilled Nursing Facility

Architectural Engineering Senior Thesis 2012

Class: AE 482
Subject: Final Report
Faculty Consultant: Dr. Ali Memari
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Brian Brunnet | Architectural Engineering | Structural Option

ECMC Skilled Nursing Facility

Buffalo, NY

Building Statistics

Location: 462 Grider Street Buffalo, NY 14215
Size: 296,000 SF
Number of Stories: 5 stories + Penthouse Level
Completion Date: July 2012
Project Cost: $95 Million
Delivery Method: Design - Build

Architecture

- Radial 8 wing design with central core
- Mainly brick facade with stone veneer base trim
- Horizontal / Vertical Aluminum panels used for solar shading
- Garden Terraces with Green Wall on each floor to provide a sufficient outdoor space

Structural System

- Spread/strip footings for foundation system
- W-flange Steel Frame Superstructure
- Composite Decking on every floor
- Concentric HSS Brace Frame Lateral System

MEP Systems

Mechanical:

- Eight Temtrol AHU's ranging from 9,200 to 42,000 CFM
- Four Energy Recovery Wheels used in resident room areas
- VAV boxes with reheat coils found throughout the building

Electrical:

- Both 120/208V and 277/480V 3 Phase 4 Wire Systems throughout building
- Three existing 750kW Generators in generator room found on site
- Use of CFL, Fluorescent, MH, LED, and Fiber optic lighting

Project Team

Owner: ECMC Corporation
Architect: Cannon Design
Construction Manager: LP Ciminelli
Structural Engineer: Cannon Design
Civil Engineer: Watts Architecture & Engineering
MEP Engineer: M/E Engineering

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http://www.engr.psu.edu/ae/thesis/portfolios/2012/bab408/index.html

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Acknowledgements

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Lastly, I would like to thank my family and friends for their continuous support and encouragement which helped me complete my senior thesis and complete my education at the Pennsylvania State University.

Executive Summary

The ECMC Skilled Nursing Facility is a new 296,000 square foot assisted living facility located on the ECMC campus in Buffalo, NY. The building has unique design features, such as a radial plan geometry and sloped roof layout, and the project cost roughly $95 million to construct. The main framing system consists of composite steel framing with a large mechanical penthouse located on the top floor. The building's main lateral system consists of 16 concentrically braced frames, where 8 frames can be found at the end of each wing while another 8 frames are located surrounding the building core.

This final thesis report examines the redesign of the buildings structural system in a different location, primarily the high seismic region of Los Angeles, CA. In this new location, the ECMC Skilled Nursing Facility will be prone to high seismic forces, soil liquefaction, and large deflections. Specifically, the structural redesign will focus on three major structural systems:

- Foundation System
- Gravity System
- Lateral Force Resisting System

To explore alternative solutions for earthquake design, base isolation was incorporated into the buildings structural lateral force resisting system. Without isolation, the building period for the original design in this new location was considered slightly flexible (T=1.475 sec). However, after base isolation was incorporated into the new design, the building period increased to 4.180 sec, reducing the damaging effects of story drift. The axial loads experienced in the ground floor columns was quite large, causing many of the column members to increase in size, some reaching sizes of W14x283.

Another alternative to reducing seismic forces was by reducing the slab on deck depth. To do this, the existing 2" composite decking was replaced with 3" composite decking, allowing for more strength at larger spans and also a reduction in slab thickness of 5-1/4" to 5". Framing members were sized up slightly from their original design; however it is potentially due to the increase in live load from 40psf to 80psf. Columns remained relatively unchanged except for a few throughout the building.

The analysis of the structural depth begins with a verification of dead and live loads found using the IBC 2006 edition as well as ASCE 7-10. Afterwards, lateral loads such as wind and seismic were calculated using ASCE 7-10, following both the Main Wind Force Resisting System procedure for wind and the Equivalent Lateral Force procedure for seismic. Once these loads were found, specific load combinations were chosen to determine which load case or combination of load cases controlled the design of the lateral system. It was found that seismic effects produced a base shear of 6550 kips and wind produced a base shear of 1071 kips in both the X and Y directions. Overturning moments of 350,694 ft-k and 54,353 ft-k were found for both seismic and wind respectively.

Not only should the structural system be evaluated in this new location, so should the mechanical HVAC systems. Los Angeles, CA is considered to have a semi-arid climate, which is largely different than that of Buffalo, NY. Although temperatures do not vary much in the summer season, winter can produce much colder temperatures in the Buffalo, NY location. An enthalpy verification check of the HVAC systems was performed for both summer and winter seasons, and it was found that the existing systems were adequate for winter heating and summer cooling. Additionally, since the HVAC system consists of a variable air volume (VAV) system, the volume of supply air can be adjusted to produce the necessary comfort levels required by industry standards.

With changes in building design come cost and schedule impacts. With the incorporation of lead rubber base isolation in the structures lateral system, the project cost increased drastically since each isolator was estimated to cost around $20,000 each. In addition, the increase in column shape sizes also produced a slight increase in structural steel costs of roughly $200,000. Deep foundations had also contributed to the project cost in a negative way, however they impacted the project schedule the most by adding another 156days to the schedule for installation. Overall, it was expected that the project cost and schedule would increase due to the use of base isolation and deep foundations. However, since the building does host a large number of residents and a higher risk category, it seemed to be the necessary solution for design in the area of Los Angeles, CA.

Building Overview

Function

The new ECMC Skilled Nursing Facility serves as a long term medical care center for citizens found throughout the region. The building is located on the ECMC campus found at 462 Grider Street in Buffalo, NY. This site was chosen to bring residents closer to their families living in the heart of Buffalo. As you can see here in Figure 1, the site sits right off the Kensington Expressway, providing ease of access to commuters visiting the ECMC Skilled Nursing Facility. Since the Erie County Medical Center is found within close proximity of the new building, residents can receive fast and effective care in an event of emergency.

Figure 1: Aerial view of ECMC Skilled Nursing Facility site shown in white. Photo courtesy of Bing Maps.

The new facility is the largest of four new structures being built on the ECMC campus located in central Buffalo, NY. The new campus will also contain a new Renal Dialysis Center, Bone Center, and parking garage. Each of the three new facilities will be connected to the main medical center via an axial corridor, which provides enclosed access to emergency rooms, operation rooms, and other facilities found within the Erie County Medical Center.

Building Architecture

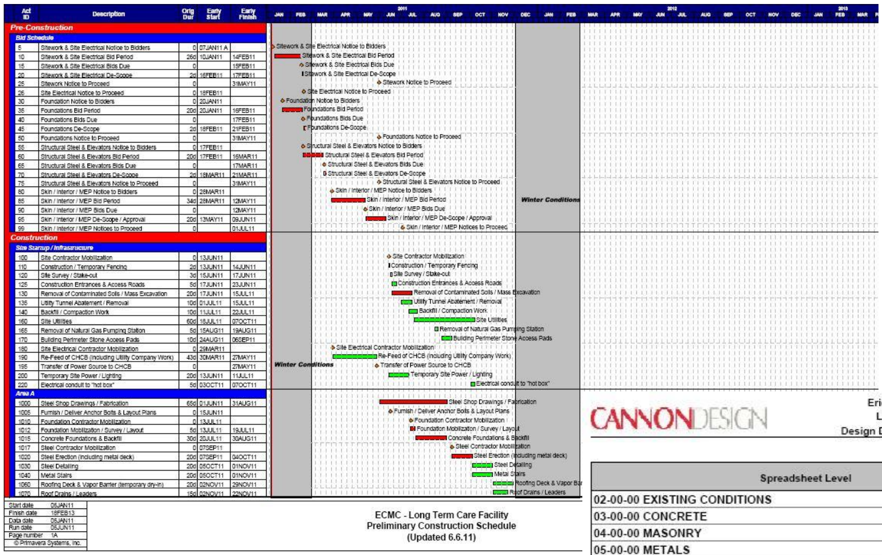
The new Erie County Medical Center Skilled Nursing Facility is a five-story 296,489 square-foot building offering long-term medical care for citizens in the region. The facility consists of an eight-wing design with a central core. The main entrance to the building is located to the east and is sheltered from the elements by a large porte-cochere. There is a penthouse level that contains the facility's mechanical and HVAC units. Each floor features one garden terrace, providing an outdoor space accessible to both residents and staff. The exterior of the building is clad in brick, stone veneers, composite metal panels, and spandrel glass curtain wall system.

Figure 2: Exterior view of stacked garden terraces, green wall, and the building's vertical and horizontal shading panels. Rendering courtesy of Cannon Design.

The facility also incorporates green building into many of its elegant features. The composite metal panels that run vertically and horizontally across each wing of the building, visible in Figure 2, provide solar shading along with architectural accent. A green wall is featured on each outdoor garden terrace, providing residence with a sense of nature and greenery. The ECMC Skilled Nursing Facility provides an eclectic, modern atmosphere and quality care for long-term care patients found within the Buffalo area.

Construction Management

The ECMC Skilled Nursing Facility was constructed as a design-bid-build delivery method. The project broke ground on June 13th, 2011 and is projected to be completed in February of 2013. The projected cost of the ECMC Skilled Nursing Facility is $79,000,000 and LP Ciminelli Construction was awarded the general contractor for the project. The ECMC-SNF is classified as a 1A Non-Combustible Fire Resistive Construction, which is one of the highest fire resistance construction types you can attain. Figure 3 below is a sample of the project cost and schedule.



Erie County Medical Center
Long Term Care Facility
Design Development - December 2010

Estimate Summary
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Spreadsheet Level		Takeoff Quantity	Total Cost/Unit	Total Amount
02-00-00 EXISTING CONDITIONS		275,000.00 GSF	0.29 /GSF	79,417
03-00-00 CONCRETE		275,000.00 GSF	11.54 /GSF	3,173,736
04-00-00 MASONRY		275,000.00 GSF	18.32 /GSF	5,036,877
05-00-00 METALS		275,000.00 GSF	29.44 /GSF	8,097,065
06-00-00 WOOD, PLASTICS & COMPOSITES		275,000.00 GSF	6.87 /GSF	1,889,780
07-00-00 THERMAL & MOISTURE PROTECTION		275,000.00 GSF	9.87 /GSF	2,714,952
08-00-00 OPENINGS		275,000.00 GSF	13.89 /GSF	3,819,879
09-00-00 FINISHES		275,000.00 GSF	28.71 /GSF	7,895,446
10-00-00 SPECIALTIES		275,000.00 GSF	7.74 /GSF	2,127,649
11-00-00 EQUIPMENT		275,000.00 GSF	5.97 /GSF	1,642,939
12-00-00 FURNISHINGS		275,000.00 GSF	0.04 /GSF	10,083
14-00-00 CONVEYING EQUIPMENT		275,000.00 GSF	5.12 /GSF	1,406,764
21-00-00 FIRE SUPPRESSION		275,000.00 GSF	3.94 /GSF	1,084,158
22-00-00 PLUMBING		275,000.00 GSF	20.52 /GSF	5,642,521
23-00-00 HVAC		275,000.00 GSF	35.99 /GSF	9,897,738
26-00-00 ELECTRICAL		275,000.00 GSF	21.03 /GSF	5,781,824
27-00-00 COMMUNICATIONS		275,000.00 GSF	10.15 /GSF	2,790,685
28-00-00 ELECTRONIC SAFETY & SECURITY		275,000.00 GSF	3.94 /GSF	1,083,727
31-00-00 EARTHWORK		275,000.00 GSF	4.45 /GSF	1,223,002
32-00-00 EXTERIOR IMPROVEMENTS		275,000.00 GSF	1.88 /GSF	516,946
33-00-00 UTILITIES		275,000.00 GSF	2.88 /GSF	791,724

Estimate Totals

Description	Amount	Totals	Rate	B.	$ / Unit
	66,706,911	66,706,911			239.13 /GSF
GMP RESERVE	1,334,138		2.00 %	T	4.78 /GSF
Bid Day Total	**1,334,138**	**68,041,050**			**243.91 /GSF**
CM GENERAL CONDITIONS	2,500,000			L	8.96 /GSF
CM FEE	1,871,129		2.75 %	T	6.71 /GSF
Total Construction Cost	**4,371,129**	**72,412,178**			**259.58 /GSF**
PARKING GARAGE ALLOWANCE	6,500,000				
Total w/ Parking Garage		**78,912,178**			**282.88 /GSF**

Figure 3: ECMC Skilled Nursing Facility cost estimate (right) and project schedule (above). Cost and schedule courtesy of Cannon Design.

Mechanical System

The mechanical system for the ECMC Skilled Nursing Facility was designed to service the multiple areas of the building, mainly patient rooms and the central public space located in the building core on each floor. The AHU's servicing these two main spaces range in size from 9,200 to 42,000 CFM. Additionally, four energy recovery wheels are used in the resident room areas. VAV boxes with reheat coils can also be found throughout the building. The majority of these AHU's can be found at the 5th story in the rooftop Penthouse, which minimalizes rooftop clutter and protects the mechanical systems from the elements. Figure 4 shows a typical VAV AHU system from Temtrol Custom Air Handlers.

