Senior Thesis Final Report

Steel Redesign, Vibration Resistance, and Lateral System Redesign

April 9, 2014

La Jolla Commons Phase II Office Tower

San Diego, California

Alyssa Stangl | Structural Option | Advisor: Dr. Linda Hanagan

La Jolla Commons Phase II Office Tower

San Diego , California | LPL Financial Office Tower

Primary Project Team

Owner | Hines
Tenant | LPL Financial
Architect | AECOM
Structural Engineer | Nabih Youssef Associates
MEP Engineer | WSP Flack + Kurtz
Civil Engineer | Leppert Engineering

General Building Data

Construction Dates | April 2012 – May 2014
Building Cost | $78,000,000
Delivery Method | Design-Bid-Build
Height| 198' – 8" | 13 Stories
2 Levels | Underground Parking
Size | 462,301 GSF

Architecture

- Modern style building with glass curtain wall
- 12 foot floor-to-floor height
- Very open and spacious office area
- Interior features and build out by tenant

Sustainability Features

- First Class A, NetZero Office Building in the USA
- Building returns more energy to the grid than it uses on an annual basis
- LEED – CS Gold Certification

Structural

- Two-way, flat plate , reinforced concrete slab
- Concrete columns on a regular column grid
- Special reinforced concrete shear walls
- Mat foundation system

Mechanical

- Chilled Water, floor-by – floor VAV Dual Path Air Handling Units
- Ventilation and cooling through underfloor air distribution, overhead air to perimeter zones.

Lighting and Electrical

- High efficiency, low glare lighting fixtures
- High power factor electronic ballasts
- Lighting control system integrated with Building Management System, local override at each floor
- Two 400 Amp, 480/277V, 3-phase, 4 wire switchboards service building
- One services the lower level bus riser and the other services the upper level bus riser
- One diesel fuel standby engine generator.

NABIH YOUSSEF ASSOCIATES STRUCTURAL ENGINEERS
AECOM Hines
LPL Financial

Alyssa Stangl [Structural Option] http://www.engr.psu.edu/ae/thesis/portfolios/2014/ams6158

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Executive Summary

La Jolla Commons Phase II Office Tower is a 13 story office building in San Diego, California. Each level is about 40,320 square feet, and the structure reaches 198' -8" from ground level to the top of the penthouse. With two levels of underground parking, the building extends 20'-0" below grade. Serving as an office building for LPL Financial, the building has large open floor plans and large areas of glass curtain wall. La Jolla Commons Tower II received a LEED-CS Gold Certification and is one of the most advanced net-zero office buildings in the country.

The original building structure begins with a mat foundation, two levels below grade. The gravity system consists of two-way, flat plate, concrete slabs on a rectangular column grid. Camber was used for the slab at each level to control deflections. The building's lateral system consists of special reinforced concrete shear walls. Due to high shear forces associated with this Seismic Design Category D structure, collector beams are required to transfer lateral loads at levels below grade in the north-south direction.

The structural depth consists of two main parts. First, the building structure was redesigned in steel, using the original column locations. The deck configuration of 1.5VLR20 with 4.25" light-weight concrete topping was selected based on an initial vibrations control assessment. RAM Structural System was used to design composite beams and steel columns. The final steel design was then verified to meet the AISC *Design Guide 11* requirements for walking induced vibrations and was found to be adequate.

Second, the original lateral system had an extreme torsional irregularity under seismic loading. In an effort to control torsion, steel moment frames were added around the building perimeter, along with the existing core concrete shear walls. These moment frames were designed to meet the requirements of special moment frames in accordance with the AISC *Seismic Design Manual* and *Seismic Provisions*. In addition, the clean column design approach was taken. Column sizes were increased in size in order to eliminate the need for web plates, flange stiffeners, or continuity plates. Ultimately, the moment frames were able to control the torsional irregularity, so torsional amplification of seismic forces was not required.

Two breadth topics are also investigated in this report; one breadth is related to the building architecture and the other construction. The architecture breadth investigates the impact on the building height and the building fire protection as a result of changing from a concrete to a steel structure. The construction breadth compares the cost and schedule of the steel and concrete structural systems. The steel system is about 23% more expensive than the concrete system, and the steel schedule is only about 2 weeks less in duration than the concrete system.

After investigations were complete, it was found that although a steel system is feasible, it may not be the most effective design for La Jolla Commons Phase II Office Tower. The concrete system allows for higher floor-to-ceiling heights, lower costs without a significant schedule increase, and does not require fire-resistive materials. Also, the concrete system will inherently control vibrations. Thus, a concrete structure is probably the most efficient choice for La Jolla Commons Phase II Office Tower.

Chapter 1 – Building Introduction

1.1 – Architectural and Site Overview

La Jolla Commons Phase II Office Tower (LJC II), rendered in **Figure 1.1 – 1**, is a high-rise structure located in San Diego, California. This Seismic Category D structure reaches 198'-8" above grade with 462,301 square feet of floor space, including two underground parking levels. LJC II is a very modern style and open building, featuring flat plate reinforced slabs on a rectangular column grid. This creates a very spacious office area for the building tenant, LPL Financial. LJC II features 13 stories of office space, a penthouse, and two underground levels of parking.

Figure 1.1 - 1 | **South East Elevation** *(Hines & AECOM)*

La Jolla Commons Phase II Office Tower is very similar to its sister building, Tower I. Although identical in architectural style, Tower I has a steel structure unlike Tower II. **Figure 1.1 – 2** shows the two towers side by side, while Tower II is under construction. The two towers help to unite the La Jolla Commons Campus around a green space and pedestrian area. Eventually, the campus will feature two acres of park space, surrounding the existing and proposed buildings. The campus will also eventually include a restaurant, bar, spa, gym, and meeting spaces. A view of the site plan can be viewed in **Figure 1.1 – 3**.

Figure 1.1 - 2 | **South East Elevation** *(Hines & AECOM)*

LJC II is built underneath a flight path, controlled by the Federal Aviation Administration (FAA). After negotiations, the building's height was limited to its current height of 198'-8".

Figure 1.1 - 3 | **Building Site Plan** *(Hines)*

After LJC Tower I achieved a LEED-CS Gold rating in 2008, Tower II was expected to reach a prestigious level of sustainability as well. LJC II includes features such as reclaimed water reuse, under-floor air distribution, double pane glazing with low emissivity coating, and energy efficient lighting systems. Furthermore, LJC II is the first Class A Net-Zero office building in the United States, and it is the nation's largest carbon-neutral office building to date. Through methods of reduced consumption and onsite generation, LJC II will actually return more power to the grid than it will use annually. LJC II also received a LEED-CS Gold Certification upon structure and shell completion.

See **Appendix A** for a typical architectural floor plan and two building elevations.

1.2 – Structural Overview

Structural Framing Summary

La Jolla Commons Tower II is a, cast-in-place concrete structure using mild reinforcing. The foundation consists of a concrete mat, ranging in thickness from 3 feet to 6.5 feet. The gravity system consists of two-way, flat plate, reinforced concrete slabs supported by a rectangular grid of reinforced concrete columns. The lateral system is a series of shear walls located at the building's core. Also, due to high seismic loading (seismic category D), the lateral system includes collector beams on the Ground Level and Lower Level 1, which are used to transmit the earthquake loads from the diaphragm into the shear walls. The building also features two 15 foot cantilever sections at the North and South ends. The mechanical penthouse, located on the roof, is framed in steel wide-flanges and hollow structural steel members with a moment frame acting as the lateral system.

Building Materials

La Jolla Commons Phase II Office Tower, primarily a concrete structure, employs several concrete and reinforcing types, shown in **Table 1.2 – 1** and **Table 1.2 – 2**, depending on the use in the building. Although concrete is the main structural material, information regarding steel is provided in **Table 1.2 – 3** for the penthouse framing.

Table 1.2 – 1 | Concrete Strengths (at 28 days, 0.5 max cement ratio)

Slab on Grade	3500 PSI	Normal Weight
Foundations	5000 PSI	Normal Weight
Shear Walls	6000 or 7000 PSI (per plans)	Normal Weight
Slabs and Beams	5000 PSI	Normal Weight
Columns	6000 or 7000 PSI (per plans)	Normal Weight
Basement Retaining Walls	5000 PSI	Normal Weight
Cantilever Retaining Walls	5000 PSI	Normal Weight
Built-up Slabs	4000 PSI	Light Weight (110 PCF)
All Other Concrete	4000 PSI	Normal Weight

Table 1.2 – 2 | Steel Reinforcement

Typical Reinforcing Bars	ASTM A-615, Grade 60
Shear Wall and Diaphragm Reinforcing	ASTM A-706
Welded Rebar	ASTM A-706

Table 1.2 – 3 | Structural Steel

All Structural Steel	ASTM A-572, Grade 50 OR ASTM A992
Steel Braced Frame Beams and Columns	ASTM A992
Structural Tubing	ASTM A-500, Grade B (Fy = 46000 PSI)
Structural Piping	ASTM A-53, Grade B (Fy = 35,000 PSI)

Foundation

The foundation system design was provided by Nabih Youssef Associates, the structural consultant for LJC II, after review of the geotechnical report and recommendations of the geotechnical engineer, Christian Wheeler Engineering. The final design consisted of a reinforced concrete mat foundation.

Foundation Walls

As stated above in the Building Introduction, La Jolla Commons Tower II has two levels of underground parking. As a result, concrete foundation walls were utilized around the building perimeter to hold back soil loads. Typical foundation walls are 14" thick concrete with #7 bars at 12 inches on center (o.c.) at the exterior and #5 bars at 12 inches o.c. at the inside face, vertical reinforcement. Also, #6 bars at 12 inches o.c. were provided for horizontal reinforcement.

The southeast corner, the area requiring surcharge loading, has 16 inch foundation walls with #9 vertical bars at 12 inches o.c. (outside face) and #6 bars at 10 inches o.c. (inside face). Also, #6 horizontal bars were provided at 12inches o.c. The thicker walls are necessary due to increased soil pressures due to soil saturation.

Mat Foundation Design

The foundation for La Jolla Commons Phase II Office Tower was designed as a reinforced concrete mat foundation with varying thicknesses and reinforcement. Originally, a system of footings and grade beams were considered for the foundation. The mat foundation was chosen for several reasons. First, the large area it covers helps to reduce the soil pressure created by the overturning moment associated with seismic loads. Second, the construction of one large mat was simply easier than forming all of the footings and grade beams required for the alternative system. **Figure 1.2 – 1** shows the variation in mat thickness across the foundation.

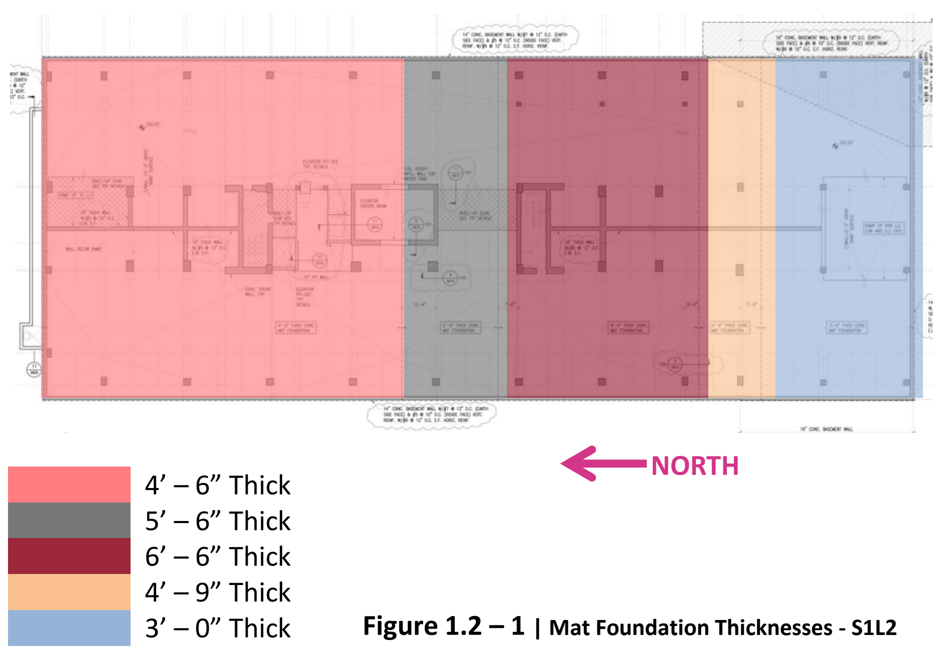
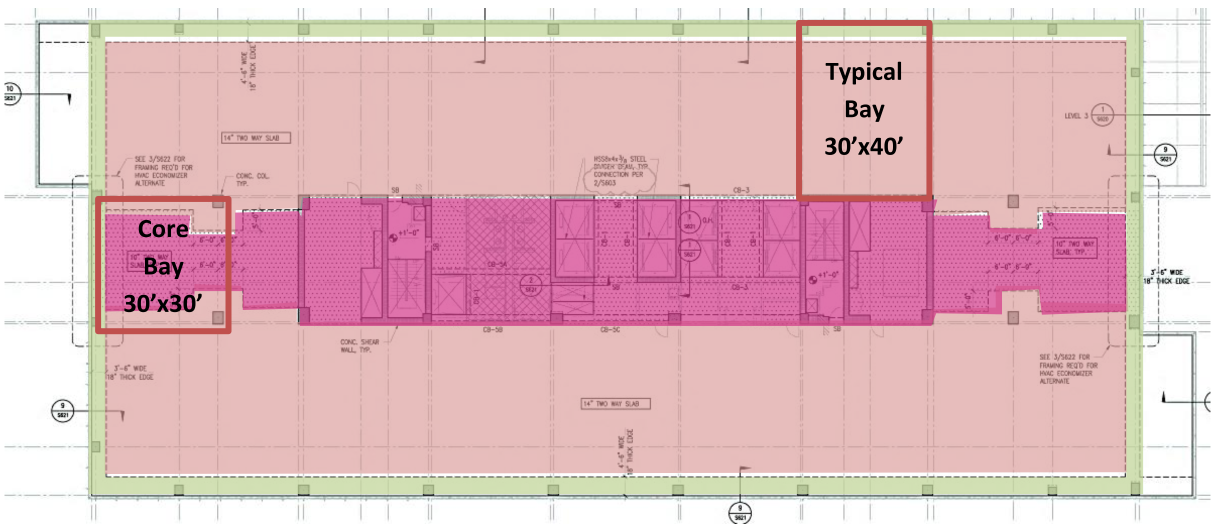


Figure 1.2 – 1 | Mat Foundation Thicknesses - S1L2

Gravity System

Floor System Overview

La Jolla Commons Tower II is rectangular building that is 315 feet long by 123 feet 8 inches wide. The building features a flat plate, two-way slab system on a rectangular column grid. As shown in **Figure 2.1 – 2**, the slab varies in thickness from 10 inches to 14 inches. The exterior edge of the slab at each level is framed by an 18 inch spandrel beam.