Temtrol®

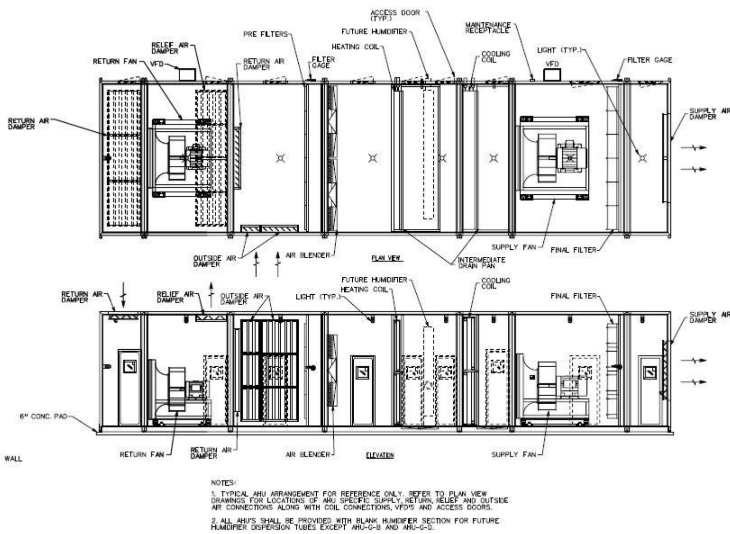


Figure 4: Typical VAV AHU. Detail courtesy of Cannon Design.

Lighting & Electrical System

The electrical service to the ECMC Skilled Nursing Facility runs on both a 120/208V and 277/480V 3-Phase 4-Wire system with the use of on-site transformers to step down voltages when necessary. As usually found in most hospitals, there are three existing 750kW generators in the generator room found on site to service the ECMC Skilled Nursing Facility in case of an emergency. The use of CFL, Fluorescent, MH, LED, and Fiber Optic lighting can be found throughout the entire facility.

Structural Systems Overview

The ECMC Skilled Nursing Facility consists of 8 wings and a central core, with an overall building footprint of about 50,000 square feet. The building sits at a maximum height of 90’ above grade with a common floor to floor height of 13’-4”. The ECMC Skilled Nursing Facility mainly consists of steel framing with a 5” concrete slab on grade on the ground floor. The Penthouse level contains 6.5” thick normal weight concrete slab on metal deck. All other floors have a 5.25” thick lightweight concrete on metal deck floor system. All concrete is cast-in-place.

Foundation System

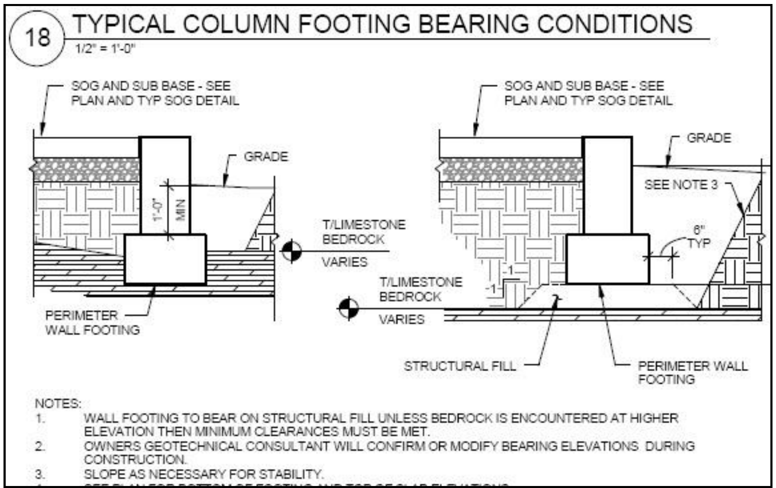


Figure 5: Footing bearing conditions. On bedrock (left detail), and on Structural Fill (right detail). Detail courtesy of Cannon Design.

The geotechnical report was conducted by Empire Geo Services, Inc. The study classified the soils using the Unified Soil Classification System, and found that the indigenous soils consisted mainly of reddish brown and brown sandy silt, sandy clayey silt, and silty sand. The ECMC Skilled Nursing Facility foundations sit primarily on limestone bedrock, although in some areas the foundation does sit on structural fill as you can see in Figure 5. Depths of limestone bedrock range from 2ft to 12ft. The building foundations of the ECMC Skilled Nursing Facility are comprised of spread footings and concrete piers with a maximum bearing capacity of 5,000 psf for footings on structural fill and 16,000 psf for footings on limestone bedrock. Concrete piers range in size from 22” to 40” square.

Floor System

The floor system on all floors except at the penthouse level consists of a 5.25" thick lightweight concrete floor slab on 2" - 20 gage metal decking, creating a one-way composite floor slab system. The concrete topping contains 24 pounds per cubic yard of blended fiber reinforcement. Steel decking is placed continuous over three or more spans except where framing does not permit. Shear studs are welded to the steel framing system in accordance to required specification. Refer to Figures 6 and 7 for composite system details.

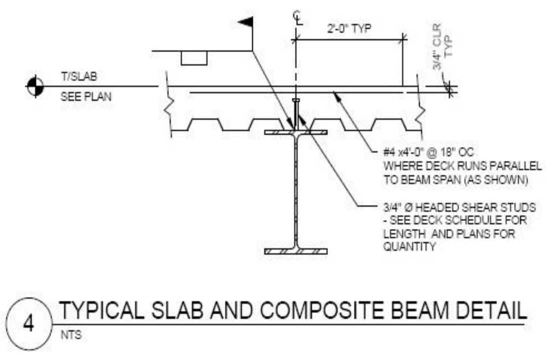


Figure 6: Composite deck system (parallel edge condition). Detail courtesy of Cannon Design.

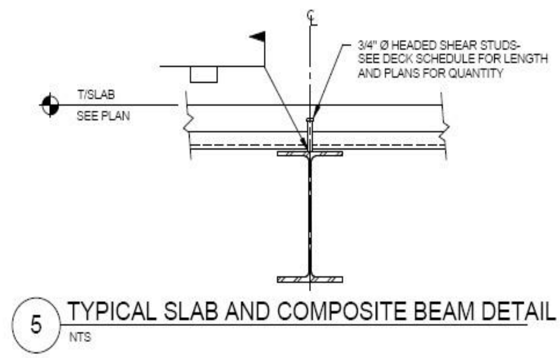


Figure 7: Composite deck system (perpendicular edge condition). Detail courtesy of Cannon Design.

Framing System

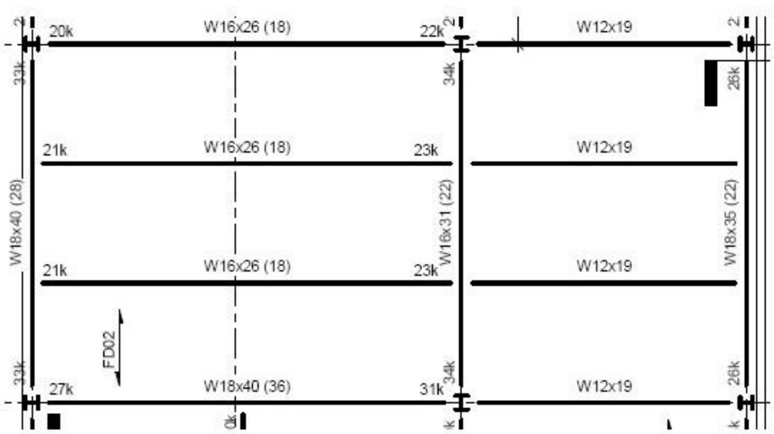


Figure 8: Typical bay layout for building wing. Detail courtesy of Cannon Design.

The structural framing system is primarily composed of W10 columns and W12 and W16 beams; however the girders vary in sizes ranging from W14 to W24, mainly depending on the size of the span and applied loads on the girder. Typical beam spacing varies from 6'-8"o.c. to 8'-8"o.c. Figure 8 shows a typical grid layout for a building wing. Columns are spliced at 4' above the 2nd and 4th floor levels, and typically span between 26'-8" and 33'-4".

Lateral System

The lateral resisting system consists of a concentrically brace frame system composed of shear connections with HSS cross bracing. Lateral HSS bracing is predominantly located at the end of each wing, and also found surrounding the central building core. Because of the radial shape of the building and symmetrical layout of the structure, the brace framing can oppose seismic and wind forces from any angle. The HSS bracing size is mainly HSS 6x6x3/8, but can increase in size up to HSS 7x7x1/2 in some ground floor areas for additional lateral strength. Figure 9 contains multiple details and an elevation of a typical brace frame for the ECMC Skilled Nursing Facility.

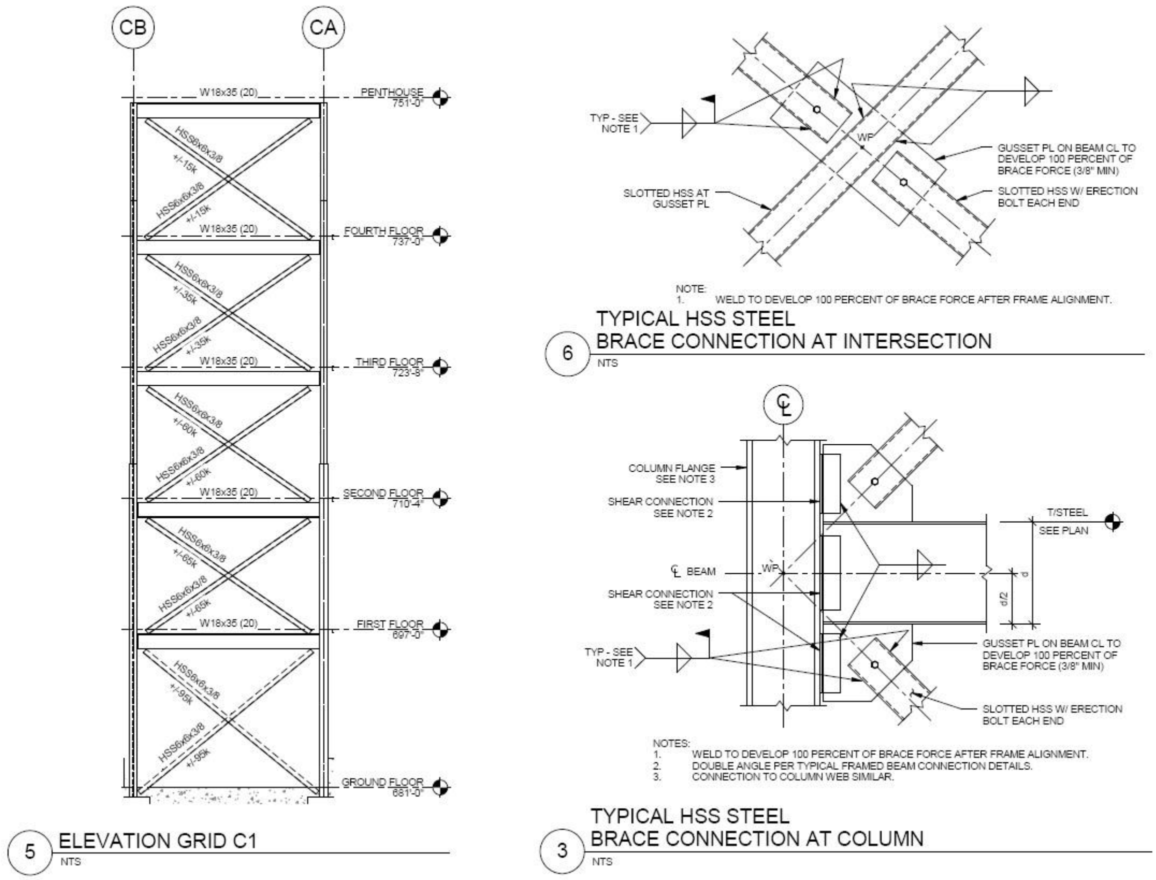


Figure 9: Typical lateral HSS brace frame (left). Typical HSS steel brace connection at intersection (upper right). Typical HSS steel brace connection at column (lower right). Details courtesy of Cannon Design.

Design Codes and Standards

Original Codes

Design Codes:

- ACI 318-02, *Building Code Requirements for Structural Concrete*
- ACI 530-02, *Building Code Requirements for Masonry Structures*
- AISC LRFD - 3rd Edition, *Manual of Steel Construction: Load and Resistance Factor Design*
- AWS D1.1 - 00, *Structural Welding Code - Steel*

Model Code:

- NYS Building Code - 07, *Building Code of New York State 2007*

Structural Standard:

- ASCE 7-02, *Minimum Design Loads for Buildings and Other Structures*

Thesis Codes

Design Codes:

- ACI 318-08, *Building Code Requirements for Structural Concrete*
- AISC Steel Construction Manual - 13th Edition (LRFD), *Load and Resistance Factor Design Specification for Structural Steel Buildings*

Model Code:

- IBC - 06, *2006 International Building Code*

Structural Standard:

- ASCE 7-10, *Minimum Design Loads for Buildings and Other Structures*

Material Properties

Structural Steel		
Wide Flange Shapes, WT Sections	ASTM A992	
Channels and Angles	ASTM A36	
Pipe	ASTM A53 Grade B	
Hollow Structural Sections (Rectangular and Round)	ASTM A500 Grade B	
Base Plates	ASTM A36 UNO	
All Other Steel Members	ASTM A36 UNO	
High Strength Bolts, Nuts, and Washers	ASTM A-325 / A-490 (Min. 3/4" Diameter)	
Anchor Rods	ASTM F1554	
Steel Shape Welding Electrode	E70XX	
Concrete	**F'c (psi)**	**Unit Weight (pcf)**
Footings	f'c = 3000psi	145
Foundation Walls	f'c = 4000psi	145
Slabs-on-Grade	f'c = 3000psi	145
Slabs-on-Steel Deck (Floor Deck 1)	f'c = 3000psi	145
Slabs-on-Steel Deck (Floor Deck 2)	f'c = 3000psi	115
All Other Concrete	f'c = 4000psi	145
Reinforcement		
Typical Bars	ASTM A-615 Grade 60	
Welded Bars	ASTM A-706 Grade 60	
Welded Wire Fabric	ASTM A-185	
Steel Fibers	ASTM A-820 Type 1	
Decking		
Floor Deck (both types)	2" Composite Metal Deck, 20 Ga.	
Roof Deck Type 1	1 1/2" Type B Metal Roof Deck, 20 Ga.	
Roof Deck Type 2	1 1/2" Type B Metal Roof Deck, 18 Ga.	
3/4" Shear Studs	ASTM A-108	

Table 1: This table describes material properties found throughout the building.

Architectural & Structural Floor Plan

The ECMC Skilled Nursing Facility is split symmetrically into four similar framing plans. Figures 10 and 11 below shows a side-by-side reference of the typical architectural floor plan and structural framing plan of one of the symmetric areas found in the existing ECMC Skilled Nursing Facility. As you can see, the columns, beams, and lateral braced frames are located within or along room partition walls.

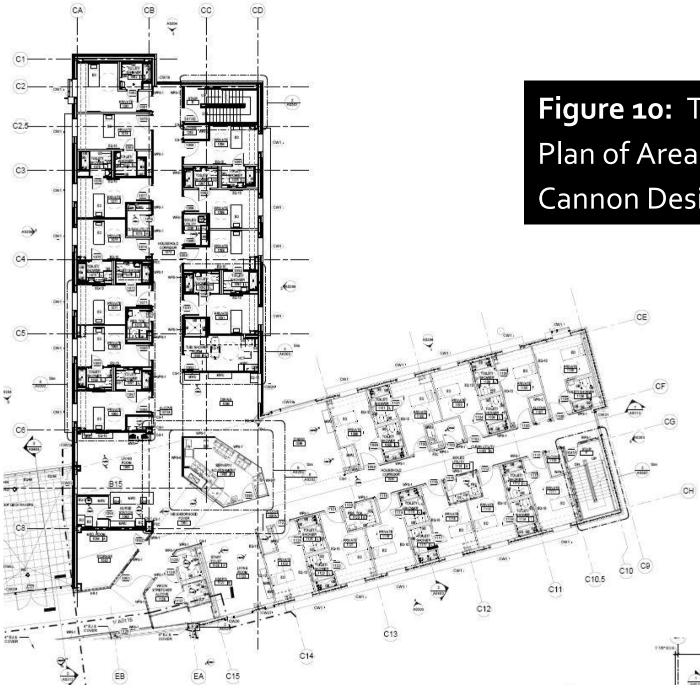


Figure 10: Typical Architectural Floor Plan of Area C (left). Detail courtesy of Cannon Design.

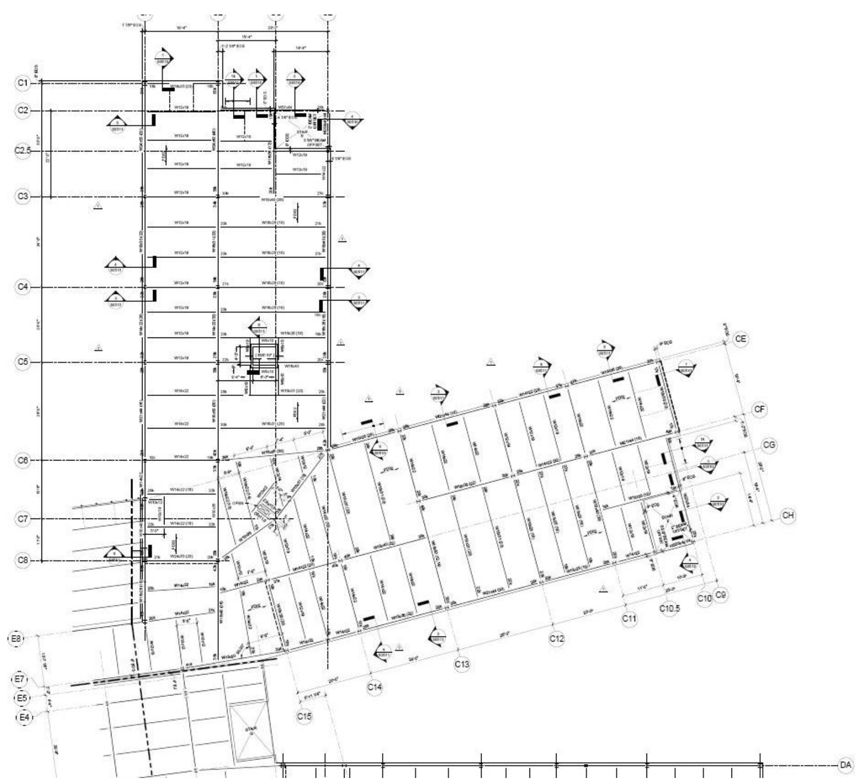


Figure 11: Typical Structural Framing Plan of Area C (right). Detail courtesy of Cannon Design.

Scope of Work

Problem Statement

After completing the analysis of the gravity and lateral force resisting systems, it is quite apparent that the existing structural system designed for the ECMC Skilled Nursing Facility is the most efficient and economical choice for design. In previous reports, it was found that the structural system met all strength and serviceability requirements and was the most economical solution for design in this area. Because of the building's symmetric radial geometry and its layout of braced frames, the design was effective in resisting torsional effects and could accommodate for lateral loading from all directions. Additionally, the gravity system, consisting of composite steel framing and decking, were sufficiently designed to support the buildings dead and live loads.

Since the ECMC Skilled Nursing Facility holds few structural flaws and challenges to redesign, it was assumed that an identical building, composed of the same composite steel structure and concentrically braced frames, was being designed for a location in downtown Los Angeles, CA. A building in this area would often be subject to high seismic activity and experience large seismic base shears and moments. Foundations and site soils would need to be considered and checked for possible soil liquefaction, as well as adequate soil bearing strength. The gravity system would also need to be reviewed to assure that it can carry the loads in this new location.

Not only should the structural system be considered in this new location, so should the mechanical system. In this new location, the climate is considered to be semi-arid, meaning the building will be subjected to higher temperatures than at its original location in Buffalo, NY. The mechanical AHU's need to be checked for their adequacy in this warmer location, otherwise they will need to be resized to meet standard requirements and comfort levels.

Additionally, with changes in design come impacts on the project cost and schedule. The changes made on the existing structural foundation system, lateral system, and gravity system, along with specification modifications for the existing mechanical system will need to be checked regarding cost for installation and materials. If new systems are added, they must also be added into the timeline found within the project schedule.

Proposed Solution

In this proposed solution, the ECMC Skilled Nursing Facility's existing structural system and foundations will be re-designed to meet code requirements in this new location. The specific systems that are a target for re-design will include the building's soil and foundation system, floor system, and lateral system.

Soils found in Los Angeles, CA will be classified and checked for adequate strength or other possible failure modes such as soil liquefaction. Additionally, the existing foundation system will be analyzed for its adequacy in this new location. Also, some research has been done on the use of lead rubber base isolators between the foundation and the structural framework and will be specified in the new foundation design as well. The use of these base isolators will prove essential in reducing forces and damage caused by earthquakes.

A floor system with the least amount of mass and weight would benefit greatly in a high seismic zone and would be chosen for re-design at this location since it will help reduce the story shear forces produced during an earthquake. With this in mind, a composite steel deck and frame floor system was chosen for design. With its ease of constructability and lightweight frame, it seemed to be the best choice for re-design in this location. It was concluded that the use of pendulums and large mass dampers would be inadequate for the new structure since it is only 5 stories high. These types of dampers are more useful in high-rise structures and skyscrapers in seismic areas.

Because of the efficiency and economic benefits of a concentrically braced lateral system found in the analysis of the existing structure, it will be used for redesign of the lateral system at this new location. The lateral system's new design will focus toward resisting frequent seismic events in this new location.

Upon changing the location of the ECMC Skilled Nursing Facility, the thermal impact on the building will change greatly since the climate is significantly different. At its current location in Buffalo, NY, the building experiences lake effect snow in the winter months and a moderate climate in the summer. Los Angeles, CA rarely experiences any snowfall and its average temperatures are significantly higher than in Buffalo, NY throughout the year. Considering these effects, the existing mechanical system will be evaluated and checked for adequacy. If the existing system is proven inadequate, a change in the specifications for the mechanical system will be made to meet industry standards. Additionally, a cost and schedule analysis will be made to compensate for any changes made to either the structural system or the mechanical system.

Project Goals

The overall design goal of this project is to redesign a concentrically braced frame lateral system that can withstand the increased seismic forces produced at this new location, as well as reduce the total building weight by optimizing the floor and framing system. Additional goals to be met throughout this course of study include:

- Minimize architectural changes in plan or elevation.
- Design most economical column and beam sizes where applicable
- Determine any affects due to structural changes
- Maintain floor-to-floor height
- Determine impacts of structural or mechanical changes on project cost and schedule
- Verify/specify efficient mechanical system in new location
- Suggest possibilities of soil liquefaction
- Use ETABS as a modeling tool to calculate building period and center of mass

Gravity and Lateral Loads

Dead and Live Loads

Before any gravity system members can be redesigned, the gravity loads must be reanalyzed using ASCE 7-10 as reference. Some changes from the original location in load calculation are the live load was increased from 40 psf to 80 psf to match the live load in the resident hallways. Additionally, the metal decking was changed from 2VLI to 3VLI decking to attain more strength and reduce depth of slab. Table 2 below shows a summary of the design loads used in the redesign of the gravity system. Refer to Appendix C for a detailed list of design loads.

Table 2: Design Load Summary				
Dead Loads (DL)				
Description	Location	NYC-BC 2007	ASCE 7-10	Redesign
Roof Deck 1	Roof	2 psf	2 psf	2 psf
Roof Deck 2	Penthouse Roof	3 psf	2 psf	2 psf
Floor Deck 1	Penthouse Floor	2 psf	2 psf	2 psf
Floor Deck 2	Floors 1-4	2 psf	2 psf	2 psf
Floor Finishings	Floors 1-4	2 psf	2 psf	2 psf
Roofing & Insul.	Roof + Penthouse Roof	8 psf	8 psf	8 psf
Leveling Concrete	Floors 1-4	5 psf	5 psf	5 psf
Ceilings	Floors 1-4 + Penthouse	5 psf	5 psf	5 psf
Typical Susp. MEP	Floors G-4	5 psf	5 psf	5 psf
Penthouse MEP	Penthouse	8 psf	8 psf	8 psf
Partitions	Floors 1-4	18 psf	18 psf	18 psf
Pavers, Potted Plants	Floors 1-4	80 psf	-	-
Green Wall (4"thick)	Floors 1-4	20 psf	-	-
Live Loads (LL)				
Description	Location	NYC-BC 2007	ASCE 7-10	Redesign
Resident Rooms	Floors G-4	40 psf	40 psf	80 psf
Ground Floor Corridors	Floor G	80 psf	100 psf	100 psf
Balconies	Floors 1-4	Not Specified	100 psf	100 psf
Resident Corridors	Floors 1-4	80 psf	80 psf	80 psf
Penthouse Floor	Penthouse	150 psf	150 psf	150 psf
Public Spaces/Exit Corridors/Stairs/Lobbies	Floors G-Penthouse	100 psf	100 psf	100 psf
* Live load reductions where applicable				

Wind Loads

The wind loads were calculated for this new location, and were determined using ASCE 7-10. The Main Wind Force Resisting System directional procedure was used to calculate wind pressures and loads. Due to the radial footprint and complex geometry that each wing created, along with the slanted and staggered roof design, the building was assumed to have a 344' x 344' square plan with a flat roof for simplification. Since the footprint is symmetric and square, wind pressures in both directions were similar, meaning either direction will see equal equivalent story forces produced by wind. The total base shear calculated was 1,071 kips, which is relatively similar to the base shear of 1,052 kips calculated for Buffalo, NY. Table 3 below lists wind design variables along with their appropriate ASCE 7-10 reference. Refer to Appendix C for detailed calculations.

Wind Variables			ASCE Reference
Basic Wind Speed	V	115mph	Fig. 26.5-1B
Directional Factor	K_d	0.85	Tab. 26.6-1
Occupancy Category		III	Tab. 1.5-1
Exposure Category		B	Sec. 26.7.3
Exposure Classification		Enclosed	Sec. 26.2
Building Natural Frequency	n_1	0.833 (flexible)	Eq. 26.9-4
Topographic Factor	K_{zt}	1	Fig. 26.8-1
Velocity Pressure Exposure Coefficient evaluated at Height Z	K_z	varies	Tab. 27.3-1
Velocity Pressure at Height Z	q_z	varies	Eq. 27.3-1
Velocity Pressure at Mean Roof Height	q_h	23.96	Eq. 27.3-1
Gust Effect Factor	G	0.859	Eq. 26.9.5
Product of Internal Pressure Coefficient and Gust Effect Factor	GC_{pi}	0.18 -0.18	Tab. 26.11-1
External Pressure Coefficient (Windward)	C_p	0.8	Fig. 27.4-1
External Pressure Coefficient (Leeward)	C_p	-0.5 (Symmetric, L/B = 1.0)	Fig. 27.4-1

Table 3: Wind Design Variables using ASCE 7 – 10 Directional Procedure.