18" Thick Spandrel Beam
10" Thick Core Slab
14" Thick Slab

NORTH

Figure 2.1 – 2 | Typical Two Way Slab Thickness Layout – S103

Reinforcing of the slab varies based on direction and slab thickness. As with the mat foundation, the floor system has increased sizes and frequency of rebar near the core (where the shear walls are located). Reinforcing also varies based on column strip and middle strip locations. As required by ACI 318-08, reinforcing for the slab does not exceed a spacing of 18 inches.

Typical bay sizes are 30 feet by 40 feet at the east and west sides of the core. Bay sizes in the core vary due to shear wall placement. Also, column spacing at the core does not exactly match that of the exterior columns; however, the largest core bay size is 30 feet by 30 feet. **Figure 2.1 – 2** calls out the two typical bay sizes.

Camber of the structural slabs is used extensively for La Jolla Commons Tower II. Due to the fast construction of LJC II, construction loads were significant and played a major role in the design. Designers assumed that the slab would be loaded to the limit during construction, causing cracking. The slab was then analyzed for creep as a cracked section to determine the worst possible conditions; deflections were great enough that camber was required. Nabih Youssef Associates consulted documents such as ACI 435 to determine creep and shrinkage.

Roof System

The roof system for La Jolla Commons Phase II Office Tower is similar to that of the floor system. The main difference in the gravity system is the introduction of drop panels on the roof system. Drop panels are utilized on the roof level due to high loads associated with the rooftop mechanical equipment. Aside from this, the slab is 10 inches thick and features an 18 inch edge beam.

Concrete Columns

The entire gravity system is supported by a series of columns of various sizes on a rectangular column grid. Column sizes range from a maximum size of 42 inches by 42 inches at Lower Level 2 (lowest level of the underground parking garage) to a minimum size of 24 inches by 24 inches at the penthouse. Vertical reinforcing varies significantly based on column height, dimensions, and location. However, all columns have #5 ties spaced at 4 to 6 inches on center. Minimum requirements from ACI 318-08 (CBC 2010) for spacing and quantity of reinforcement have been met. When the columns were designed, they were considered fixed when applying only gravity loads to account for any eccentricity in the loading. However, when the lateral system was designed, the columns were considered pinned. In the event of an earthquake, the column bases would crack and create a pinned condition; the columns would, therefore, take minimal lateral load.

Lateral System

Shear Walls and Moment Frame

La Jolla Commons Phase II Office Tower has a lateral system of special reinforced concrete shear walls; moment frames are utilized for the lateral support of the penthouse at the roof cooling tower. All lateral systems were designed and detailed following Chapter 21 of ACI 318-08 for earthquake loading. See **Figure 1.2 – 3** for the concrete shear wall layout for the lateral force resisting system.

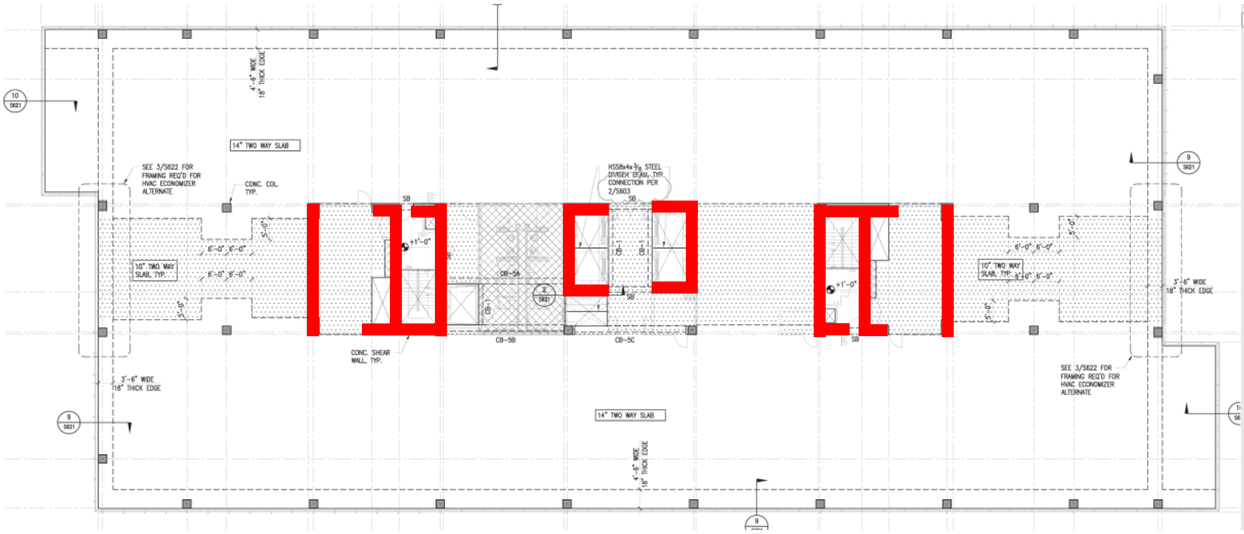


Figure 1.2 – 3 | **Typical Shear Wall Layout – S109** NORTH

Collector Beams

Collector beams are utilized on Lower Level 1 (upper level of parking) and the Ground Level of LJC II. Collector beams are used in high seismic areas to transmit earthquake forces into the main lateral system components. These elements give you the stiffness to transmit the forces through the diaphragm which cannot efficiently transmit the earthquake loads to the lateral system on its own.

Collector beams mainly run in the north-south direction, except for a few collector beams in the east-west direction on the Ground Level. Collector elements provide a direct path for the lateral loads from the diaphragm into the shear walls. This is especially important if the shear walls are not continuous, are spaced far apart, or are minimal, as is the case with the shear walls in the north-south direction. ACI 318-08 covers the requirements of collector elements in great detail in Section 21.11.

1.3 – Design Codes and Standards

Design Codes and Standards Used in the Original Design

- ✓ *California Building Code 2010*
- ✓ *Metal Building Manufacturers Association*
    - MBMA Recommended Design Practice Manual
- ✓ *American Iron and Steel Institute*
    - Applicable sections of the AISI Specifications
- ✓ *American Society of Civil Engineering*
    - ASCE 7-05 (as Adopted by IBC 2009) – Minimum Design Loads for Buildings
- ✓ *American Concrete Institute*
    - ACI 318 – 08 (as Adopted by IBC 2009) – Building Code Requirements for Structural Concrete

Design Codes and Standards Used in the Redesign

- ✓ *International Building Code 2012*
- ✓ *California Building Code 2013*
- ✓ *American Society of Civil Engineering*
    - ASCE 7-10 – Minimum Design Loads for Buildings
- ✓ *American Concrete Institute*
    - ACI 318 –11 – Building Code Requirements for Structural Concrete
- ✓ American Institute of Steel Construction
    - Steel Construction Manual, 14^{th} Edition
    - Seismic Design Manual and Seismic Provisions
    - Design Guide 11: Floor Vibrations Due to Human Activity

1.4 – Structural Proposal

Design Scenario

As previously mentioned, La Jolla Commons Phase II Office Tower is a completely concrete structure. After the investigations in Technical Reports 1 through 4, there are no obvious problems with the building's current structural system. Therefore, a scenario has been created in which the building owner, HINES, would like the structural engineer to design a composite steel structure. The owner would like the structural engineer to investigate the implications of the steel redesign on the construction schedule and building cost as compared to the concrete structure. The structural designer must investigate the potential serviceability issues associated with switching the system from concrete to steel; the main one to be investigated is vibrations due to human live loading.

It has also been requested by the owner that the lateral system be modified to include steel moment frames around the building perimeter in addition to the shear walls at the core. The structural engineer must consider cost and schedule effects of the additional frames and provide a recommendation as to their effectiveness and feasibility.

Learning Objectives

La Jolla Phase I Office Tower, the building nearly identical to La Jolla Phase II Office Tower, is a steel structure located right next to LJC II. The building's lateral system also consists of shear walls at the core, much like Tower II. Therefore, the design of Tower II in steel is possible and considerably feasible. One learning objective of this redesign is to investigate both systems and gain a better understanding of the advantages and disadvantages of a steel versus a concrete gravity system. By considering the effects of changing the structural system on the schedule, cost, and serviceability conditions, the advantages and disadvantages of the floor systems can be critically compared from several viewpoints, allowing the designer to make a more informed decision.

The lateral system for LJC II is special reinforced concrete shear walls. Many of the shear walls are very thick and require significant reinforcing. In order to learn more about the seismic detailing for steel moment frames and their efficiency in resisting lateral loads, the incorporation of steel frames as part of the lateral system will be investigated.

An investigation of structural vibrations due to human live loading will be performed for the steel gravity system in the office space. This will be done because the spans are quite long for many of the steel girders, and vibrations are more of a concern with the steel system than the concrete system.

Overall, the goal of this redesign is to develop a better understanding of the design of steel structures and special steel moment frames and a better understanding of the cost, schedule, and serviceability considerations for steel versus concrete. Another major goal is to develop a better understanding of the design of steel structures for seismic loading conditions.

Proposed Methods and Solution

The building's gravity system will be redesigned in composite steel utilizing the same column locations as the original concrete system, limiting impact on the current building layout and architecture. The gravity system for the two underground parking levels will remain concrete. Because the gravity system consists of many members of the same length and loading, beams and girders will be initially designed by hand to determine appropriate member sizes using the AISC Steel Construction Manual, Fourteenth Edition. Next, a detailed RAM Steel gravity model will be developed using the dead loads associated with the new system and the previously determined live loads. The model will aid in the determination of member adequacy when considering both strength and economy.

As determined in Technical Report 3, the floor system for the proposed redesign will consist of composite metal deck such as 2 VLI 18 with a 4.5 inch normal-weight concrete topping, total thickness of 6.5 inches. The girders are expected to reach a maximum depth of 30 inches, and the infill beams are expected to reach a maximum depth of 14 inches. See **Figure 1.4 – 1** for the possible layout for a typical 30 ft x 40 ft bay. In order to limit the overall depth of the system, additional rows of columns may need to be added at mid-span. However, in order to limit impacts on the original architectural layout of the space, the original column locations will be investigated first. Different infill beam spacing and layouts will be investigated to determine the most efficient and "architecturally friendly" system. The columns will then be designed and tested using the RAM gravity model.