Wind Loads								
Floor	Story Height (ft)	Height Above Ground (ft)	Controlling Wind Pressure (PSF)		Total Controlling Pressure (psf)	Force of Windward Pressure (K)	Story Shear Windward (K)	Moment Windward (ft-k)
			Windward	Leeward				
Pent. Roof	-	90	23.96	-16.84	40.8	140.4	0	12636
Pent. Floor	20	70	22.57	-16.84	39.41	235.3	140.4	16471
4th Floor	14	56	21.47	-16.84	38.31	182.8	375.7	10237
3rd Floor	13.3	42.67	20.26	-16.84	37.10	172.5	558.5	7360
2nd Floor	13.3	29.33	18.76	-16.84	35.60	166.3	731.0	4878
1st Floor	13.3	16	16.44	-16.84	33.28	173.2	897.3	2771
Ground Floor	16	0	0	0	0	0	1070.5	0
						Σ	**1070.5**	**54353**

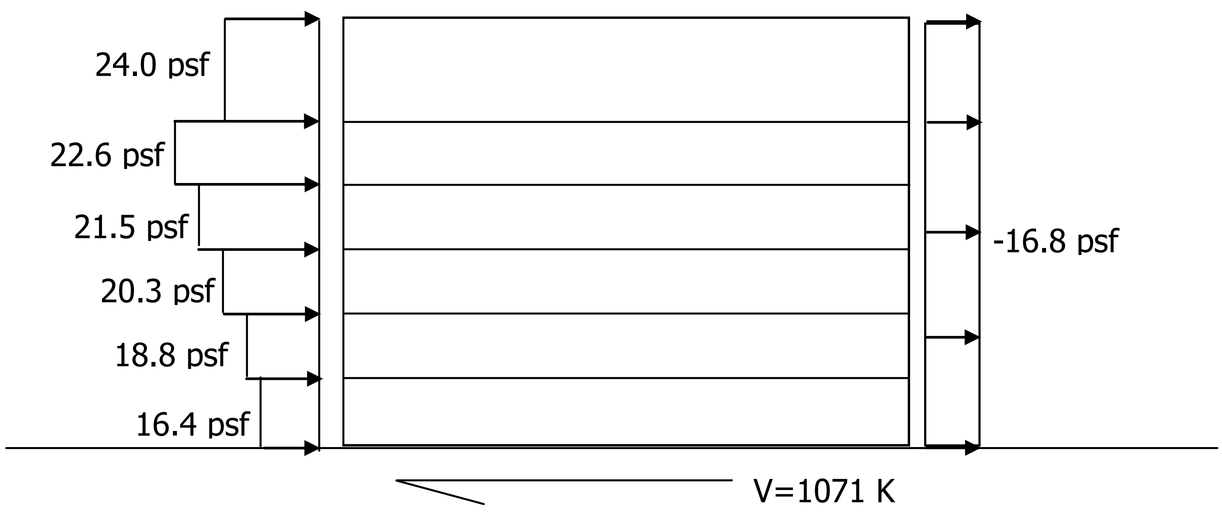


Table 4: The table above shows design wind pressures and forces for Los Angeles, CA, along with shear/moment forces on the building.

Figure 12: The figure above shows story design wind pressures applied to the windward and leeward side of the building, along with the total base shear.

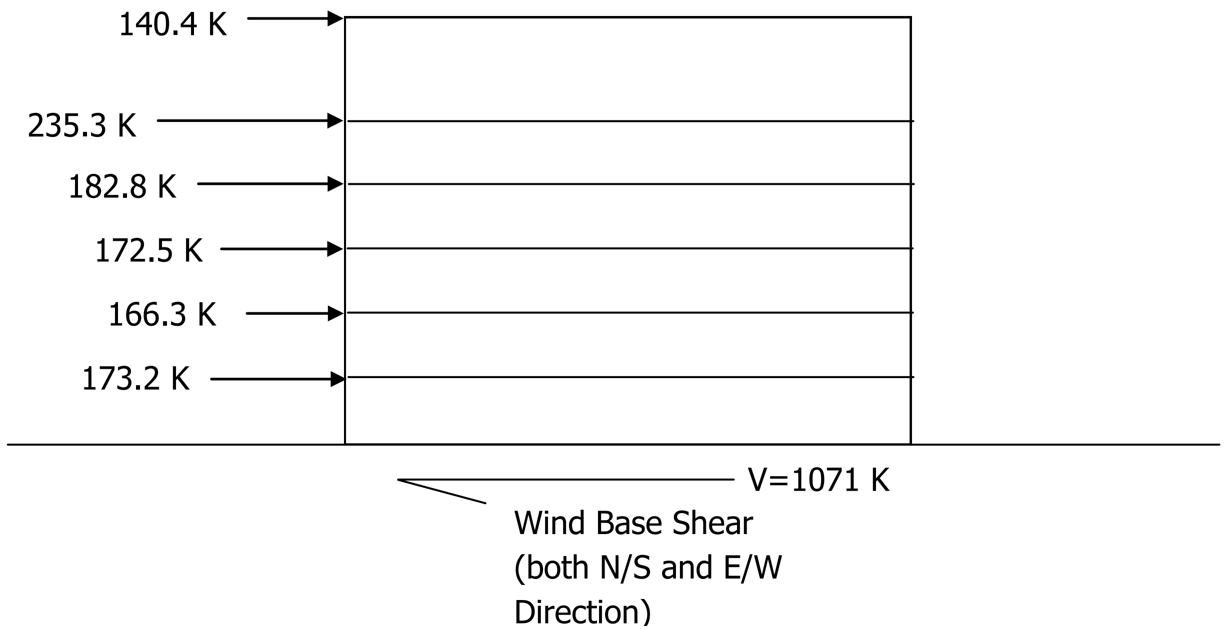


Figure 13: The figure above shows story shear forces caused by wind applied at each story, along with the total base shear.

Seismic Loads

The redesign of the lateral system used the ASCE 7-10 Equivalent Lateral Force Procedure found in Section 12.8 to determine the seismic loads produced in Los Angeles, CA. This procedure used dead loads from floor slabs, roof deck, MEP, and framing to calculate seismic shears. Seismic calculations were performed by hand, and approximate square footages were taken from construction documents. The total base shear at this new location from seismic loads was calculated to be 6,550.6 kips, which is roughly 14 times higher than the 455 kip base shear found in Buffalo, NY. Table 5 below shows seismic design variables used in the calculation. Refer to Appendix C for a detailed seismic calculation.

Seismic Design Variables		No Base Isolation		Base Isolated		ASCE Reference
Site Class		D		D		Sec. 20.3.2
Occupancy Category		III		III		Sec. C1.5.1
Importance Factor		1.25		1.25		Tab. 1.5-2
Structural System		Steel Special Concentrically Braced Frames		Steel Special Concentrically Braced Frames		Tab. 12.2-1
Spectral Response Acceleration, short	S_s	2.432		2.432		Fig. 22-1
Spectral Response Acceleration, 1 s	S_1	0.853		0.853		Fig. 22-2
Site Coefficient	F_a	1		1		Tab. 11.4-1
Site Coefficient	F_v	1.5		1.5		Tab. 11.4-2
MCE Spectral Response Accel., short	S_{ms}	2.432		2.432		Eq. 11.4-1
MCE Spectral Response Accel., 1 s	S_{m1}	1.279		1.279		Eq. 11.4-2
Design Spectral Acceleration, Short	S_{ds}	1.622		1.622		Eq. 11.4-3
Design Spectral Acceleration, 1 s	S_{d1}	0.853		0.853		Eq. 11.4-4
Seismic Design Category	S_{dc}	E		E		Sec. 11.6
Response Modification Coefficient	R	6.0		6.0		Tab. 12.2-1
Building Height (above grade) (ft)	h_n	90		90		
		N/S	E/W	N/S	E/W	
Approximate Period Parameter	C_t	0.02	0.02	0.02	0.02	Tab. 12.8-2
Approximate Period Parameter	x	0.75	0.75	0.75	0.75	Tab. 12.8-2
Calculated Period Upper Limit Coeff.	C_u	1.4	1.4	1.4	1.4	Tab. 12.8-1
Approximate Fundamental Period	T_a	0.584	0.584	0.584	0.584	Eq. 12.8-7
Fundamental Period	T	1.4081	1.4754	4.1803	4.1866	Sec. 12.8.2
Long Period Transition Period	T_L	8	8	8	8	Fig. 22-12
Seismic Response Coefficient	C_s	0.304	0.304	0.304	0.304	Eq. 12.8-2

Structural Period Exponent	k	1.042	1.042	1.042	1.042	Sec. 12.8.3

Table 5: Seismic Design Variables using ASCE 7-10 Equivalent Lateral Force Procedure.

Equivalent Lateral Force Procedure following Table 12.6-1							
Floor	Weight w_x (K)	Height h_x (ft)	$w_k h_x^k$ (K)	C_{vx}	Lateral Force F_x (K)	Story Shear V_x (K)	Moment M_x (ftK)
Penthouse Roof	904.9	90	98,383	0.089	583.0	583.0	52,470
Penthouse Floor	3,330.6	70	278,685	0.253	1,657.3	2,240.3	116,011
4th Floor	4,317.9	56	286,341	0.260	1,703.2	3,943.5	95,379
3rd Floor	4,297.4	42.67	241,663	0.194	1,270.8	5,214.3	54,221
2nd Floor	4,297.4	29.33	145,276	0.131	858.1	6,072.4	25,171
1st Floor	4,379.2	16	78,720	0.071	465.1	6,550.6	7,442
Ground	0	0	0	0	0	0	0
TOTAL	21,527		1,102,068	1		**6,550.6**	**350,694**

Table 6: The table above shows the Equivalent Lateral Force Procedure for Los Angeles, CA, along with the calculated story and base shears/moments.

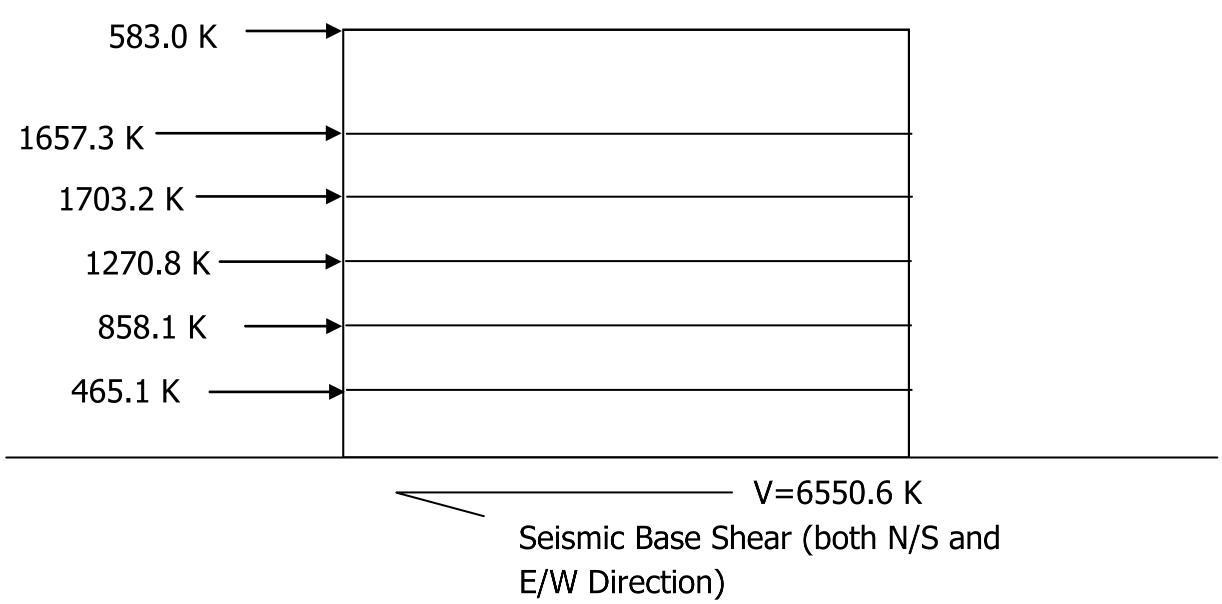


Figure 14: The figure above shows story shear forces due to seismic applied at each story, along with the total base shear.

Gravity System Redesign

In this section, the gravity system will be analyzed and redesigned for loads in this new location. Each bay is unique in size and shape, thus a column, beam, girder, and the floor decking will be redesigned and checked for strength and deflection.

In order to maintain the architectural floor plan layout of the structure, the redesign of the structural system followed the original framing plan. By maintaining this similar layout, the floor plan remained unchanged, however since an additional 40lbs of live load was added, some of the floor framing members increased slightly in depth to support the additional weight, which shouldn't pose as a problem since the floor to ceiling height allows for about a 4' space. Since the beams and girders carrying this extra load frame into their supporting columns, the columns increased in size as well, however they were sized at the same W10 depth as is found in the original plan to maintain wall and column thickness. Figure 15 below shows a 3D view of the redesigned framing layout.