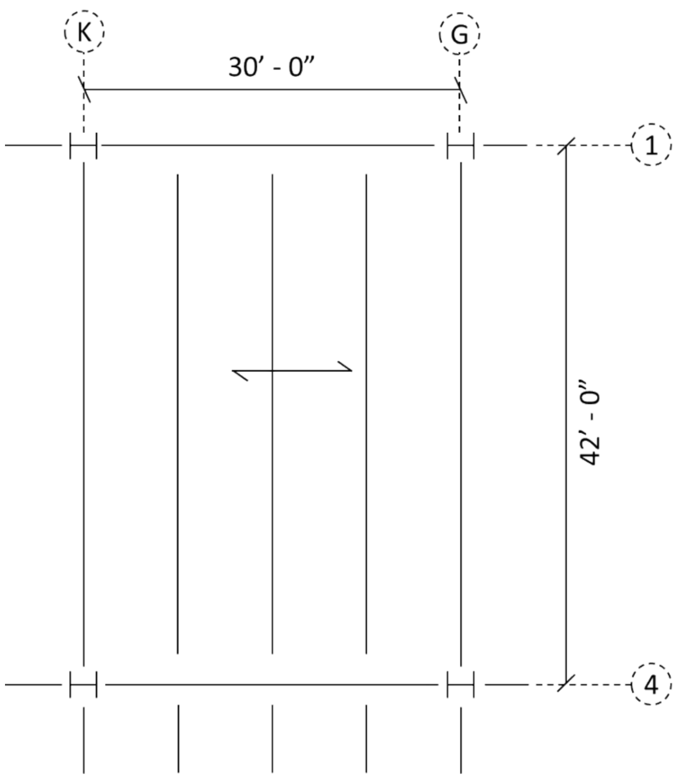


Figure 1.4 – 1 | **Potential Steel Framing Layout**

Once the development of the composite steel gravity system is complete, an analysis of the structure's lateral system will be performed. First, the building lateral loads will need to be recalculated using ASCE 7-10. ETABS 2013 will be used to perform a Modal Response Spectrum Analysis on the building's lateral system to determine the seismic loads. ETABS 2013 will also be used to generate the building wind loads. The ETABS 2013 model used in Technical Report 4 will be modified to accurately represent the shear walls. The model will then be modified to incorporate steel moment frames. A redesign of the concrete shear walls will need to be performed, and the moment frames will also be designed and detailed for seismic considerations. **Figure 1.4 – 2** shows a potential layout for the added moment frames.

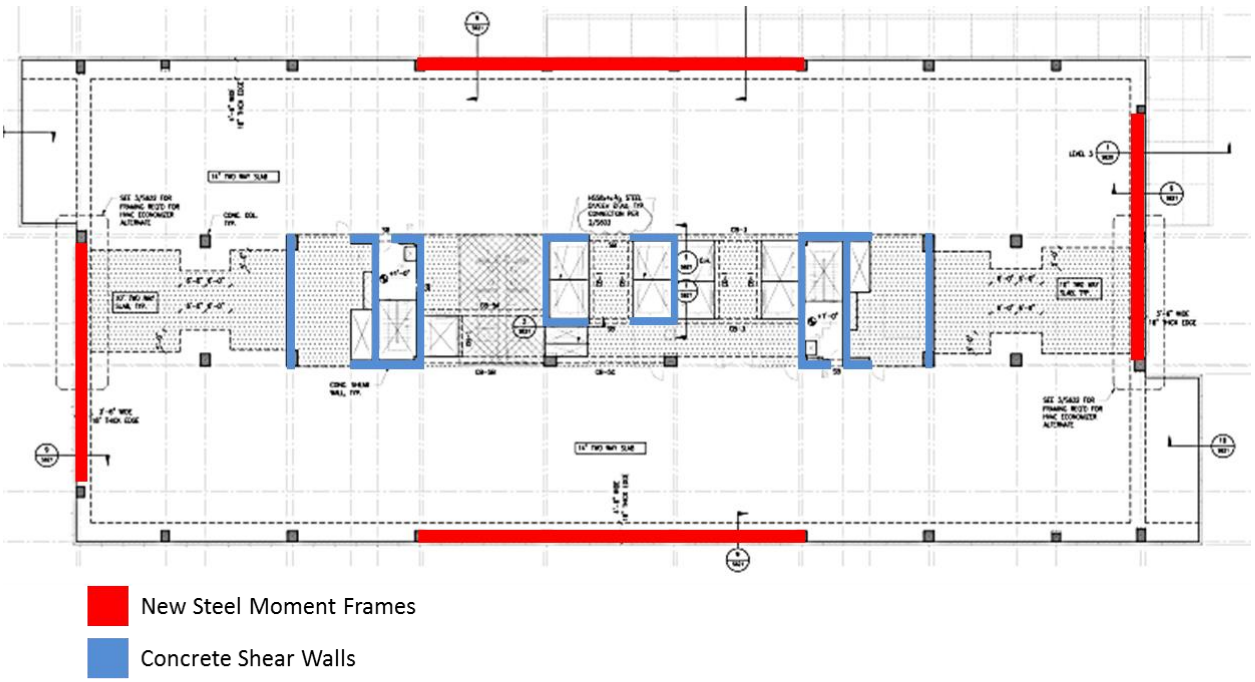


Figure 1.4 - 2 | **Potential Lateral System Layout**

An investigation of the vibrations associated with human activity on a typical bay of the steel gravity system will be performed. Calculations will be done by hand, following the provisions of AISC Design Guide 11: Floor Vibrations Due to Human Activity. These calculations may also be verified, if time allows, using the RAM Steel model.

MAE Requirements

Graduate level work will be used throughout the design and analysis of the proposed structural system. AE 530 – Advanced Computer Modeling of Building Structures will be utilized in the creation and evaluation of both an ETABS lateral model and a RAM Steel gravity model. Because the building is in SDC D, material from AE 538 – Earthquake Resistant Design for Buildings will be used to design and detail the building lateral system of concrete shear walls and steel moment frames. Also, additional work is being done to expand into an area of study not yet learned by the designer: vibrations analysis.

Breadth Studies

Cost and Schedule Analysis

A detailed cost estimate of the proposed structural system will be completed. This cost will then be compared to that of the existing structural system. In addition, a construction schedule for the redesigned system will be studied and compared to that of the existing structural system. These analyses will then be used to determine which system is more economical. RS Means will be used for most durations and costs; however, information will be requested from the project general contractor.

Architectural/ Fire Protection Analysis

Changing the structure from concrete to steel will have different effects on the building's architecture. One item to be investigated is the fire protection of the building structure. Although the structural slab will provide the 2 hour fire rating between floor levels, steel beams and columns will remain exposed. As a result, an investigation will be performed on the ceiling system, floor systems, and wall systems to determine their fire protection adequacy. An analysis on the impact of the structural changes on the building height will also be performed. The building height is limited due to FAA regulations; therefore, an analysis on the floor-to-floor heights will be done to determine if a height increase will be necessary.

Chapter 2 – Structural Depth

2.1 – Gravity System Design

For this redesign, it was desired to leave the layout and architecture of the spaces as unchanged as possible. Therefore, the column locations were not changed, and the original grid was used, with girders spanning between original column locations. Also, the serviceability criteria associated with vibrations was a major design parameter for the new steel gravity system. Final plans of the gravity design can be viewed in **Appendix B**.

Preliminary Vibrations Analysis

Vibrations were a primary concern when redesigning the steel floor system for the office space at La Jolla Commons. Vibrations can result from walking down a corridor, vibrating equipment, and other sources. For this particular building, the spans for the bays in the lease space were quite long at about 40 feet. As a result, vibrations due to human excitations could cause noticeable motion for the workers in the office space.

The design of the gravity system began with a preliminary vibrations analysis following *The Preliminary Assessment for Walking-Induced Vibrations in Office Environments* article by Dr. Linda Hanagan and Taehoo Kim. The procedure outlined in this paper allows the designer to select a slab and deck configuration and a beam spacing that will produce a suitable floor system for human induced floor vibrations, according to the criteria of AISC *Design Guide 11*. This calculation can be done without having to complete the arduous vibrations calculation outlined in *Design Guide 11* for each potential design.

Table 2.1 – 1 | Deck Configuration for Vibration Control

Concrete Strength	3000 psi
Steel Grade	50
Deck Type	*1.5VLR20*
Topping (in)	*4.25*
LW/NW?	*LW*
Total Slab Thickness (in)	5.75
Class from Table 1	4
Select C1 from Table 2	0.413
Select C2 from Table 4	0.019
Evaluate C1 + C2	0.432
C1 + C2 < 0.5?	*GOOD*

Using this procedure, many different deck and framing configurations were considered. After several iterations, a deck size and slab thickness was determined. The results of the preliminary vibrations assessment can be viewed in **Table 2.1 – 1**. See **Appendix C** for a spreadsheet of all the calculations for the other design options that were considered. As can be seen, 1.5VLR20 Composite Deck with 4.25" lightweight topping was selected with beam spacing at 7'-6" to 8'-0". This configuration allows for un-shored construction which has potential for cost and time savings. Lightweight concrete is desirable for the slab on deck design in order to reduce the building's seismic weight; therefore, most designs considered used lightweight concrete. The calculations verifying strength requirements of the chosen deck can be viewed in **Appendix D** of this report.

Gravity System Layout

The next step was to determine whether to span the infill beams in the long or the short direction; **Figure 2.1 – 1** and **Figure 2.1 – 2** show a typical bay for each option. It was thought that the shorter span would be a more economical option because it would yield lighter beams with smaller depths. However, the option of the beams spanning in the longer direction produced a design that, although heavier overall, actually required significantly less members for the floor system. It was determined that the cost of the heavier system was offset by the reduction in time to erect the floor system, mainly considering crane rental and operations costs. **Table 2.1 – 2** shows the weight and the number of members for each framing option. The table reflects a typical floor from levels 3 to 7. As a result of this analysis, the long direction layout was chosen as shown in **Figure 2.1 – 1**.

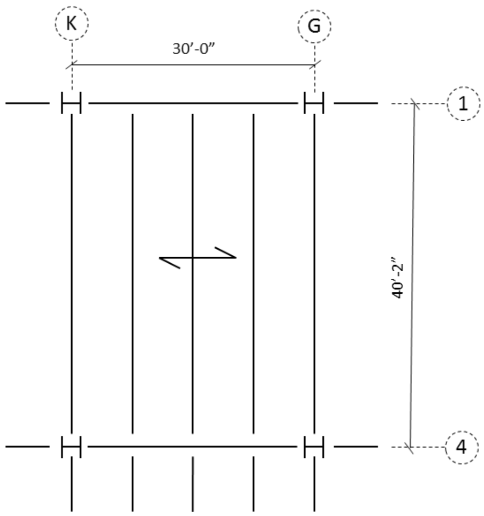


Figure 2.1 - 1 | Infill Beams Long Direction

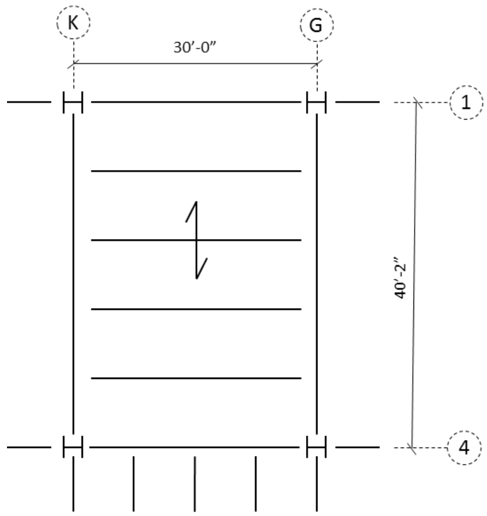


Figure 2.1 - 2 | Infill Beams Short Direction

Table 2.1 – 2 | Infill Beam Comparison for Typical Level 3-7 Layout

	Steel Weight (lbs)	Number of Members	Number of Studs
Long Direction	212936	155	3490
Short Direction	179608	225	4489

Also, an additional option for the gravity framing system layout was considered that required an extra row of columns to break up the long 40 foot span. However, the system created 75% more floor framing members than the long span option show above in **Figure 2.1 – 1**, and it only saved a maximum of three inches in structural depth for each level. This did not seem to be enough of a depth savings to warrant the interruption of the office layout by adding the row of columns or to add additional time to the construction schedule. Therefore, this layout was not investigated further.

RAM Structural System – Gravity Model

Next, keeping the constraints for vibrations, economics, and constructability in mind, a 3D RAM Structural System model was created to design the gravity system. **Figure 2.1 – 3** shows a 3D view of the gravity model. Girders span the N-S direction between columns at the original locations, and the infill beams span the long E-W direction. 1.5VLR20 deck with 4.25" LW concrete topping is used for all floor levels. Roof deck was designed to be 1.5B20. The maximum beam spacing is 7'-6". See **Appendix D** for the deck design calculations. The RAM layout for a typical level is shown in **Figure 2.1-4**.

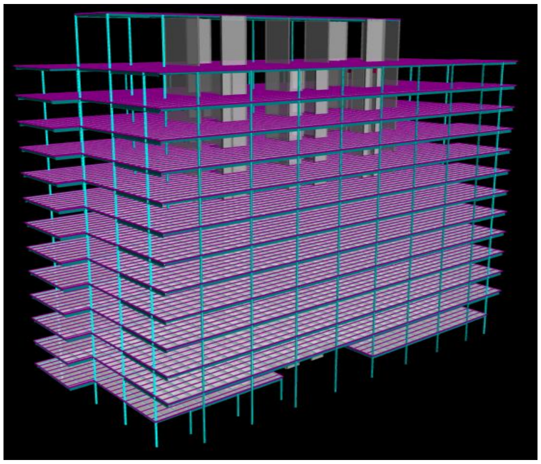


Figure 2.1 – 3 | 3D View of RAM Gravity Model

All members were drawn as line elements on the grid system that was used for the concrete system to limit impact on the architecture and floor layouts. The original shear walls were left in their original locations, and later moment frames will be added to the perimeter. Shear walls were placed using shell elements and have been defined as lateral elements using the thicknesses of the original shear walls. Concrete coupling beams have also been included in this model at the original sizes.