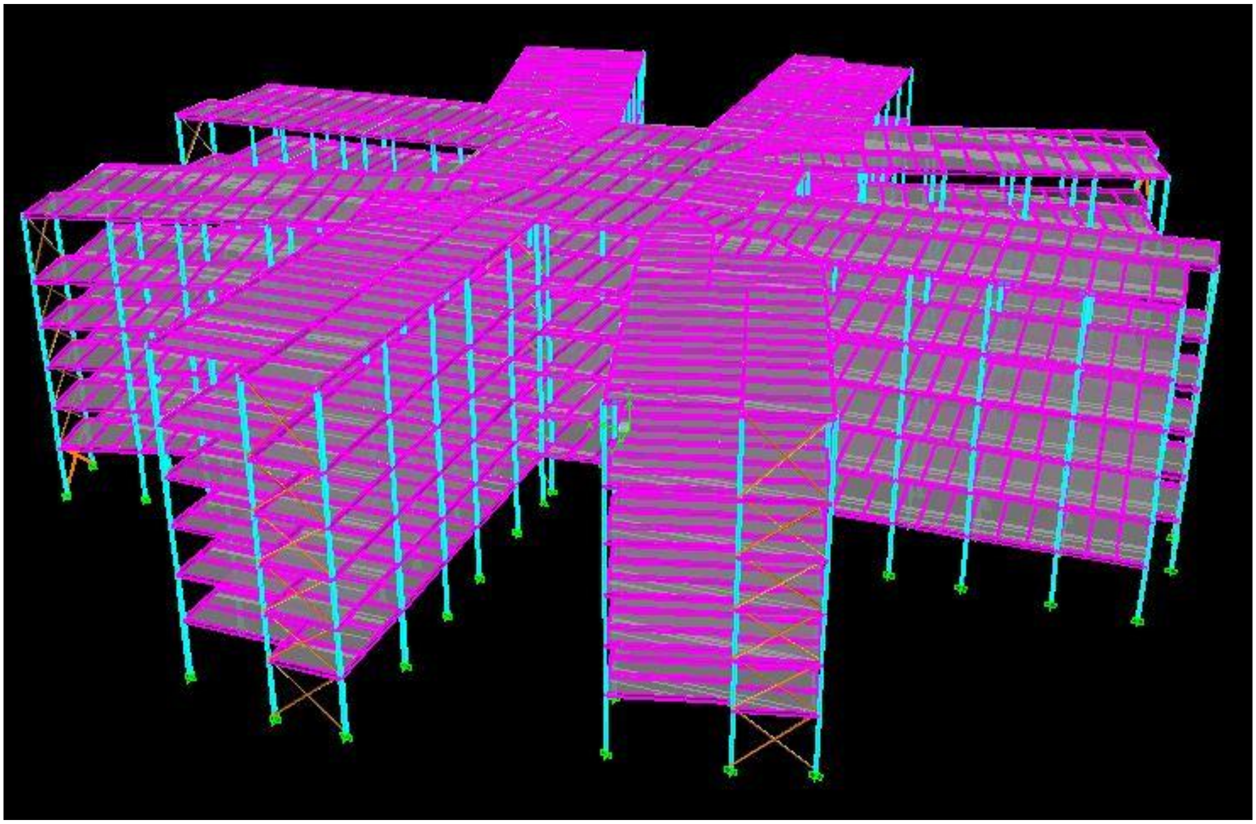


Figure 15: ETABS Model of Structural Steel Gravity System.

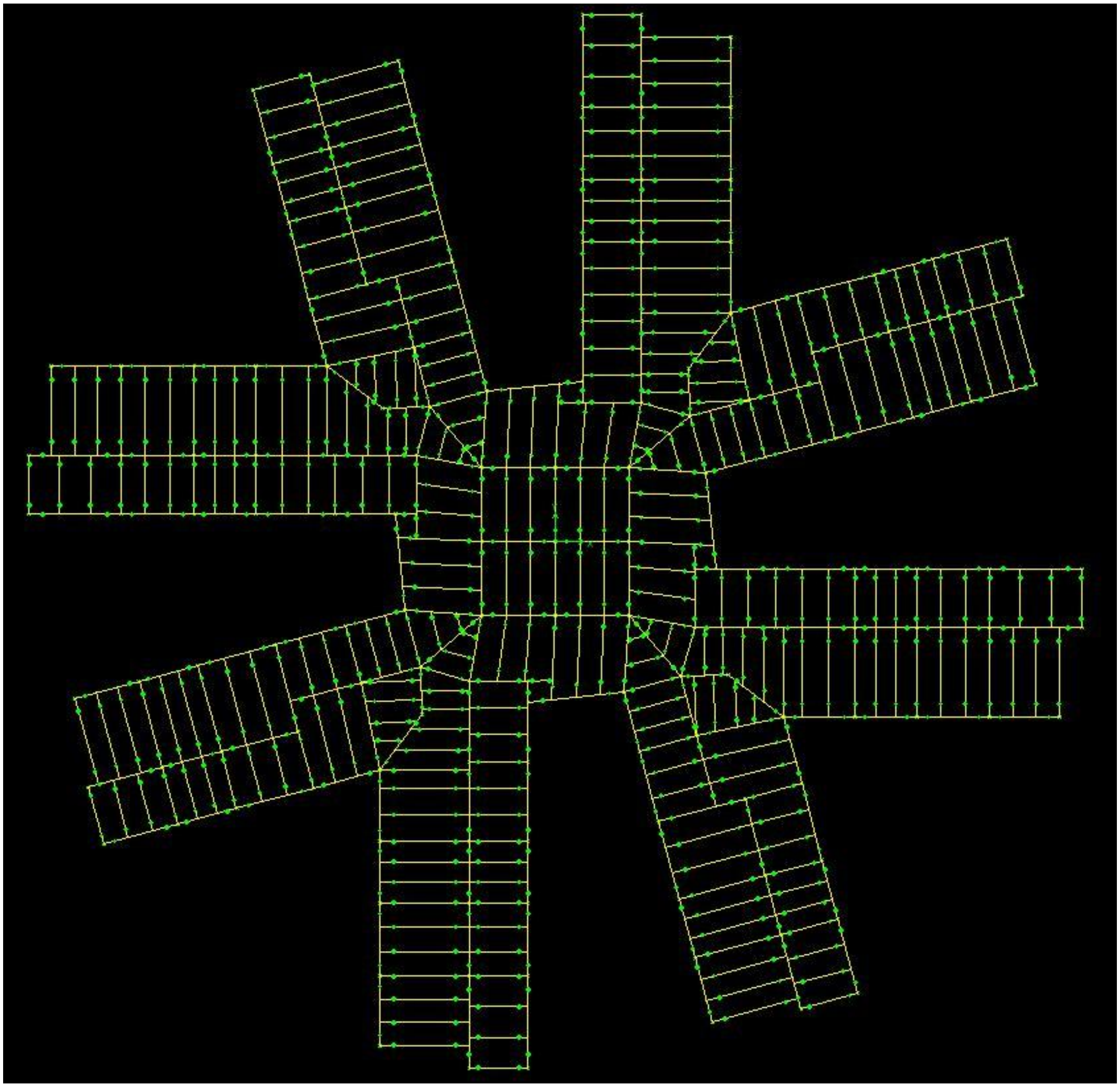
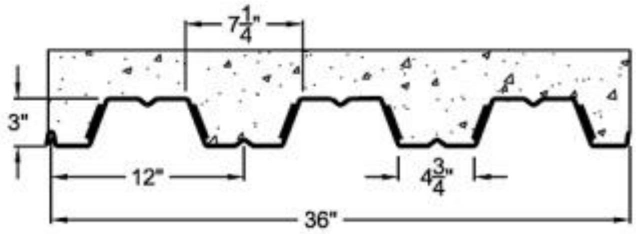


Figure 16: ETABS Model of Framing Plan Layout.

Composite Steel Decking

Since the building was being redesigned in a highly seismically active location, it was essential to try and reduce the weight of the gravity system to help minimize these increased earthquake load effects. The floor decking was redesigned as a 3VLI floor deck since it had a higher strength, allowing for a thinner floor deck which reduced the floor weight from 42psf to 35psf. The floor deck still maintained a 2-hour fire rating as did the original design. The topping was reduced from 3.25" to 2" as well. Figure 16 shows the redesigned ETABS framing layout.

Thus said, the new redesigned floor decking system will comprise of 3VLI20 metal decking with a 2” topping and total thickness of 5”. Figure 17 below shows the typical dimensions of this specified decking.



3" Composite Floor Deck

Figure 17: Dimensions and Specifications of Vulcraft 3VLI Decking.

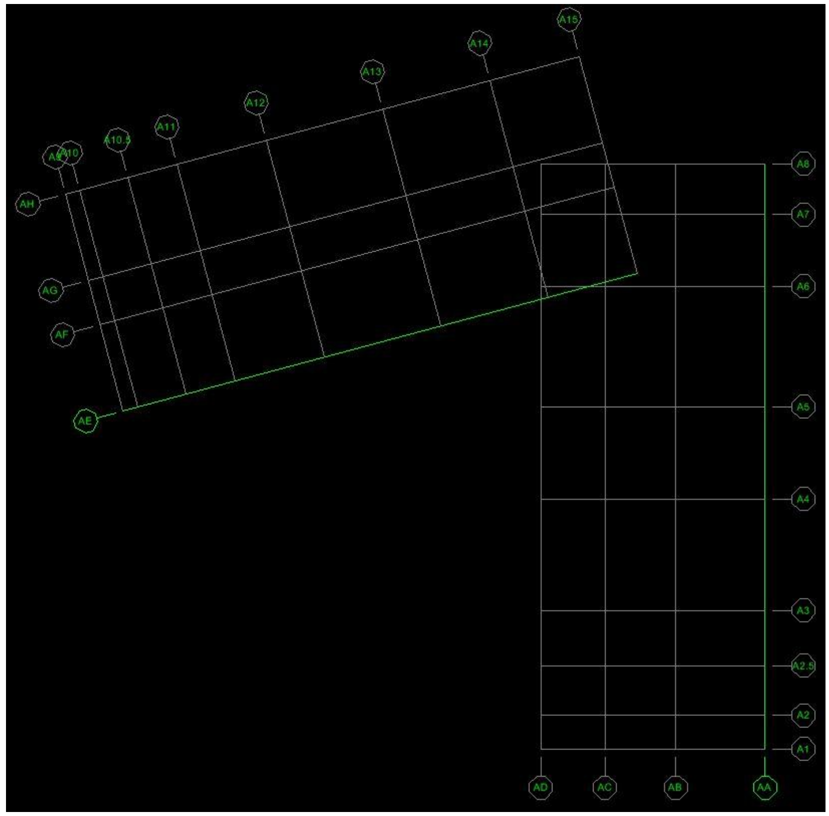
Typical Beam and Girder Design

After confirming that the new redesign of the composite floor system is adequate, the steel beams and girders needed to be redesigned to accommodate the higher live load and reduction in floor weight. All beams and girders were redesigned in accordance with Load and Resistance Factor Design (LRFD) methods and the 13^{th} Edition AISC Steel Construction Manual. This method applies a load factor to the design loads such that the design strength of the members exceeds the factored loads.

The gravity system was redesigned using ETABS finite element analysis software and was checked using hand calculations in critical areas. It was found that the typical beam consisted of a W14x26 utilizing 16 shear studs to create a composite structural system. When compared to the original system, this beam is slightly heavier and deeper than the original design. These W14x26 beams then framed into a W18x35 girder designed with 20 shear studs. This girder is also slightly deeper and heavier than the original design; however the difference is very minimal. Deflections were checked for both the beams and girders, and it was found that they passed for both live and total load deflection of L/360 and L/240 respectively.

Column Design

Due to the larger live load on the building, it was expected that the size of the column would increase. The original columns all shared the same W10 depth and in order to keep consistency and minimize architectural changes, the same W10 size was considered during column redesign. Figure 18 shows the grid layout for Area A. Due to symmetry, the same framing layout was used for Areas B, C, and D. Upon completion of the redesign, it was found that gravity columns ranged in size from W10x33 to W10x60.



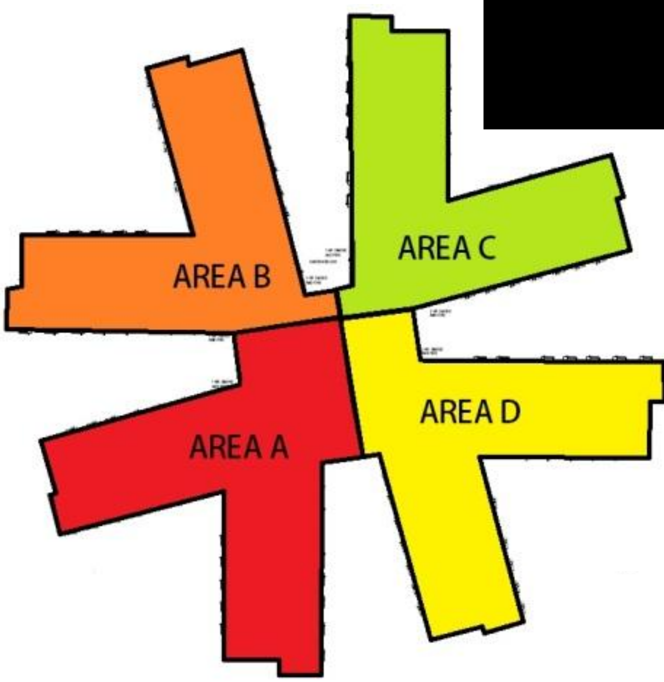


Figure 18: Grid framing layout for areas A, B, C, and D (above). Building area layout (left).

Lateral System Redesign

When designing a new building for downtown Los Angeles, CA, one must carefully design the building's lateral system such that it can withstand the large magnitude earthquakes produced by the multiple faults within the area. Often, buildings in southern California utilize some type of dampening or isolation system to increase the building's natural period. By increasing the building's period, maximum building deflections are reduced and damage becomes minimal.

Figure 19: Typical round base isolator under lateral deformation (above-left). Cross section of a lead-core rubber base isolator (above-right). Images courtesy of AGOM Metal Rubber Engineering (http://www.agom.it/)

In this lateral redesign, a comparison will be made between base isolated structures and non-isolated structures, both which are designed to resist the massive lateral shears produced in downtown Los Angeles, CA. The comparison will be based on member sizes, building periods, and deflections. Specifically, lead rubber base isolators (LRBs) are intended to be used in the structure. LRBs are comprised mainly of steel plates sandwiched between layers of natural rubber. It also incorporates the use of a lead core, which acts as a damper and also conforms back to its original shape over long periods of time. Figure 19 above shows a typical round LRB as it deforms under lateral forces. An ETABS model was used to help model the structures, as well as collect valuable data.

Load Combinations

Various load combinations were used in the analysis of the lateral system for this report. The following list shows these load combinations according to ASCE 7-10 for factored loads using strength design and from the IBC-2006 edition.

1. 1.4D
2. 1.2D + 1.6L + 0.5Lr
3. 1.2D + 1.6Lr + 0.5W
4. 1.2D + 1.0W + 1.0L + 0.5Lr
5. 1.2D + 1.0E + 1.0L
6. 0.9D + 1.0W
7. 0.9D + 1.0E

It was found that seismic controlled the design of the lateral system, primarily from the large increase in loads due to the highly seismic location. In this case, load cases 5 and 7 governed due to seismic and were used in the ETABS model to show the worst case scenarios on the lateral system. Load case 5 was used for strength and deflection checks while case 7 was considered for any uplift effects. Direction of load was irrelevant due to the buildings symmetric floor plan layout.

Seismic Comparison

Tables 7 and 8 show the weight comparison between the original design and the existing design, as well as a base isolation comparison. By minimizing the weight of the structure, the new design would essentially reduce the base shear produced by earthquakes in the Los Angeles region by about 17%. Additionally, using base isolation increased the original building period by 2.705 seconds.

Seismic Weight Comparison (Los Angeles, CA)		
	Existing Building Design	New Building Design
Building Weight	26,045 kips	21,527 kips
Base Shear	7918 kips	6550 kips
Total Moment	423,898 ft-k	350,694 ft-k

Table 7: Seismic Weight Comparison.

Seismic Base Isolation Comparison (Los Angeles, CA)		
	No Base Isolation	Base Isolation
Building Period	1.4754 sec	4.1803 sec
Base Shear	6550 kips	6550 kips
Total Moment	350,694 ft-k	350,694 ft-k
Displacement (@ 90')	2.971"	2.64"
Drift (@ 90')	0.025"	0.018"
Member Size	W14x370	W14x233

Table 8: Seismic Base Isolation Comparison.

Concentrically Braced Frame Design

The original design for the ECMC Skilled Nursing Facility's lateral system consisted of steel frame members and normal concentrically braced frames. In previous technical reports, it was determined that the existing lateral braced frame layout provides great lateral resistance from all directions and also provides adequate torsional stiffness due to its radially symmetric design. In the redesign, the same lateral system layout was chosen since it minimized the effect on the architectural layout of the floor plans as well as provided lateral and torsional stiffness and rigidity in all directions. However, since the building was moved to an area where the seismic site class changes from A to D, a special height requirement in the ASCE 7-10 guidelines states that the building must be equal to or below 60' in total height to use ordinary concentrically braced frames. With that in mind, it was assumed that the concentrically braced frames in this new location would be considered special and would need additional connection detailing to attain an R-value equal to 6. Figure 20 at left shows the typical braced frame layout for the ECMC Skilled Nursing Facility.

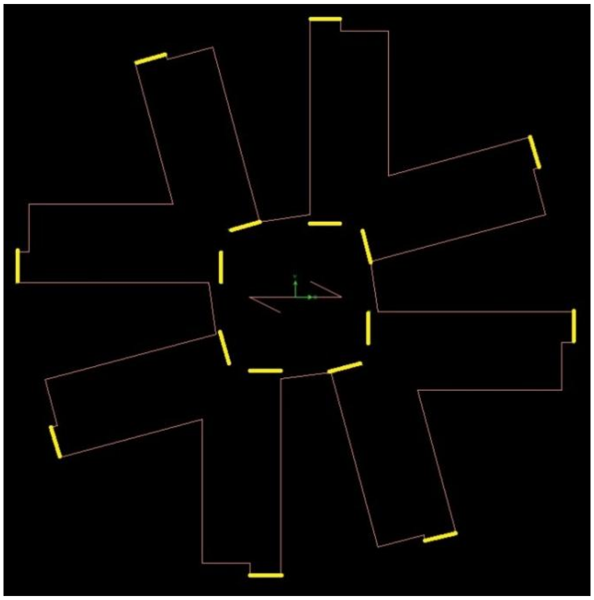


Figure 20: Special Concentrically Braced Frame Layout.

Load Path and Distribution

In this report, each floor system was modeled in ETABS as a rigid diaphragm. This allows story shears produced by wind or seismic to transfer through the floor slab directly into the concentrically braced frames. The loads transfer from the braced frames downward into the buildings foundation system. In order to calculate the relative stiffness for each braced frame, a 1000 kip horizontal load was applied to the top of the frame, and then finding the displacement associated with that force. Using the relative stiffness, further calculations determined the total load capacity for each braced frame.

In order to find an accurate center of mass and center of rigidity for the ECMC Skilled Nursing Facility, a finite elements computer model was generated using ETABS. Only the concentrically braced frames were modeled, since these are the main elements in the building that resist lateral loads. Each floor system was created as a rigid diaphragm, with an added area mass to account for the floor dead loads. Line elements were used to model the columns, beams, and cross bracing. The beams and columns consist of W-Flange steel shapes and the cross bracing is comprised of square steel HSS tubing. The model was created using 8 local grids, where 4 of those grids are rotated 15 degrees to match the angles of each wing. Figure 21 on the left shows the ETABS calculated center of rigidity. Tables 9 and 10 show the relative story stiffness for each frame at each story level.

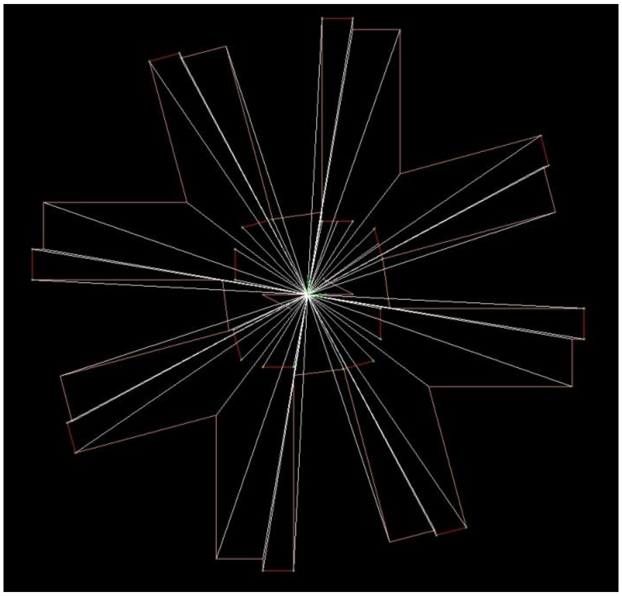


Figure 21: Center of rigidity of lateral load resisting system.

Relative Story Stiffness Ratio (R_{ix})										
P = 1000 kips										
X-Direction Displacement Δ_p (in)	level	A1	A8	B9	B15	C1	C8	D9	D15	
	PH_{RF}	4.127	4.173	-	-	-	-	-	-	
	PH	3.147	3.130	3.104	3.117	3.100	3.117	3.144	3.130	
	4	2.147	2.126	2.093	2.110	2.089	2.110	2.144	2.126	
	3	1.317	1.296	1.264	1.280	1.260	1.280	1.313	1.296	
	2	0.665	0.652	0.632	0.642	0.629	0.642	0.663	0.652	
	1	0.263	0.257	0.246	0.252	0.245	0.252	0.262	0.257	
Story Stiffness $K_{ix} = P/\Delta_p$ (kip/in)	level	A1	A8	B9	B15	C1	C8	D9	D15	ΣK_{ix}
	PH_{RF}	242.31	239.64	-	-	-	-	-	-	481.94
	PH	317.78	319.49	322.21	320.83	322.55	320.78	318.06	319.48	2561.18
	4	465.81	470.48	477.74	474.00	478.68	473.87	466.53	470.32	3777.42
	3	759.58	771.90	791.45	781.37	793.97	781.01	761.44	771.55	6212.27
	2	1504.35	1534.68	1583.03	1558.12	1589.57	1557.15	1508.98	1533.74	12369.62
	1	3796.52	3897.12	4060.09	3974.56	4081.63	3972.98	3812.43	3894.08	31489.42
									$\Sigma k_{ix,total}$:	56891.84
Relative Story Stiffness Ratio $R_{ix} = K_{ix}/K_{ix,total}$	level	A1	A8	B9	B15	C1	C8	D9	D15	
	PH_{RF}	0.5028	0.4972	-	-	-	-	-	-	
	PH	0.1241	0.1247	0.1258	0.1253	0.1259	0.1252	0.1242	0.1247	
	4	0.1233	0.1246	0.1265	0.1255	0.1267	0.1254	0.1235	0.1245	
	3	0.1223	0.1243	0.1274	0.1258	0.1278	0.1257	0.1226	0.1242	
	2	0.1216	0.1241	0.1280	0.1260	0.1285	0.1259	0.1220	0.1240	
	1	0.1206	0.1238	0.1289	0.1262	0.1296	0.1262	0.1211	0.1237	
Average		0.1224	0.1243	0.1273	0.1257	0.1277	0.1257	0.1227	0.1242	

Table 9: Relative Story Stiffness Ratios for frames in the X-direction.

Relative Story Stiffness Ratio (R_{iy})										
P = 1000 kips										
Y-Direction Displacement Δ_p (in)	level	A9	A15	B1	B8	C9	C15	D1	D8	
	PH_{RF}	-	-	-	-	-	-	4.172	4.125	
	PH	3.122	3.130	3.128	3.165	2.985	2.992	3.001	3.010	
	4	2.120	2.139	2.098	2.123	2.141	2.115	2.132	2.002	
	3	1.296	1.280	1.296	1.317	1.264	1.280	1.260	1.313	
	2	0.652	0.642	0.652	0.665	0.632	0.642	0.629	0.663	
	1	0.257	0.252	0.257	0.263	0.246	0.252	0.245	0.262	
Story Stiffness $K_{iy} = P/\Delta_p$ (kip/in)	level	A9	A15	B1	B8	C9	C15	D1	D8	Σkiy
	PH_{RF}	-	-	-	-	-	-	239.69	242.42	482.12
	PH	320.31	319.49	319.69	315.96	335.01	334.22	333.22	332.23	2610.13
	4	471.70	467.51	476.64	471.03	467.07	472.81	469.04	499.50	3795.31
	3	771.90	781.01	771.55	759.58	791.45	781.37	793.97	761.44	6212.27
	2	1534.68	1557.15	1533.74	1504.35	1583.03	1558.12	1589.57	1508.98	12369.62
	1	3897.12	3972.98	3894.08	3796.52	4060.09	3974.56	4081.63	3812.43	31489.42
									$\Sigma k_{iy,total}$:	56958.86
Relative Story Stiffness Ratio $R_{iy} = K_{iy}/K_{iy,total}$	level	A9	A15	B1	B8	C9	C15	D1	D8	
	PH_{RF}	-	-	-	-	-	-	0.4972	0.5028	
	PH	0.1227	0.1224	0.1225	0.1210	0.1283	0.1280	0.1277	0.1273	
	4	0.1243	0.1232	0.1256	0.1241	0.1231	0.1246	0.1236	0.1316	
	3	0.1243	0.1257	0.1242	0.1223	0.1274	0.1258	0.1278	0.1226	
	2	0.1241	0.1259	0.1240	0.1216	0.1280	0.1260	0.1285	0.1220	
	1	0.1238	0.1262	0.1237	0.1206	0.1289	0.1262	0.1296	0.1211	
Average		0.1238	0.1247	0.1240	0.1219	0.1271	0.1261	0.1274	0.1249	

Table 10: Relative Story Stiffness Ratios for frames in the Y-direction.

Drift Criteria

The allowable drift criteria according to the International Building Code 2006 edition were used to check deflection and drift for the redesigned lateral force resisting system. Below is a list of the deflection and drift criteria:

- Δ_{wind} = H/400 (Allowable Building Displacement)
- $\Delta_{seismic}$ = $0.02H_{sx}$ (Allowable Story Drift)

Controlling Seismic Drift (x-direction)			
Floor	Story Drift (in)	Allowable Story Drift (in)	Is this OK?
Roof	0.0184	0.400	yes
PH Floor	0.0152	0.280	yes
4th Floor	0.0168	0.267	yes
3rd Floor	0.0156	0.267	yes
2nd Floor	0.0123	0.267	yes
1st Floor	0.0073	0.320	yes

Table 11: Seismic Drift in the x direction.

Controlling Seismic Drift (y-direction)			
Floor	Story Drift (in)	Allowable Story Drift (in)	Is this OK?
Roof	0.0194	0.400	yes
PH Floor	0.0148	0.280	yes
4th Floor	0.0159	0.267	yes
3rd Floor	0.0145	0.267	yes
2nd Floor	0.0109	0.267	yes
1st Floor	0.0053	0.320	yes

Table 12: Seismic Drift in the y direction.

Controlling Wind Displacement (x-direction)				
Floor	Height above Ground (ft)	Displacement (in)	Allowable Displacement (in)	Is this OK?
Roof	90	2.489	2.700	yes
PH Floor	70	1.661	2.100	yes
4th Floor	56	1.265	1.680	yes
3rd Floor	42.667	0.874	1.280	yes
2nd Floor	29.333	0.517	0.880	yes
1st Floor	16	0.230	0.480	yes

Table 13: Wind Displacement in the x direction.