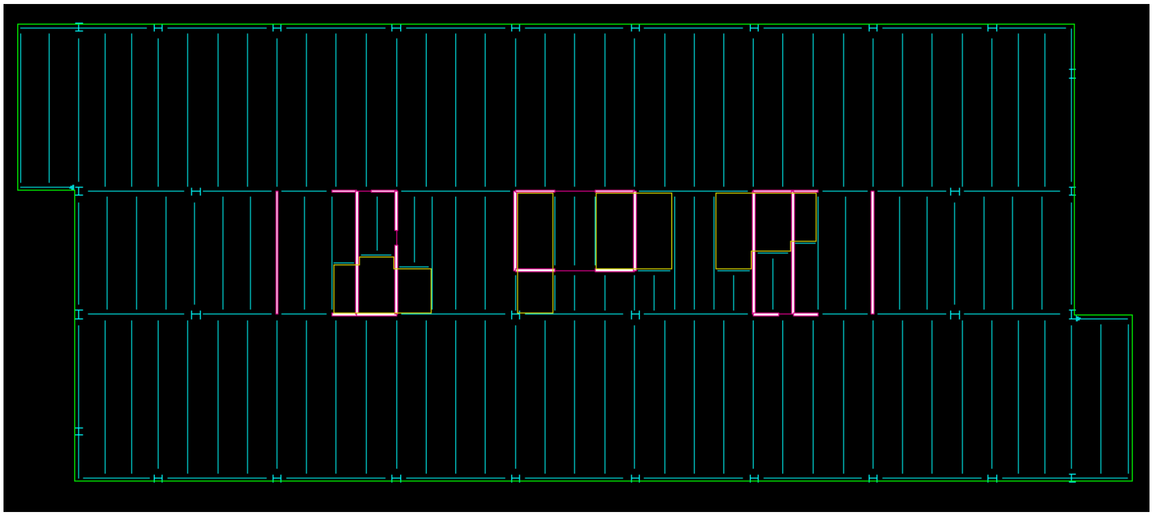


Figure 2.1 - 4 | Framing Layout for a Typical Level

NORTH

It is also important to note that gravity members will be supported by the existing, load-bearing concrete shear walls. This connection was modeled as pinned using RAM Structural System. Later in this report, a suggestion for the connection type will be discussed.

Gravity Design Loads

Table 2.1 – 3 includes the gravity loads that were applied to the RAM Model for the gravity system design. The loads include the dead load of the decking, live load, partition loads, wall loads, and construction loads. Construction loads were used to determine the strength of beams before concrete cures to allow composite action.

Table 2.1 – 3 | Live and Dead Loads Applied to RAM Model

Location	Dead (PSF)	Live (including Partitions) (PSF)	Construction Live Load (PSF)	Construction Dead Load (PSF)	Exterior Wall Load (KLF)
Lobby Around Core	90	100	20	50	n/a
Exit Stairs	90	100	20	50	n/a
Building Core Egress	90	250	20	50	n/a
Restrooms	90	60	20	50	n/a
Lease Space	90	80	20	50	0.118

Location	Dead (PSF)	Live (including Partitions) (PSF)	Construction Live Load (PSF)	Construction Dead Load (PSF)	Exterior Wall Load (KLF)
Building Core Egress	90	250	20	50	n/a
Mechanical	90	200	20	50	n/a
Lobby Around Core	90	100	20	50	n/a
Exit Stairs Core	90	100	20	50	n/a
Restrooms	90	60	20	50	n/a
Lease Space	90	80	20	50	0.118

Location	Dead (PSF)	Live (including Partitions) (PSF)	Construction Live Load (PSF)	Construction Dead Load (PSF)	Exterior Wall Load (KLF)
Penthouse Area	90	250	20	50	0.1
Roof	23.5	20	n/a	n/a	n/a

Gravity Beam Design

The RAM Structural System model was used to design the gravity beams for every level of the structure. The beams were designed to be composite with a minimum size of W12x14 for fire protection concerns. The following design decisions were made in the RAM Beam Design Module: AISC 360-10 LRFD was selected as the design code, unbraced lengths were considered by RAM for design, C_b=1.0 was used for all simple span beams, no camber was allowed in beam design, 100%-25% composite action was allowed for efficiency, and a uniform distribution and even number of studs were specified for each beam. No camber was allowed due to the increased cost associated with cambering the large number of steel beams on this project.

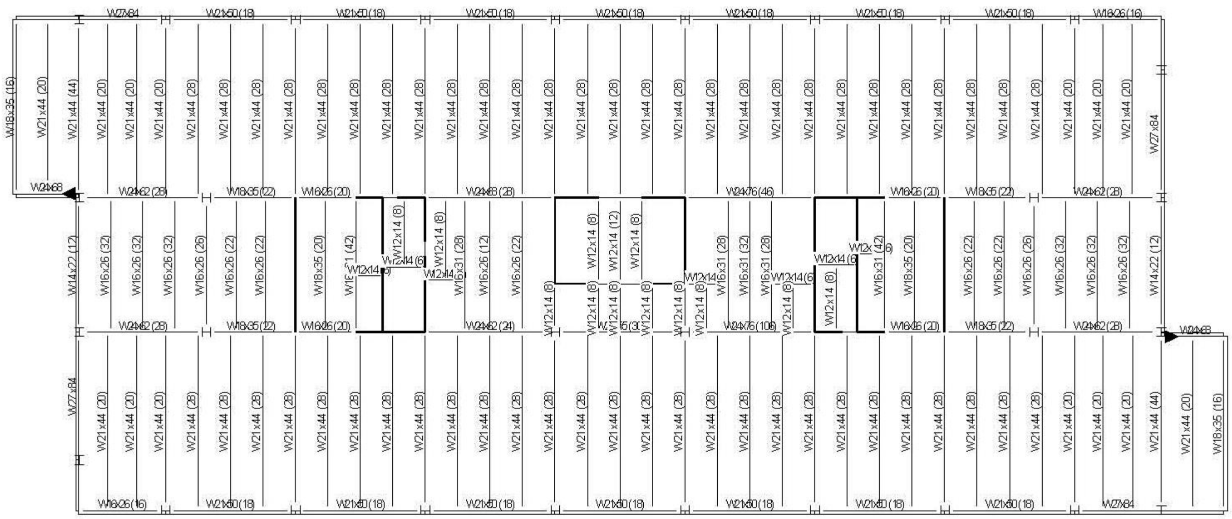


Figure 2.1 - 5 | Gravity Beam Designs – Level 3-7

The RAM design was then checked using hand calculations to verify the composite strength, unshored strength, wet concrete deflection, and live load deflections for the infill members in a typical bay and for a typical girder. Also, a non-composite, cantilever beam was checked for strength and deflections. These calculations can be viewed in **Appendix E**. Common member sizes were W21x44 at the exterior 41'-2" x 30' bays and W16x26 at the interior 30' x 30' bays. The deepest member at each level was a W24x62 near the building core. **Figure 2.1 – 5** shows the final and verified gravity floor system design for a typical level. The final gravity design plans can be viewed in **Appendix B**.

Some steel gravity members frame into the existing load bearing shear walls. The connection between the concrete shear walls and the new steel members will require some special attention. A potential option for the shear connection between the gravity beams and lateral shear walls is shown in **Figure 2.1 – 6**. This detail shows a steel plate embedded into the concrete shear wall with shear studs. The steel beam is then bolted or welded to the steel plate.

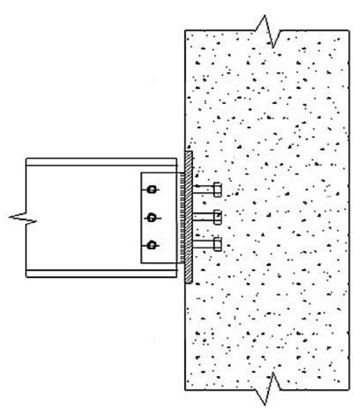


Figure 2.1 - 6 | Beam to Shear Wall Connection

Gravity Column Design

RAM Structural System was also used to design the columns under gravity loading. The AISC 360-10 LRFD design code was utilized. Columns were spliced every two stories, this was done for safety reasons during construction. Also, the length of a two-story column is reasonable to transport at a height of 24 feet. During the design process, the column trial groups were limited to W14, W12, and W10 in order to maintain a relatively square cross section to limit buckling concerns. The columns were designed using the gravity loading previously shown in **Table 2.1 – 3**.

The design of the columns utilized the live load skip loading, provided by the RAM Steel Column module. This allows the program to determine the worst case loading for the column in order to design the most effective cross section. Beams that connected into each steel column were assumed to brace the columns at that location, and, where applicable, the floor system was assumed to brace columns as well.

Gravity columns were optimized to have an interaction below 1.0 in accordance with Equation H1-1a of the AISC *Steel Manual*. Also, the design column depth at a particular column location was consistent over the entire building height. For example, column line Y-7 utilized column sizes W10x33, W10x39, W10x49, W10x60, and W10x88, the sizes getting heavier as you move down the column line. This helps to make the splicing of the columns possible.

The column designs ranged widely depending on the location in the building. The designs of columns near the center of the building, carrying more gravity load than exterior columns, ranged in size from W14x193 to W14x176 at Level 2 of the building. At the roof level, most column sizes were found to be W14x43. For lighter loaded columns near the building perimeter, the column designs ranged from W10x88 to W12x136 at Level 2. At roof level, the sizes ranged from W10x33 to W12x40.

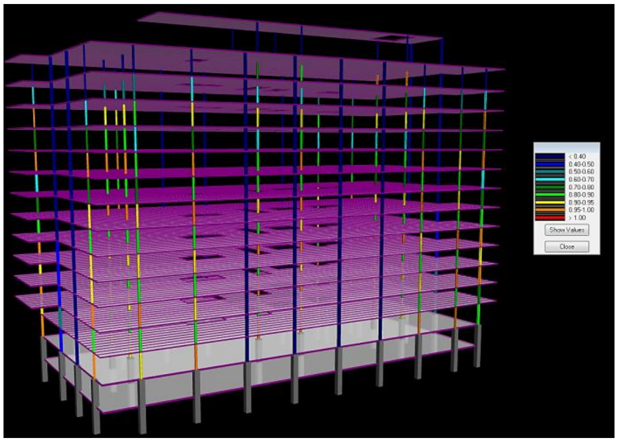


Figure 2.1 - 7 | RAM Steel Column Module Interactions

Figure 2.1 – 7 shows a screen shot from the RAM Steel Column module. This image shows the interaction values of the gravity columns, using a color scale. Orange indicates an interaction between 1.0 and 0.95, yellow indicates 0.90 – 0.95, and so on as the colors get cooler in color. Notice, that the only blue members, which have an interaction of 0.40 or less, are either the top story columns or columns in the rigid moment frames. The moment frame columns have been significantly increased to resist lateral forces, and therefore, the gravity interactions are minimal.

An interior and an exterior column have been hand checked for strength under gravity loads. The designs produced by RAM are based on a detailed analysis, including P-Delta effects and skip loading. The hand spot check of columns performed was very simple and only verifies strength under concentric, axial gravity loading. This was done to make sure that the designs from RAM seemed reasonable without performing all the arduous calculations. See **Appendix E** for this rough column hand check.

Final Vibrations Analysis

As mentioned before, vibrations was a primary concern when designing the floor framing system for the office space. Although vibrations are a serviceability condition, annoying vibrations can impact the occupants and their productivity in a space. The human response to walking in an office space can vary based on the magnitude and frequency of the motion, the environment, and the particular person sensing the motion. Vibrations are a continuous or steady state motion which can often be more annoying than a single impulse. Vibrations are of particular importance in an office space where computer monitors and other items can shift on desks and stationary sensors will be more likely to notice the motion.

A detailed vibration analysis was performed for a typical bay following the procedure described in *AISC Design Guide 11 – Floor Vibrations Due to Human Activity*. This was done in order to determine if the floor design was adequate for human induced floor vibrations to create a more comfortable and productive work environment. The analysis was done on the typical bay shown in **Figure 2.1 – 8**. The bay followed the basic requirements of the *Design Guide 11* procedure.

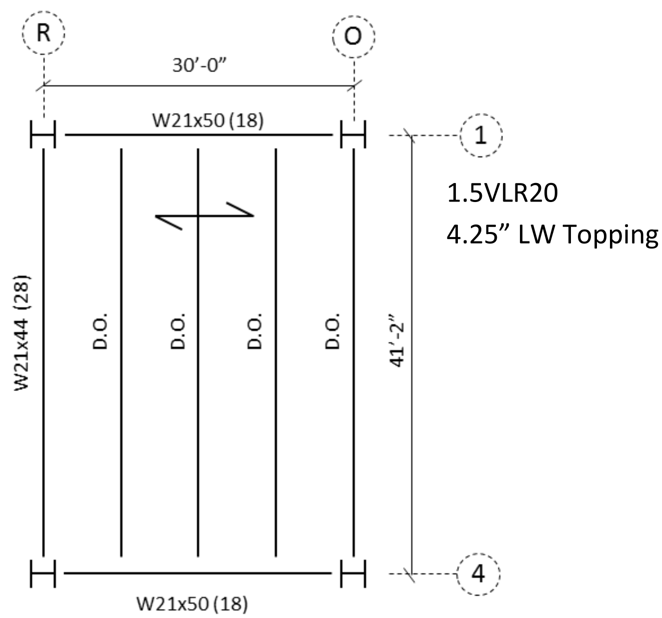


Figure 2.1 - 8 | Typical Bay Checked for Vibrations Performance

A live load of 11 PSF was assumed as suggested for office spaces by *Design Guide 11*. Table 4.1 pf *DG-11* was used to determine P_0, B, and a_0/g for the office space. After calculating the combined mode properties of the beams and girders, the natural frequency of the floor system and the equivalent panel weight could be determined. This was then used to calculate a_p/g to compare to the acceptable a_0/g value from Table 4.1. It was then determined that the bay is acceptable for human induced vibrations according to AISC *Design Guide 11*. See **Appendix F** for the full vibration calculation of a typical bay.

$$\frac{a_0}{g} \geq \frac{a_p}{g} \rightarrow 0.5\% \geq 0.38\% \quad \textit{(Equation 2.3 – AISC DG 11)}$$

2.2 – Lateral System Design

The original lateral system of La Jolla Commons Phase II Office Tower was found to have an extreme torsional irregularity through Technical Report 4 investigations. In an effort to control the building torsion, the lateral system was modified to include perimeter moment frames. The frames and existing shear walls are designed and analyzed according to IBC 2012, ASCE 7-10, ACI 318-11 and AISC-360 LRFD.

Wind and Seismic Loads

Wind and seismic loads were calculated according to ASCE 7-10. Wind and seismic loads were generated by RAM Frame and were verified using hand calculations. See **Table 2.2 – 1** and **Table 2.2 – 2** for hand calculated wind and seismic loads, respectively. As expected, seismic loads, even with the reduced weight of the steel structure, control the lateral design. **Figure 2.2 – 1** shows the seismic load distribution over the building height.

It was determined that, if the torsional irregularity for La Jolla Commons could be eliminated, the Equivalent Lateral Force Procedure would be allowed to be used to find the design seismic forces. Because the building exceeds 160 feet, the building period must be less than $3.5T_s$ in order to use the ELF Procedure. The building period is 2.74 seconds which is much less than $3.5T_s$ which is 9.42 seconds. A Response Modification Coefficient, R, of 6 was used, and a redundancy factor of 1.3 was applied. These values are for specially reinforced concrete shear walls which will remain part of the lateral force resisting system. As previously stated, moment frames will be added; however, because the moment frames do not take 25% or more of the seismic forces, the R value was not increased to 7 for dual systems. See **Table 2.2 – 3** for the verification that a dual system does not exist. **Appendix G** shows the hand calculations done based on ASCE 7-05; these hand calculations were then modified using Excel to update to ASCE 7-10. **Figure 2.2 – 2** shows the response spectrum used for the calculation of the new seismic loads. This information was generated by the *USGS* online calculator.

Wind Loads

Table 2.2 – 1 | Wind Loads ASCE 7-10

Wind Pressures | North South				Wind Pressures | East West			
Level	**Height**	**Force (k)**	**Story Shear (K)**	**Level**	**Height**	**Force (k)**	**Story Shear (K)**
Ground	0	44.7	583.31	**1**	0	23.28	1614.85
2	15	34.61	538.6	**2**	15	106.06	1591.57
3	28.17	35.2	504	**3**	28.17	105.68	1485.51
4	41.34	37.14	468.8	**4**	41.34	110.13	1379.83
5	54.51	38.64	431.66	**5**	54.51	113.57	1269.7
6	67.68	39.87	393.02	**6**	67.68	116.4	1156.14
7	80.85	40.93	353.14	**7**	80.85	118.82	1039.74
8	94.02	41.86	312.21	**8**	94.02	120.96	920.91
9	107.19	42.7	270.35	**9**	107.19	122.86	799.96
10	120.36	43.45	227.65	**10**	120.36	124.59	677.09
11	133.53	44.14	184.2	**11**	133.53	126.18	552.5
12	146.7	44.78	140.06	**12**	146.7	127.65	426.32
13	159.87	47.67	95.27	**13**	159.87	135.53	298.67
PH	173.04	36.29	47.6	**PH**	173.04	117.03	163.14
PH Roof	198.67	11.31	11.31	**PH Roof**	198.67	46.11	46.11
Vb =	**583 kips**			**Vb=**	**1615 kip**		

Seismic Loads

Table 2.2 – 2 | Seismic Story Forces ASCE 7-10

T=	1.056 s		k=	1.278	V_b=	4293.3 K
			Story Forces Calculation			
Level	**hi (ft)**	**h (ft)**	**W (kip)**	**W*h^k**	**Cvx**	**Story Forces Fi (kip)**
Penthouse Roof	24.33	198.70	380	328408	0.0158	67.91
Penthouse Floor	14.50	174.37	3735	2734571	0.1317	565.46
13	13.17	159.87	4631	3034717	0.1462	627.52
12	13.17	146.70	4631	2718953	0.1310	562.23
11	13.17	133.53	4631	2410981	0.1161	498.55
10	13.17	120.36	4631	2111350	0.1017	436.59
9	13.17	107.19	4631	1820712	0.0877	376.49
8	13.17	94.02	4631	1539854	0.0742	318.41
7	13.17	80.85	4631	1269753	0.0612	262.56
6	13.17	67.68	4631	1011655	0.0487	209.19
5	13.17	54.51	4631	767221	0.0370	158.65
4	13.17	41.34	4633	539025	0.0260	111.46
3	13.17	28.17	4631	330013	0.0159	68.24
2	15.00	15.00	4569	145491	0.0070	30.08
		SUM:	**59630**	**20762706**		**4293.34**
Base Shear =	**4293.3**	**kip**				

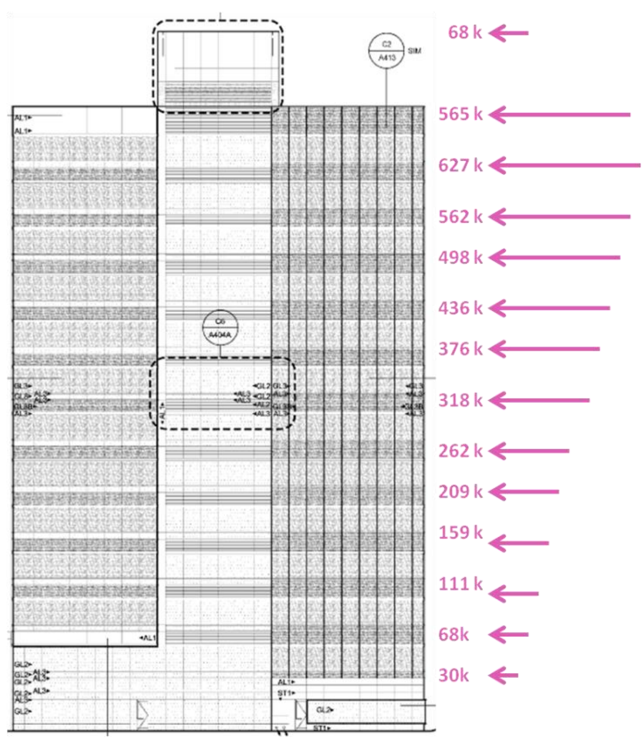


Figure 2.2 - 1 | Seismic Load Distribution

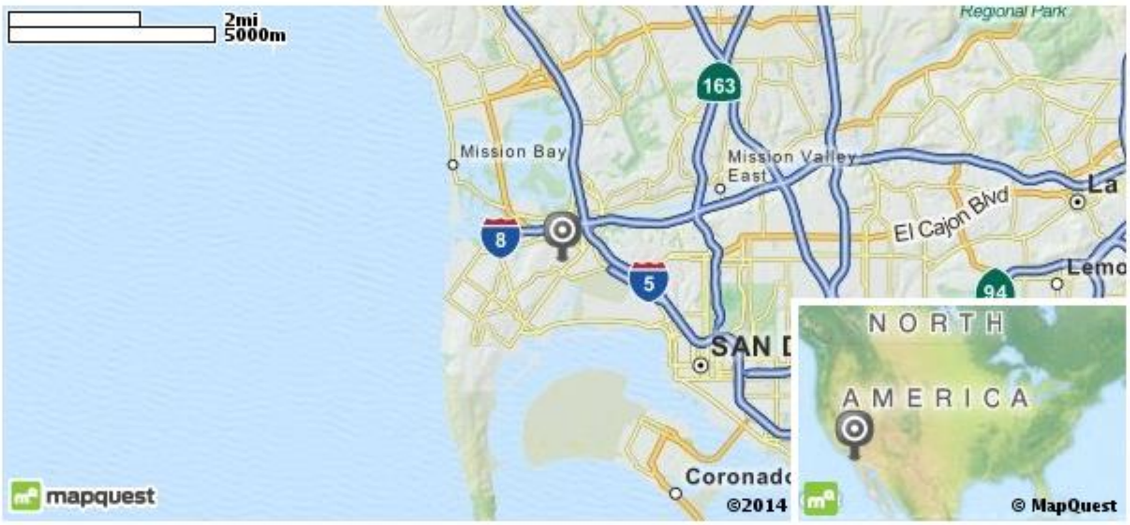
Table 2.2 – 3 | Dual System Check

X-Direction Direct Shear			
Item	Shear (kip)	% of Total Shear	Dual System?
Frame 1	595.21	18.27%	No
Frame 2	643.37	19.74%	No
Shear Walls	2020.00	61.99%	-
Total Shear	3258.58 kip		

Y-Direction Direct Shear			
Item	Shear (kip)	% of Total Shear	Dual System?
Frame 3	35.61	1.18%	No
Frame 4	32.41	1.08%	No
Shear Walls	2941.00	97.74%	-
Total Shear	3009.02 kip		

User–Specified Input

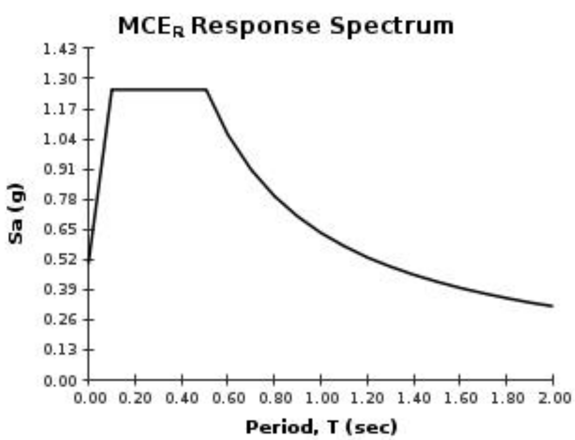
Report Title	LJC II Thu March 6, 2014 21:17:25 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	32.74714°N, 117.21179°W
Site Soil Classification	Site Class C – "Very Dense Soil and Soft Rock"
Risk Category	I/II/III



USGS–Provided Output

$S_S =$ 1.252 g	$S_{MS} =$ 1.252 g	$S_{DS} =$ 0.835 g
$S_1 =$ 0.482 g	$S_{M1} =$ 0.635 g	$S_{D1} =$ 0.424 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.



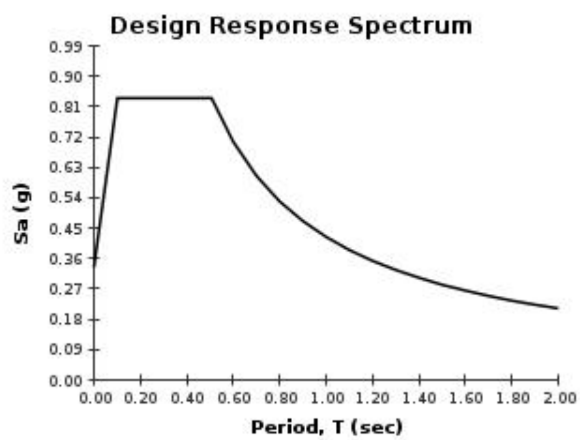


Figure 2.2 - 2 | **Seismic Response Spectrum Information**

Modeling Considerations and Verification

The lateral system was analyzed and designed using RAM Structural System. The model included the original special concrete shear walls with additional moment frames around the building perimeter. The model was used for the analysis and design of the new lateral system.

Load and Model Verification

First, RAM was used to generate the wind and earthquake loads for the building structure; this was done according to ASCE 7-10. Wind loads were calculated with the mean roof height at the top of the penthouse level and a K_{zt} of 1.0. A spreadsheet was created to determine the wind loads, **Table 2.2 – 1**; these wind loads were compared to those generated by RAM. The wind loads calculated by hand were found to be within 3.31% of the values calculated by RAM.

The earthquake forces were also calculated by hand as shown in **Table 2.2 – 2**. Again, these forces were then compared to the forces generated by RAM. Seismic loads were found to be within 15% of those generated by RAM. The loads generated by RAM were ultimately the forces used for the design of the building structure; however, the difference between the loads calculated by RAM and those calculated by hand could be due to one or a combination of the following:

- The difference in the approximate period used for the hand calculated ELF method and the actual building period calculated by RAM
- RAM uses more accurate story masses than the hand calculation, and masses were also updated as the structural design changed

RAM was used to generate load combinations according to the requirements of ASCE 7-10 and IBC 2012. A F1 value of 0.5 was used for the live loads. This was done because the building is business use only and will not be used for public assembly. Also, a redundancy factor of 1.3 was used to increase the seismic loads. This is a requirement of ASCE 7-10 Section 12.3.4.2 for Seismic Category D structures.