Controlling Wind Displacement (x-direction)				
Floor	Height above Ground (ft)	Displacement (in)	Allowable Displacement (in)	Is this OK?
Roof	90	2.523	2.700	yes
PH Floor	70	1.519	2.100	yes
4th Floor	56	1.127	1.680	yes
3rd Floor	42.667	0.751	1.280	yes
2nd Floor	29.333	0.413	0.880	yes
1st Floor	16	0.153	0.480	yes

Table 14: Wind Displacement in the y direction.

Torsional Effects

The ECMC Skilled Nursing Facility will see some slight torsional effects due to torsion, however nothing overly significant. Because of the buildings radial geometry in plan along with the circular layout of each braced frame, the buildings center of mass is relatively in the same location as the buildings center of rigidity. The ETABS model was used to obtain both the center of mass and rigidity for each floor. ETABS applies an eccentricity of 5% of the building length when checking seismic torsional effects, which accounts for accidental torsion that occurs in the building. Technical Report 3 shows that torsion on the building plan should not pose as a problem.

Foundation Redesign

Since the ECMC Skilled Nursing Facility is being relocated to an arbitrary location in downtown Los Angeles, CA, it was almost impossible to attain a geotechnical report for the area of interest. However, after further research, some geotechnical reports were found from the surrounding areas, such as Hollywood, CA and Vernon, CA.

Soil Properties and Liquefaction

After further review, it was found that the main type of soil is medium dense to loose sand layers and that limestone bedrock is located roughly at a depth of 80'. Soil bearing capacities from multiple reports ranged from 2,000 to 5,000 psi. There is a possibility of liquefaction in some geotechnical reports and others state that there is no risk of liquefaction.

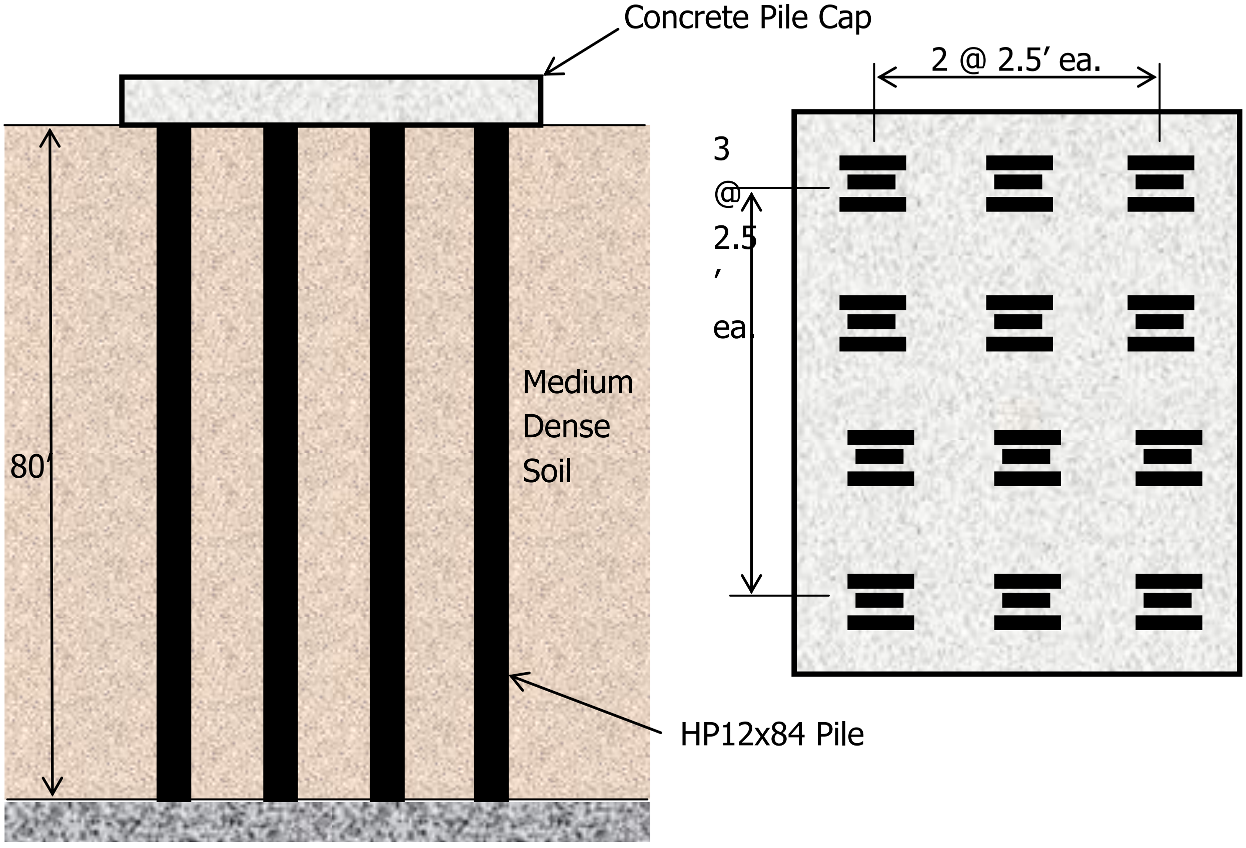
Figure 22: Building collapse due to liquefaction of soil sediments. Image courtesy of Wikispaces.com (http://earthscienceinmaine.wikispaces.com/7.4+Staying+Safe+in+Earthquakes)

Liquefaction is where saturated and unconsolidated soils act similar to quicksand or liquid when under the effects of an earthquake. Structures built over areas where liquefaction occurs tend to sink into the soil, as shown above in Figure 22. Although there is a possible risk of liquefaction in the area, this factor is reasonably site specific and in this proposed redesign it will be assumed that there is no risk of liquefaction on site. Since the vertical and horizontal forces caused by earthquakes induced on the foundation by the columns is much larger than what the bearing capacity can withstand, and with bedrock at such a large depth, the ECMC Skilled Nursing Facility will utilize deep foundations for redesign.

Deep Foundation Design

The deep foundation will consist of a group of HP shaped piles with a pile cap at the surface to support the base of the column. The piles will be designed for a length of 80' and will be installed using a Bodine Resonant Pile Driver, specifically an ICE Model 14C Hydraulic Vibratory Driver. The redesign of the foundation followed the following assumptions:

- OCR = 2.0
- V_p = 0.005 ft/sec
- Soil consists of mainly medium dense sand
- Piles will bear on limestone bedrock at a depth of 80'
- F.S. = 3.5



After further calculation, it was determined that each lateral system foundation will need a group of 12 piles consisting of HP12x84 shapes to reach adequate bearing capacity.

Breadth 1: Mechanical Study

When relocating a building design to a new location, one must not only consider the effects on the building structural system but must also consider the impact it has on the HVAC systems as well. The existing mechanical system was designed for a location in the heart of Buffalo, NY, where the building is subject to relatively hot summers and bitter cold winters. The new location in Los Angeles, CA hosts a very different, semi-arid climate. It is expected that heating loads will be reduced in this new location, however the cooling load may remain unchanged. Enthalpy calculations were performed to determine the significance of the existing Air Handling Units (AHU's) and checked to see if the systems could handle the different heating and cooling loads in this new location. Additionally, a thermal gradient comparison was determined on the exterior walls to check for any moisture issues as well as heat transfer through the materials to determine the wall's R-value. The existing system consists of a Variable Air Volume system, or VAV system, which adjusts the volume of supply air to meet heating and cooling needs. This adjustment in volume can greatly save on energy costs and can adapt to various conditions in temperature and moisture. The exterior wall consists of a brick cavity wall design, as shown in Figure 23 below.

Thermal Gradient Calculations

To ensure that the building can withstand the new temperature and moisture effects in the new location, a thermal gradient calculation was performed which checked for any condensation issues as well as determined the wall's existing R-value. The ASHRAE Fundamentals Handbook was used to determine R-values for the exterior brick wall system, as well as determine the summer and winter dry bulb temperatures for the two different locations. The indoor design temperatures for both summer and winter were assumed to be at 70 degrees Fahrenheit. Upon determining each materials R-value, the change in temperature was calculated using the following equation:

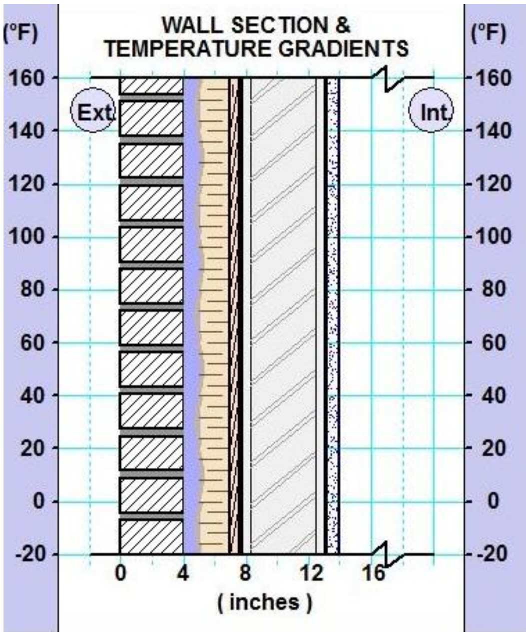
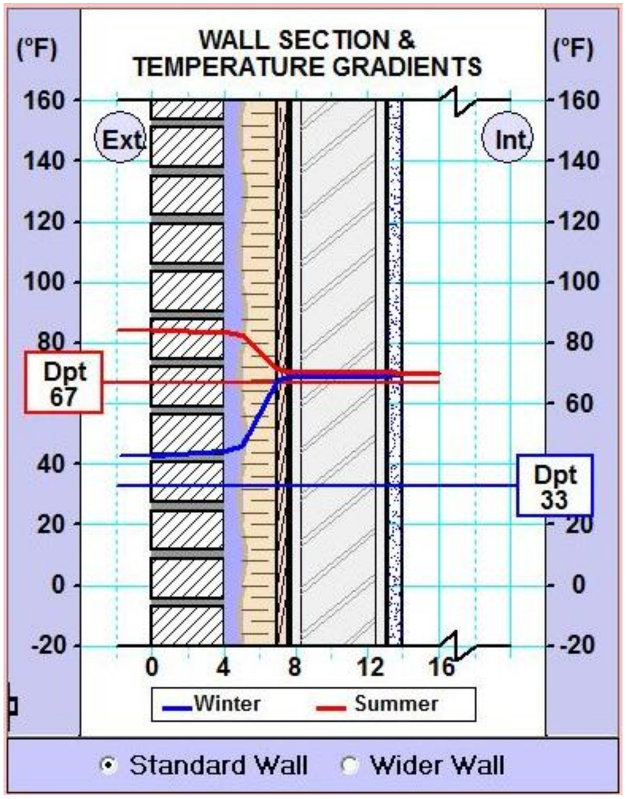


Figure 23: Typical Brick Cavity Wall.

- $T_x = T_{out} + (T_{in} - T_{out})(\Sigma R_{o-x}/\Sigma R_{o-i})$



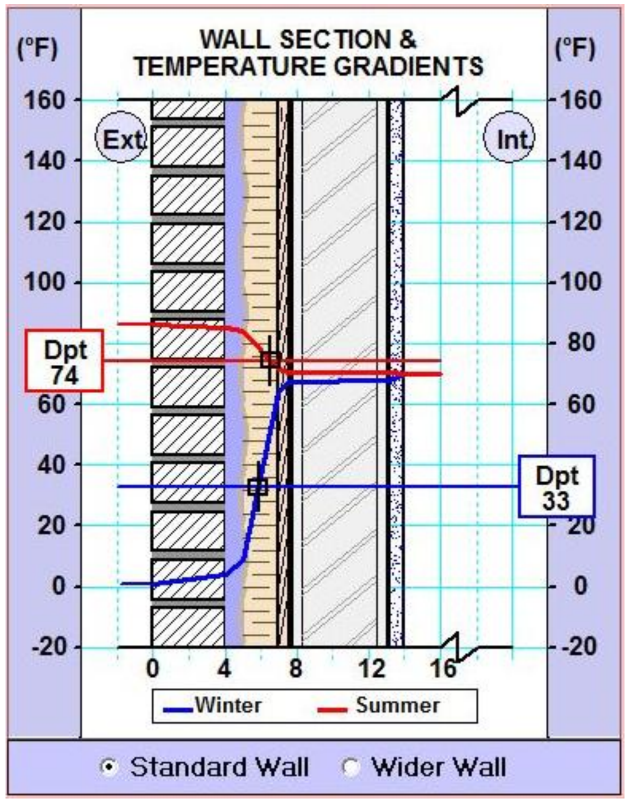


Figure 24: Typical Brick Cavity Wall.

Figure 25: Typical Brick Cavity Wall.

As shown in Figure 24, the temperatures transmitting through the wall do not reach the dewpoint in both the summer and winter months in the Los Angeles, CA location, meaning that there will be little to no condensation within the wall cavity. It was found that the R-value of the wall assembly is 15.35, which is relatively good for a wall system. In Figure 25, the building in the existing location does experience condensation, however, at the spray-on urethane insulation layer on the exterior of the plywood. This could possibly cause mold, rotting, or rusting of wall components; yet since the moisture barrier is between the insulation and plywood layers and if proper drainage is used, this issue can be avoided.

HVAC Verification

Since the building is subject to different temperatures in this new location, an enthalpy check was performed on the existing air handling units to verify if the existing HVAC systems were powerful enough to handle the differences in temperature. Tables 15 and 16, shown in a landscape view on the next page, show a sample enthalpy calculation as well as a total comparison and conclusion of HVAC performance for both locations.

Table 15: Heating calculation for Los Angeles, CA (above).

Table 16: Location system comparison of both locations.