The center of mass and center of rigidity calculated by RAM were then verified using an Excel spreadsheet. Also, a 2D distribution of forces was done by hand on level 7 and compared to the RAM load distribution. All of these calculations were within a reasonable percent error of the values generated by RAM Frame. See **Appendix H** for spreadsheets of these calculations. **Table 2.2 – 4** shows the percent error for several items verified.

Table 2.2 – 4 | Model Verification Summary

	% Error X-Direction	% Error Y-Direction
Center of Mass	0.284%	1.265%
Center of Rigidity	2.813%	1.681%
Floor Mass	11%	
Seismic Loads	15%	
Wind Loads	0.25%	3.31%
2D Analysis	10 - 20 %	

Modeling Considerations

RAM Frame was used for the design of the concrete shear walls and steel moment frames; several modeling decisions needed to be made before design began. First, each floor diaphragm was modeled as rigid due to the 4.25" concrete topping on each floor. The roof diaphragm, however, is unfilled metal deck; therefore, the roof will need to be properly braced to behave as rigid. Because this is a reasonable addition to the design, the roof diaphragm was also modeled a rigid.

Next, the modeling of the moment frames was performed. According to the AISC *Seismic Design Manual,* panel zones must be considered in the design of moment frames; this is due to the significant increase it can have on lateral drift. As a result, panel zones were considered in the lateral design and analysis model. Also, P-Delta effects were considered using mass loads; this was done by RAM Frame using the Direct Analysis Method. RAM generated a B1 factor; however, a B2 factor was not required because the model was analyzed using P-Delta effects. According to the RAM Frame Manual, when P-Delta effects are considered in the analysis model, B2 is permitted to be taken as zero.

All moment frame bases were modeled as pinned. This was done because the moment frames will terminate on a foundation wall with concrete pilasters. This connection would be very difficult to design to transmit the rotational forces. Therefore, a pinned condition was assigned.

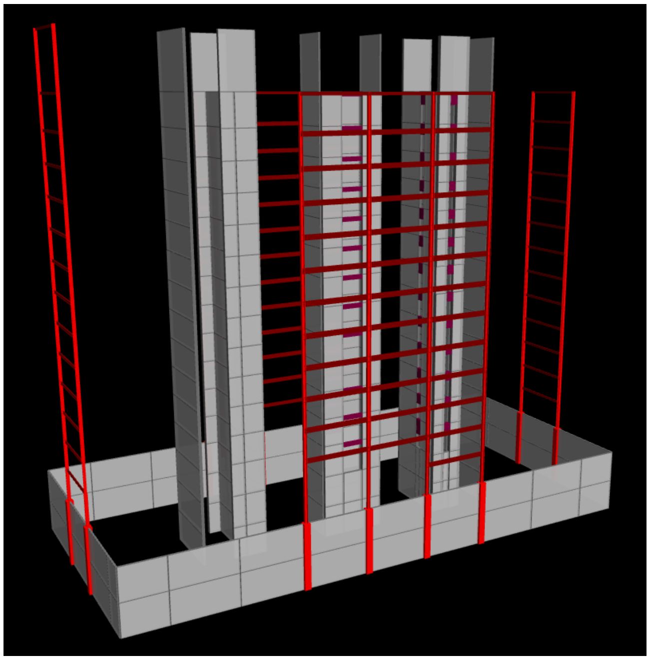


Figure 2.2 - 3 | 3D View RAM Lateral Model – Gravity Hidden

The shear walls were modeled as shell elements at the thickness of the original walls. They were modeled as cracked, with a 65% reduction on the wall stiffness in bending and shear, as required by ASCE 7-10 Section 17.7.3. As stated in this code section, when modeling for seismic design concrete, shear walls must include the effects of cracked concrete sections. According to ACI 318-11 § 8.8, a 65% reduction in wall stiffness is allowed for analysis and design purposes. Also, according to industry professionals, it was found that it is common practice on the west coast to crack all shear walls in accordance with ACI 318-11. The cracked walls are used for both strength and serviceability design of the building structure. This ensures that upon cracking and yielding of rebar in the shear walls, that the other LFRS elements can handle the increased loading. Also, the shear walls were modeled with a fixed condition at the base. The walls are tied to the mat foundation, which ranges thickness from 6.5 feet to 4.5 feet, by hooks at the end of each vertical bar. Therefore, it was decided that a fixed condition was reasonable. **Figure 2.2 – 3** shows the RAM Lateral Model with the gravity system hidden for clarity.

The modeling of the core shear walls posed a challenge. A strange phenomenon was occuring in the connected shear walls with respect to torsion; forces were developing in irregular patterns that did not correspond to what would be expected based on 2D analysis. Further investigations were done to determine if there were problems with the current modeling technique and to potentially find a new modeling technique that would eliminate this problem. It was discovered that the modeling of intersecting shear walls is a somewhat controversial topic; many different methods are used by practicing engineers. Because of this discrepancy, a modeling technique suggested by Bentley was selected. As stated by Bentley Technical Support Group, "…since RAM Frame assembles the stiffness coefficients of its elements in a 3D fasion, walls that intersect (and share common nodes) form a 3D system and the 3D behavior is captured by analysis. This is correct and consisted with finite element analysis." However, the inclusion of flanges is subject to wall detailing and limited flange lengths based on ACI 318. Therefore, in order to not account for flanged behavior that may not exist, a more conservative approach was taken as outlined by Bentley. Shear walls were disconnected by reducing shear wall lengths by 5 inches at each end. Gravity beams weret then placied in the gaps to prevent a "framing tables" error in RAM. The gravity member will not effect the lateral analysis and design of the structure. Using this modeling technique, the flange walls are not relied upon to resist bending and shear forces out-of-plane. This technique also eliminated the torsional anomally produced by the connected shear walls. **Figure 2.2 – 4** shows this modeling technique applied to a set of intersecting walls.

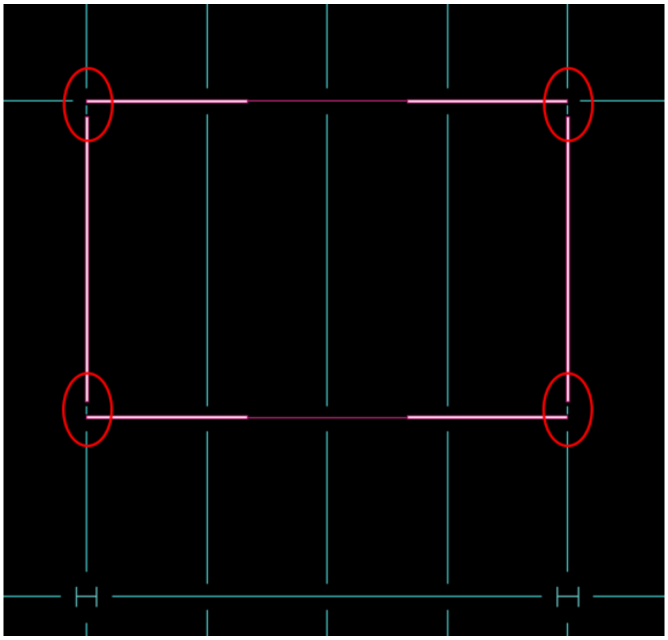


Figure 2.2 - 4 | Core Wall Modeling Technique

The gravity system was included in the lateral model. This was done partially because RAM Structural System requires a gravity system to perform any lateral analysis. However, the AISC *Seismic Design Manual* also requires that all gravity loads be accurately modeled in order to properly account for second-order effects and to accurately capture the distribution of gravity load effects on vertical force resisting members. This can be fulfilled by accurately modeling the gravity system with the lateral system in RAM SS. **Figure 2.2 – 5** shows the RAM Model with both lateral and gravity elements.

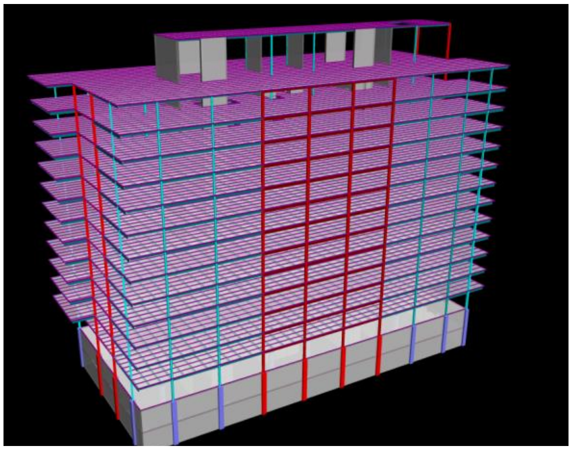


Figure 2.2 - 5 | RAM Lateral Model – Gravity Shown

The foundation walls and concrete columns below grade were modeled to account for the added flexibility the two lower levels add to the shear walls. Although a redesign of these elements was not done, it was important to include the lower levels to obtain an accurate portrayal of the building's overall flexibility. Also, it is important to note that the foundation walls in combination with the rigid diaphragms below grade cause a significant amount of shear reversal in the shear walls at the lower levels. These forces will not be used for design of the shear walls because they will not be accurate.

Lateral System Redesign

Moment Frames

The original lateral system for La Jolla Commons was tested last semester in Technical Report 4. The building was determined to have an extreme torsional irregularity, which caused an increase in lateral forces, special detailing requirements, and other complications. It was the goal of this redesign to eliminate the torsional irregularity by adding moment frames to each façade of the building. The moment frames were initially designed as ordinary moment frames. They were then optimized for strength to be classified as a special moment frames, which require some special seismic detailing. More information about the seismic detailing can be found later in this report. Special moment frames were used because ASCE 7-10 does not allow intermediate or ordinary moment frames to be used on structures taller than 65 feet, for Seismic Design Category D structures. The moment frames are placed as indicated in **Figure 2.2 – 6**.

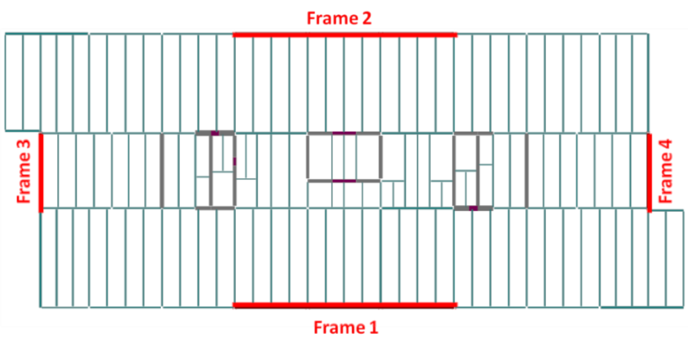


Figure 2.2 - 6 | Moment Frame Numbers and Locations

As stated above, the effects of cracked concrete sections and panel zones were included in this redesign. The effects of panel zones were carefully checked using RAM Frame joint analysis. All columns in the moment frames were designed to be "clean columns." The clean column option for design was selected because it has been shown that it is cheaper to increase column sizes than it is to add web plates and flange stiffeners. This detailing requires significant labor and will increase the project cost and schedule. According to the article *In the Moment* by Victor Shneur, PE, "When possible, consider using a deeper W-Shape to reduce flange forces and possibly eliminate stiffeners at columns. The increase in material weight is typically offset by eliminating stiffeners and using a less expensive/lighter moment connection." The frame joints were verified using RAM to not require web plates or stiffeners, and panel zone shear capacities were verified. This was done for both the standard steel provisions and for the seismic provisions of special moment frames. In addition, seismic provisions verified that the strong-column weak-beam failure mechanism occurs at each joint.

The moment frames were optimized for strength under the controlling load case. The controlling load case for most frame members is $1.367D + 0.5L_p + 1.3E$ as generated by RAM Frame. This load combination includes the effects of vertical earthquake forces, F_v applied to the live load according to IBC 2012, the over-strength factor of 1.3 for seismic loads, and vertical earthquake forces. Strength design was done for standard provisions and then refined for special moment frame seismic provisions.

After all strength and joint optimizations were complete, the finals designs are as follows. For the three bay moment frames, Frames 1 and 2, beams range in size from W24x131 to W24x250, and columns range in size from W14x233 to W14x500. For the single bay moment frames, Frames 3 and 4, beams were sized at W14x145, and columns were sized at W14x370. The moment frame detailing will be discussed later in this report. The final designs of all the moment frames can be found in **Appendix I**.

Lateral Redesign – Shear Walls

The original thicknesses of the special concrete shear walls were used for the analysis and design of the new moment frames. Drift limits were checked before shear wall redesign began, and it was found that the drift values were particularly close to the code limits. As a result, the original wall thicknesses will not be changed; instead, the reinforcing design will be modified for the new loads.

The existing shear walls were checked under seismic loading from ASCE 7-05 in Technical Report 4; however, ASCE 7-10 will be used for this redesign, seismic forces are reduced due to a lighter weight structure, and a more complete list of load combinations has been generated by RAM Frame. Therefore, a strength verification of the original shear wall designs was performed. Shear Wall U was selected for this analysis. This was done so that it could be compared to the same check performed on Shear Wall U under the ASCE 7-05 seismic loads. **Figure 2.2 – 7** shows the location of Shear Wall U in plan. The original design for Shear Wall U is 18 inches thick with #6's @ 9" horizontally and #6's @ 12" vertically, in two curtains.

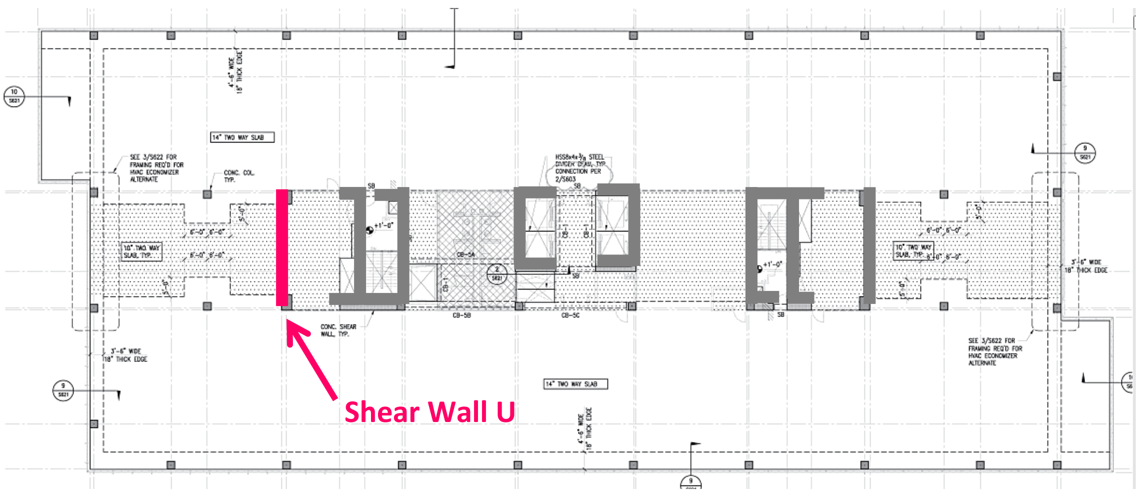


Figure 2.2 - 7 | Location of Shear Wall U

The strength of this wall was checked at Level 2 under the new seismic loads. This is the first level above the foundation levels which induce shear reversal; therefore, the shear forces at Level 2 will be accurate. The wall was found to meet the required strength conditions of ordinary and special reinforced concrete shear walls in accordance with § 21.9 of ACI 318-11. The reinforcement of the shear walls also meets the requirements of special reinforced shear walls as outlined in § 21.9.2.1 and § 21.9.2.2 of ACI 318-11. This verifies that, even under the new concrete code, the existing shear walls can be classified as special. To view these checks in detail, see **Appendix J**.

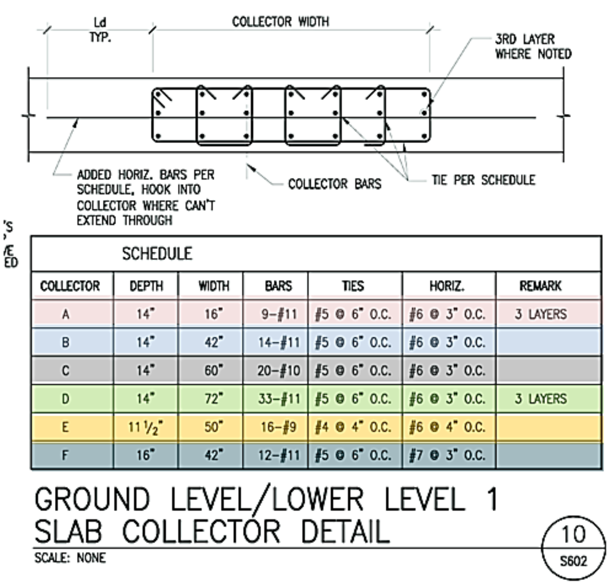
It is important to note that the existing shear wall was found to be "overdesigned" for strength considerations according to ACI 318-11 under the seismic loads calculated using ASCE 7-10. For example, ΦV_n for the wall was found to be 1534 kip, but, V_u was only found to be 872 kip. Also, the shear wall was found to no longer require two curtains of reinforcing as the current design specifies.

As mentioned before, the shear wall thicknesses will not be reduced. Modifications to the shear wall designs have only been done to the reinforcing. Reducing the thickness of the walls will negatively affect the building drift, which is already reaching code limits. Therefore, the reinforcing layout was redesigned to meet the minimum reinforcement ratio, strength, and spacing requirements for special reinforced shear walls under the new loads. The modified design calls for a single curtain of #6 @ 9" vertically and #6 @ 9" horizontally. See **Appendix J** for the design calculations.

A similar design process should be performed for all of the existing shear walls in the building. If it is desired to reduce the thicknesses of the shear walls, it will be necessary to increase the stiffness of the moment frames or to design additional moment frames to control building drift under seismic loads.

Collector Beams

Collector beams will still be required at the lower levels of the building structure, even though the seismic forces have been reduced. The forces that must be transmitted to the lateral elements in the North-South direction are too high for the diaphragm to direct the loads into the LFRS elements. The LFRS elements in the North-South direction are not continuous. In order for them to work together and share the lateral load based on relative stiffness, collector elements must be utilized. As a result, it is recommended that the existing collector elements be redesigned using ACI 318-11. The collector locations, however, should not change. See **Figure 2.2 – 8** and **Figure 2.2 – 9** for the existing collector beam designs and locations, respectively.



SCHEDULE

COLLECTOR	DEPTH	WIDTH	BARS	TIES	HORIZ.	REMARK
A	14"	16"	9-#11	#5 @ 6" O.C.	#6 @ 3" O.C.	3 LAYERS
B	14"	42"	14-#11	#5 @ 6" O.C.	#6 @ 3" O.C.	
C	14"	60"	20-#10	#5 @ 6" O.C.	#6 @ 3" O.C.	
D	14"	72"	33-#11	#5 @ 6" O.C.	#6 @ 3" O.C.	3 LAYERS
E	11 1/2"	50"	16-#9	#4 @ 4" O.C.	#6 @ 4" O.C.	
F	16"	42"	12-#11	#5 @ 6" O.C.	#7 @ 3" O.C.	

Figure 2.2 - 8 | Original Collector Beam Designs

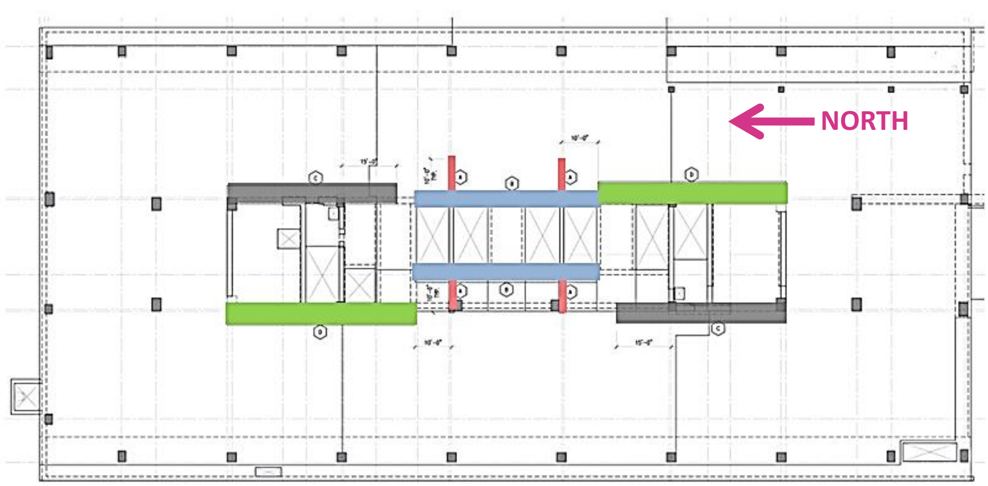


Figure 2.2 - 9 | Original Collector Beam Locations – LL1

New Lateral System Drift, Torsion, and Stability Analysis

The next step in the design process was to verify that the newly modified lateral system would control the drift and torsional irregularity associated with La Jolla Commons Phase II Office Tower under wind and seismic loads. The stability coefficient, θ, is also verified for each of the seismic load cases. This verifies the control of P-delta effects.

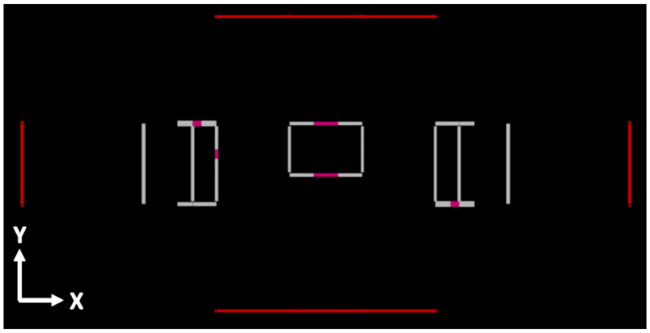


Figure 2.2 - 10 | X and Y Direction Definition

Drift Analysis

First, the drift of the building under ASCE 7-10 wind loads was determined using RAM Frame. These drifts were then checked against H/400, which is and accepted industry standard for wind serviceability. All applicable wind load cases were analyzed, and the highest resulting deflections were found. As expected, the wind deflections all met the H/400 industry standard for both the X and Y load cases. **Figure 2.2 – 10** indicates which directions in the model have been labeled as X and Y. The results of the wind deflection check can be viewed in **Table 2.2 – 5**.

Table 2.2 – 5 | Wind Displacement Determination

Load Case	X - Deflection (in)	Y - Deflection (in)	L/400 (in)	Pass/Fail?
Wind_ASCE710_1_X	1.91	0.00	5.940	Pass
Wind_ASCE710_1_Y	0.00	2.11	5.940	Pass
Wind_ASCE710_2_X+E	1.43	-0.01	5.940	Pass
Wind_ASCE710_2_X-E	1.43	0.01	5.940	Pass
Wind_ASCE710_2_Y+E	0.01	1.68	5.940	Pass
Wind_ASCE710_2_Y-E	-0.01	1.49	5.940	Pass
Wind_ASCE710_3_X+Y	1.43	1.58	5.940	Pass
Wind_ASCE710_3_X-Y	1.43	-1.58	5.940	Pass
Wind_ASCE710_4_X+Y_CW	1.07	1.11	5.940	Pass
Wind_ASCE710_4_X+Y_CCW	1.08	1.27	5.940	Pass
Wind_ASCE710_4_X-Y_CW	1.07	-1.26	5.940	Pass
Wind_ASCE710_4_X-Y_CCW	1.08	-1.10	5.940	Pass

Next, drift under seismic loads was determined and checked against the requirements of ASCE 7-10 § 12.8.6. Using Table 12.12-1 of ASCE 7-10, it was determined that for La Jolla Commons the story drift limit is $0.020h_{sx}$. The elastic story drift taken from RAM Frame was modified as required by equation 12.12-15 of ASCE 7-10, where $C_d = 5$ for special concrete shear walls with special moment frames (not behaving as a dual system). **Figure 2.2 – 11** shows the method used to determine drift values, using ASCE 7-10 § 12.8.6.

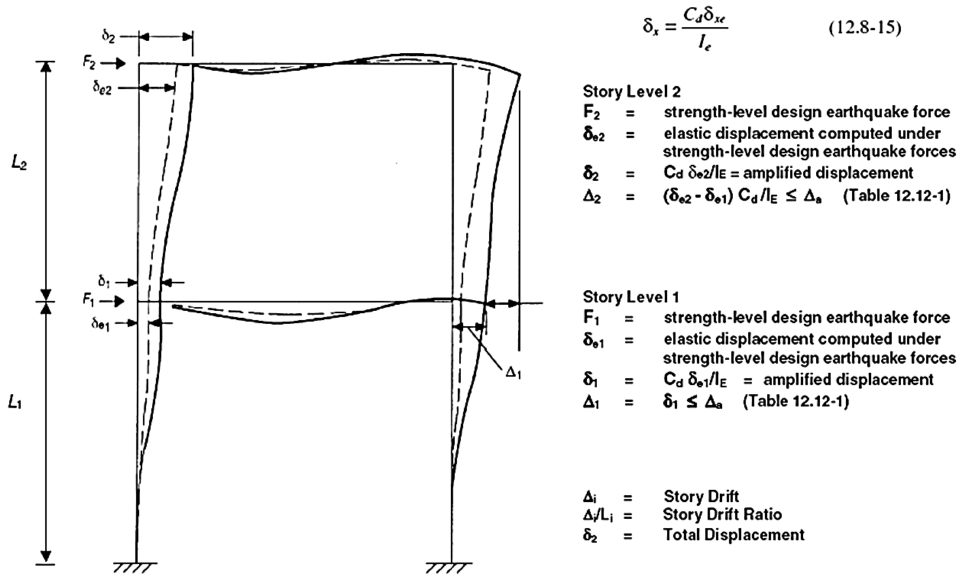


Figure 2.2 - 11 | **Story Drift Determination – ASCE 7-10 Figure 12.8-2**

The controlling load case for drift in the X-direction was found to be EX + EXT (x-direction seismic forces with 5% eccentricity). The controlling case in the Y-direction was EY + EYT (y-direction seismic forces with 5% eccentricity). As can be seen in **Table 2.2 – 6**, the amplified story drifts were all found to be within the allowable drift limits. The overall building deflection in the X-direction under the controlling load case was found to be 36.32 inches. The overall building deflection in the Y-direction under the controlling load case was found to be 12.46 inches. The story drifts can be viewed below.