Winter Heating Load (Full Capacity)									Location:	Los Angeles, CA
AHU-G-C	Mech. Room G033	Ground Floor Behavioral	1.10	13000	119.5	=	1708135	BTU/hr		
AHU-G-D	Mech. Room G154	Ground Floor Admin., PT/OT & Lobby	1.10	27000	119.5	=	3547665	BTU/hr		

IS THE MECHANICAL HVAC SYSTEM SUFFICIENT?														
			Summer Season					Winter Season					
			Buffalo, NY		Los Angeles, CA		Does it Pass?	Buffalo, NY		Los Angeles, CA		Does it Pass?
Unit #	Location	Service	Q_s		Q_s			Q_s		Q_s		
AHU-A	Mech. Penthouse	Area A Core	1321320	BTU/hr	1288980	BTU/hr	yes	6197730	BTU/hr	5518590	BTU/hr	yes
AHU-B	Mech. Penthouse	Area B Core	1321320	BTU/hr	1288980	BTU/hr	yes	6197730	BTU/hr	5518590	BTU/hr	yes
AHU-C	Mech. Penthouse	Area C Core	959530	BTU/hr	936045	BTU/hr	yes	4500733	BTU/hr	4007548	BTU/hr	yes
AHU-D	Mech. Penthouse	Area D Core	959530	BTU/hr	936045	BTU/hr	yes	4500733	BTU/hr	4007548	BTU/hr	yes
ERU-A	Mech. Penthouse	Area A Residents	289432	BTU/hr	282348	BTU/hr	yes	1357598	BTU/hr	1208834	BTU/hr	yes
ERU-B	Mech. Penthouse	Area B Residents	289432	BTU/hr	282348	BTU/hr	yes	1357598	BTU/hr	1208834	BTU/hr	yes
ERU-C	Mech. Penthouse	Area C Residents	289432	BTU/hr	282348	BTU/hr	yes	1357598	BTU/hr	1208834	BTU/hr	yes
ERU-D	Mech. Penthouse	Area D Residents	289432	BTU/hr	282348	BTU/hr	yes	1357598	BTU/hr	1208834	BTU/hr	yes
AHU-G-B	Mech. Room G033	Ground Floor Support Areas	849420	BTU/hr	828630	BTU/hr	yes	3984255	BTU/hr	3547665	BTU/hr	yes
AHU-KIT	Mech. Room G033	Kitchen	361790	BTU/hr	352935	BTU/hr	yes	1696998	BTU/hr	1511043	BTU/hr	yes
AHU-G-C	Mech. Room G033	Ground Floor Behavioral	408980	BTU/hr	398970	BTU/hr	yes	1918345	BTU/hr	1708135	BTU/hr	yes
AHU-G-D	Mech. Room G154	Ground Floor Admin., PT/OT & Lobby	849420	BTU/hr	828630	BTU/hr	yes	3984255	BTU/hr	3547665	BTU/hr	yes

Breadth #2: Construction Management Study

In addition to analyzing the structural and mechanical systems in this new location, the construction cost and schedule must also be analyzed to determine whether or not the new system changes are financially feasible for redesign.

Project Cost

As of any changes made to the structural system, lateral columns and HSS braces were redesigned and resized to meet structural strength and deflection requirements. Additionally, the foundation was changed beneath the lateral system to a deep foundation to help distribute the large lateral axial loads applied onto the foundation. Base isolators were also incorporated into the structure, which increased the total cost dramatically due to material costs. The mechanical system checked out and no changes were made to it. Unit costs were taken from the original estimate summary. Tables 17 and 18 display a summary of the lateral steel weight measured by the ton. Table 19 on the next page shows a cost comparison between the existing design and the redesign for the new location in Los Angeles, CA.

Wt. (lbs)	Length (ft)	# of Members			Total Wt. (tons)
		Gr. /1st	2nd/3rd	4th/PH	
W14x82	16	0	0	8	5.248
W14x90	30.6	0	14	0	19.278
W14x99	30.6	0	18	0	27.2646
W14x211	21.3	4	0	0	8.9886
W14x233	21.3	10	0	0	24.8145
W14x257	21.3	6	0	0	16.4223
W14x283	21.3	12	0	0	36.1674
				TOTAL	138.1834

Table 17: Weight of W-Flange Shapes.

Frame	HSS Steel Weights					
	Ground	1st Floor	2nd Floor	3rd Floor	4th Floor	Penthouse
A1	1110	705	625.1	545.2	554.48	547.66
A8	1110	705	625.1	545.2	554.48	547.66
A9	750	625.1	545.2	545.2	470.83	547.66
A15	750	625.1	545.2	545.2	470.83	547.66
B1	1110	705	625.1	545.2	554.48	547.66
B8	1110	705	625.1	545.2	554.48	547.66
B9	750	625.1	545.2	545.2	470.83	547.66
B15	750	625.1	545.2	545.2	470.83	547.66
C1	1110	705	625.1	545.2	554.48	547.66
C8	1110	705	625.1	545.2	554.48	547.66
C9	750	625.1	545.2	545.2	470.83	547.66
C15	750	625.1	545.2	545.2	470.83	547.66
D1	1110	705	625.1	545.2	554.48	547.66
D8	1110	705	625.1	545.2	554.48	547.66
D9	750	625.1	545.2	545.2	470.83	547.66
D15	750	625.1	545.2	545.2	470.83	547.66
SUM	14880	10640.8	9362.4	8723.2	8202.48	8762.56
					TOTAL (tons)	30.28572

Table 18: Weight of HSS tube shapes.

Component	Quantity	Labor		Material		TOTALS	
		Unit Cost	Amount	Unit Cost	Amount	Redesigned	Original Design
WF Lateral Steel Columns	138.183 TN	715.68/TN	196,784	2,074.64/TN	286,674	$385,567	$118,605
HSS Steel Bracing	30.3 TN	715.65/TN	21,684	2,074.64/TN	62,862	$84,726	$95,099.00
HP Steel Piles	30720 VLF	-	-	44.25/VLF	1,359,360	$1,359,360	-
Lead Rubber Base Isolators	207	-	-	20,000/LRB	4,140,000	$4,140,000	-
					TOTALS	$5,969,653	$213,704

Table 19: Cost analysis of redesign.

Project Schedule

Since there were virtually no changes done to the architectural layout or column and beam layouts, there weren't many changes to the project schedule. However, with the incorporation of base isolation, it was found that the installation of these isolators would increase the construction schedule by about two weeks. The construction project was mainly set back by the installation of the deep foundation piles. A normal crew could install roughly 590 vertical linear feet of HP piles per day, which led to an increase of 156 days to the construction schedule. It is possible to hire multiple crews such that this delay could be compensated for, however it would increase the project cost to hire multiple crews and equipment. Figure 26 below shows a portion of the schedule for the Area A redesign. The next page shows the task list for the schedule.

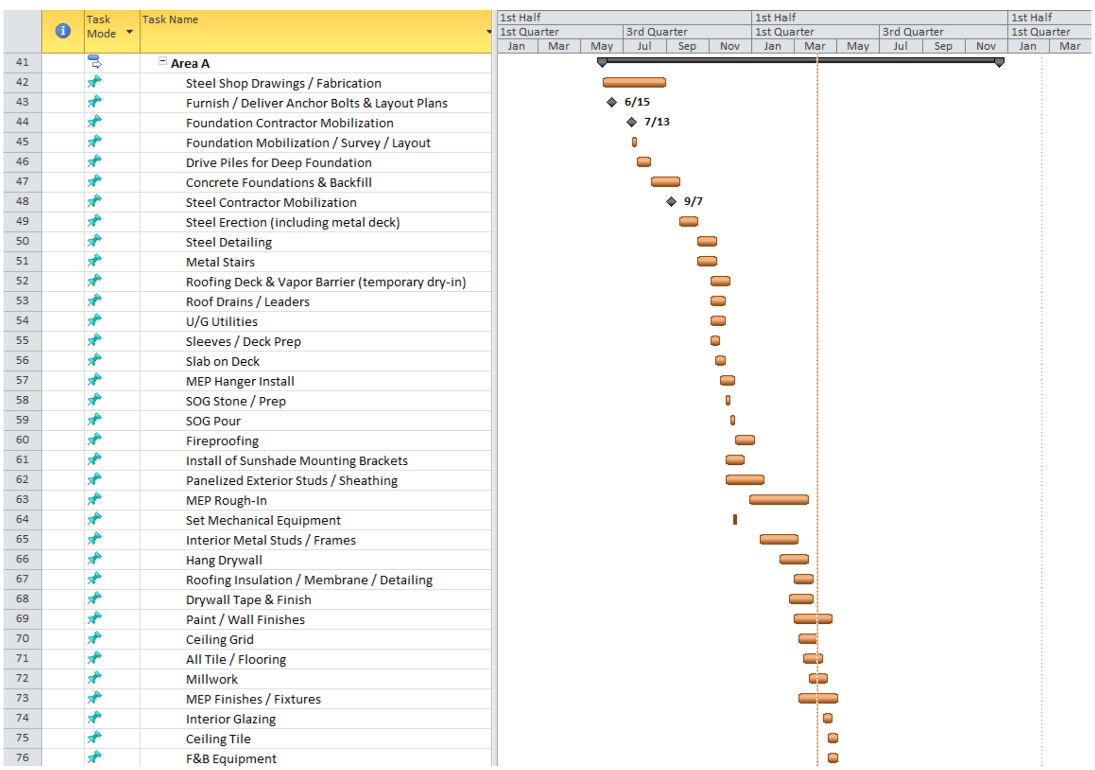


Figure 26: Sample of Project Schedule.

Task Name	Duration	Start	Finish
Area A	406 days	Wed 6/1/11	Thu 12/20/12
Steel Shop Drawings / Fabrication	65 days	Wed 6/1/11	Tue 8/30/11
Furnish / Deliver Anchor Bolts & Layout Plans	0 days	Wed 6/15/11	Wed 6/15/11
Foundation Contractor Mobilization	0 days	Wed 7/13/11	Wed 7/13/11
Foundation Mobilization / Survey / Layout	5 days	Wed 7/13/11	Tue 7/19/11
Drive Piles for Deep Foundation	14 days	Wed 7/20/11	Mon 8/8/11
Concrete Foundations & Backfill	30 days	Tue 8/9/11	Mon 9/19/11
Steel Contractor Mobilization	0 days	Wed 9/7/11	Wed 9/7/11
Install Base Isolators for each column in Area A	12 days	Mon 9/19/11	Fri 10/14/11
Steel Erection (including metal deck)	20 days	Fri 10/14/11	Thu 11/10/11
Steel Detailing	20 days	Fri 10/14/11	Thu 11/10/11
Metal Stairs	20 days	Fri 10/14/11	Thu 11/10/11
Roofing Deck & Vapor Barrier (temporary dry-in)	20 days	Wed 11/2/11	Tue 11/29/11
Roof Drains / Leaders	15 days	Wed 11/2/11	Tue 11/22/11
U/G Utilities	15 days	Wed 11/2/11	Tue 11/22/11
Sleeves / Deck Prep	10 days	Wed 11/2/11	Tue 11/15/11
Slab on Deck	10 days	Wed 11/9/11	Tue 11/22/11
MEP Hanger Install	15 days	Wed 11/16/11	Tue 12/6/11
SOG Stone / Prep	5 days	Wed 11/23/11	Tue 11/29/11
SOG Pour	5 days	Wed 11/30/11	Tue 12/6/11
Fireproofing	20 days	Wed 12/7/11	Tue 1/3/12
Install of Sunshade Mounting Brackets	20 days	Wed 11/23/11	Tue 12/20/11
Panelized Exterior Studs / Sheathing	40 days	Wed 11/23/11	Tue 1/17/12
MEP Rough-In	60 days	Wed 12/28/11	Tue 3/20/12
Set Mechanical Equipment	2 days	Tue 12/6/11	Wed 12/7/11
Interior Metal Studs / Frames	40 days	Wed 1/11/12	Tue 3/6/12
Hang Drywall	30 days	Wed 2/8/12	Tue 3/20/12
Roofing Insulation / Membrane / Detailing	20 days	Wed 2/29/12	Tue 3/27/12
Drywall Tape & Finish	25 days	Wed 2/22/12	Tue 3/27/12
Paint / Wall Finishes	40 days	Wed 2/29/12	Tue 4/24/12
Ceiling Grid	20 days	Wed 3/7/12	Tue 4/3/12
All Tile / Flooring	20 days	Wed 3/14/12	Tue 4/10/12
Millwork	20 days	Wed 3/21/12	Tue 4/17/12
MEP Finishes / Fixtures	40 days	Wed 3/7/12	Tue 5/1/12
Interior Glazing	10 days	Wed 4/11/12	Tue 4/24/12
Ceiling Tile	10 days	Wed 4/18/12	Tue 5/1/12
F&B Equipment	10 days	Wed 4/18/12	Tue 5/1/12
Interior Doors / Hardware	25 days	Wed 4/11/12	Tue 5/15/12
Specialties	10 days	Wed 5/2/12	Tue 5/15/12
Preliminary DOH / Building Walk-Thru's	5 days	Wed 5/16/12	Tue 5/22/12
Masonry Contractor Mobilization	0 days	Thu 5/10/12	Thu 5/10/12
Exterior Masonry	80 days	Thu 5/10/12	Wed 8/29/12
Windows / Exterior Glazing	25 days	Fri 8/17/12	Thu 9/20/12
Exterior Architectural Sunshades	20 days	Fri 9/7/12	Thu 10/4/12
Final Cleaning	10 days	Fri 9/14/12	Thu 9/27/12
Interior Punchlist Inspections	5 days	Fri 9/28/12	Thu 10/4/12
Interior Punchlist Work	20 days	Fri 10/5/12	Thu 11/1/12