Table 2.2 – 6 | Seismic Story Drift Check

Level	Level Height (ft)	$C_d*\delta_x$		Allowable Drift (in)	Pass/Fail?	
		X-Direction	Y-Direction		X-Direction	Y-Direction
PH Roof	24.33	5.15	2.04	5.839	Pass	Pass
PH	14.5	3.08	1.22	3.480	Pass	Pass
13	14	2.83	1.02	3.360	Pass	Pass
12	14	2.87	1.01	3.360	Pass	Pass
11	14	2.89	1.00	3.360	Pass	Pass
10	14	2.89	0.97	3.360	Pass	Pass
9	14	2.85	0.93	3.360	Pass	Pass
8	14	2.76	0.88	3.360	Pass	Pass
7	14	2.62	0.82	3.360	Pass	Pass
6	14	2.41	0.74	3.360	Pass	Pass
5	14	2.13	0.64	3.360	Pass	Pass
4	14	1.76	0.53	3.360	Pass	Pass
3	14	1.29	0.40	3.360	Pass	Pass
2	15	0.78	0.27	3.600	Pass	Pass
Overall Displacement=		36.32	12.46			

Torsional Analysis

The next item to be checked was the torsional properties of the building structure under seismic loads. As previously mentioned, the main purpose of the new moment frames was to control the torsional irregularity associated with the original lateral system. As defined by Table 12.3-1 of ASCE 7-10, there are two types of torsional irregularities – regular and extreme. The formal definition of each of these horizontal structural irregularities can be viewed in **Figure 2.2 – 12**.

Table 12.3-1 Horizontal Structural Irregularities

Type	Description	Reference Section	Seismic Design Category Application
1a.	**Torsional Irregularity:** Torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	D, E, and F B, C, D, E, and F C, D, E, and F C, D, E, and F D, E, and F B, C, D, E, and F
1b.	**Extreme Torsional Irregularity:** Extreme torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1 12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	E and F D B, C, and D C and D C and D D B, C, and D

Figure 2.2 - 12 | Horizontal Structural Irregularities – ASCE 7-10 Table 12.3-1

The torsional analysis was done using two points at either end of the structure – Point A and Point B as shown in **Figure 2.2 – 13**. These two points were then used to calculate δ_{avg} and δ_{max}, which were compared to determine if a torsional irregularity or an extreme torsional irregularity existed for both the X and Y directions. It was determined that neither existed in the new structure, in neither the X nor Y direction. See **Table 2.2 – 7** for the analysis performed in the X-direction and **Table 2.2 – 8** for the Y-direction.

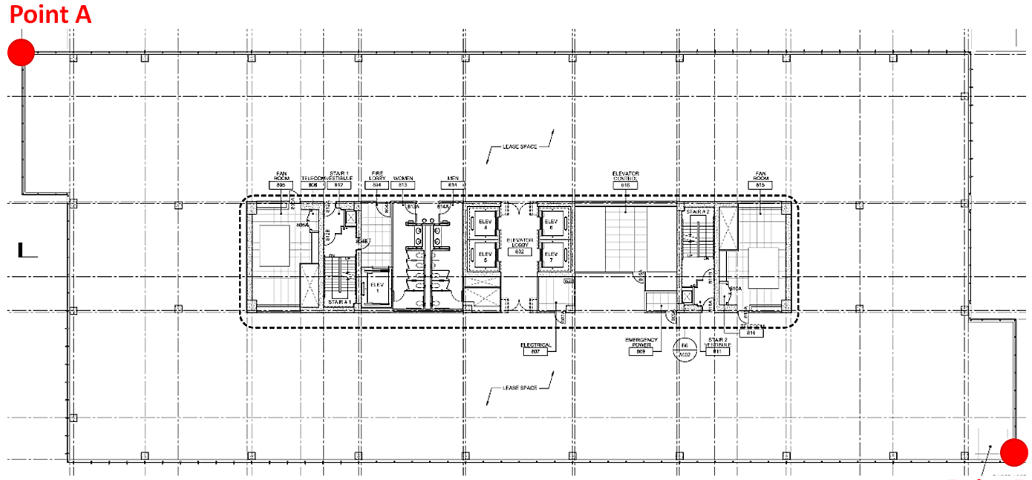


Figure 2.2 - 13 | Points A and B Required for Torsional Analysis

Table 2.2 – 7 | Check for Torsional Irregularities X Direction					
Level	δ_A	δ_B	δ_{avg}	δmax	Does a torsional irregularity exist?
PH	0.6268	0.6269	0.63	0.6269	No
Level 13	0.5762	0.5764	0.58	0.5764	No
Level 12	0.5846	0.5847	0.58	0.5847	No
Level 11	0.5887	0.5888	0.59	0.5888	No
Level 10	0.5877	0.5878	0.59	0.5878	No
Level 9	0.5788	0.5789	0.58	0.5789	No
Level 8	0.5614	0.5615	0.56	0.5615	No
Level 7	0.5320	0.5321	0.53	0.5321	No
Level 6	0.4901	0.4902	0.49	0.4902	No
Level 5	0.4330	0.4331	0.43	0.4331	No
Level 4	0.3582	0.3582	0.36	0.3582	No
Level 3	0.2116	0.2637	0.24	0.2637	No

Table 2.2 – 8 | Check for Torsional Irregularities Y Direction					
Level	δ_A	δ_B	δ_{avg}	δmax	Does a torsional irregularity exist?
PH	0.3206	0.3086	0.31	0.32	No
Level 13	0.2910	0.2809	0.29	0.29	No
Level 12	0.2891	0.2800	0.28	0.29	No
Level 11	0.2847	0.2767	0.28	0.28	No
Level 10	0.2771	0.2702	0.27	0.28	No
Level 9	0.2657	0.2599	0.26	0.27	No
Level 8	0.2513	0.2470	0.25	0.25	No
Level 7	0.2329	0.2300	0.23	0.23	No
Level 6	0.2101	0.2084	0.21	0.21	No
Level 5	0.1826	0.1820	0.18	0.18	No
Level 4	0.1505	0.1506	0.15	0.15	No
Level 3	0.1280	0.1139	0.12	0.13	No

This analysis verifies that the additional perimeter moment frames were able to control the torsional irregularity associated with the original building structure. The frames are stiff enough to resist enough torsional shears to limit the building rotation. The frames are also a significant distance away from the center of rigidity to help control torsional effects. Therefore, it can be said that the frames were useful in controlling the torsional irregularities and eliminate the need to amplify the seismic forces.

Stability Coefficient Analysis

P-delta effects can have a major impact on moment frame design. Although RAM Structural System will verify that P-delta effects are under control, the stability coefficient was analyzed to verify that the P-delta effects were properly accounted for.

The stability coefficient was calculated as required by ASCE 7-10 § 12.8.7. The stability coefficient is calculated using Equation 12.8-16 below and is then compared to the maximum allowable value found using Equation 12.8-17 below. In these equations, Cd = 5 and β = 1.0.

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d} \qquad (12.8\text{-}16)$$

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25 \qquad (12.8\text{-}17)$$

These values were then compared using an Excel spreadsheet. All stability coefficients were found to be below the maximum allowed values for all applicable load combinations. **Table 2.2 – 9** shows a sample of the stability coefficient comparison table for one load case.

Table 2.2 – 9 | ASCE 7-10 Stability Coefficients

RAM Frame
LOAD CASE: ELF EQ
Type : EQ_ASCE710_X_+E_F

Level	θx	θy	θx/(1+θx)	θy/(1+θy)	θx max	θy max	θx Ok?	θx Ok?
Roof	0.092	0	0.084	0	0.1	0.1	Pass	Pass
PH	0.07	0	0.065	0	0.1	0.1	Pass	Pass
Level 13	0.072	0	0.067	0	0.1	0.1	Pass	Pass
Level 12	0.077	0	0.072	0	0.1	0.1	Pass	Pass
Level 11	0.082	0	0.076	0	0.1	0.1	Pass	Pass
Level 10	0.088	0	0.081	0	0.1	0.1	Pass	Pass
Level 9	0.093	0	0.085	0	0.1	0.1	Pass	Pass
Level 8	0.095	0	0.086	0	0.1	0.1	Pass	Pass
Level 7	0.097	0	0.088	0	0.1	0.1	Pass	Pass
Level 6	0.097	0	0.088	0	0.1	0.1	Pass	Pass
Level 5	0.093	0	0.085	0	0.1	0.1	Pass	Pass
Level 4	0.084	0	0.078	0	0.1	0.1	Pass	Pass
Level 3	0.068	0	0.064	0	0.1	0.1	Pass	Pass
Level 2	0.038	0	0.036	0	0.1	0.1	Pass	Pass

Overturning Moment and Impact on Foundations

Overturning Moment

The building overturning moment and impact on foundations were next to be analyzed. The controlling load combination for overturning was found to be 0.9D + 1.0E. The maximum overturning moment was caused by earthquake loading in the Y-Direction. The resulting overturning moment was found to be $M_{overturning}$ = 381,110 ft-kip, rotating about the X axis of the building plan. The resisting moment for this overturning moment was found to be 3,170,446 ft-kip, using a moment arm of 57.5 and the total building weight. See **Figure 2.2 – 14** for the moment arm location and moment direction. A factor of safety of 0.67 was applied to the resisting moment in accordance with the IBC 2012. Even with the factor of safety applied, the overturning moment under the worst case seismic loading was well below the resisting moment for the building.

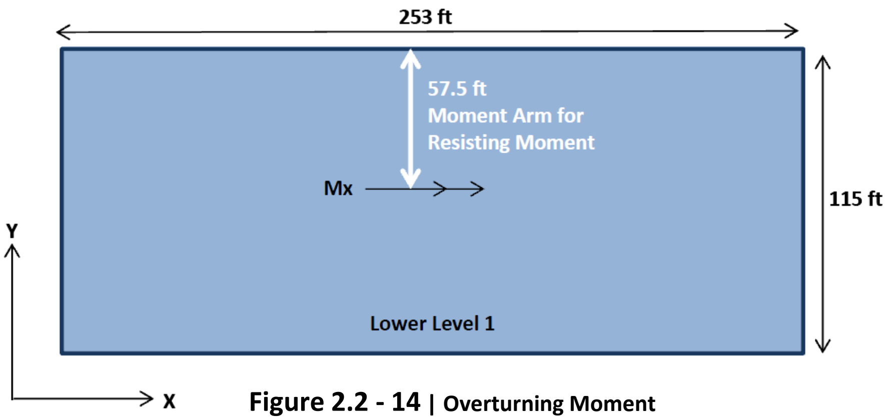


Figure 2.2 - 14 | **Overturning Moment**

Impact on Foundations

The original foundation for La Jolla commons Phase II Office Tower is a concrete mat. The original mat ranges in thickness from 4.5 to 6 feet. The foundation is required to withstand the total base shear and total moment associated with the worst case loads. The controlling load combination for the mat foundation design will be 1.2D + 1.0E + L + 0.2S. The mat foundation design will need to be adjusted and verified to accommodate the loads for the new lateral system and the lighter overall building dead load.

For the foundation wall designs, the controlling load combination will be 0.9D + 1.0E + 1.6H. The foundation walls will also need to be redesigned to carry the loads from the four new moment frames added to the building perimeter. Concrete columns or pilasters should be considered as part of the foundation design for the moment frames. The columns or pilasters must be designed to carry the loads to the mat foundation.

As indicated above, seismic loading controls the foundation design. The maximum base shear, V_b, was found to be 3216k. The maximum overturning moment was found to be 381,110 ft-kip. Both of these are a result of seismic loads in the Y-Direction.

See **Appendix K** for the overall stability checks.

Special Moment Frame Detailing

Moment frames have been designed to help resist torsional forces due to earthquake loading. It has been determined that the moment frames must be detailed as special moment frames. Although the frames do not allow the lateral system to be considered dual system in either direction, the moment frames are required to be special by ASCE 7-10 § 12.2.5.5. Intermediate and ordinary moment frames are only allowed to be used in SDC D when the overall building height is no more than 65 feet – La Jolla Commons is 198 feet tall. As a result, the moment frames were designed under the Special Moment Frame Seismic Provisions in RAM Frame, with the columns designed as clean columns. However, additional detailing is required for these moment frames and some of the requirements investigated by RAM Frame will also be discussed.

Special moment frames (SMF) derive their ductility during a seismic event through the flexural yielding of beams, the shear yielding of column panel zones, and the flexural yielding of columns. They allow for high ductility as well as architectural versatility. The lateral displacement is controlled by the flexural stiffness of the framing members and the ability of frame joints to resist rotation. As can be seen from the frame designs in **Appendix I**, many of the framing members are the same size as members below them. This is because SMF are often sized for drift and rotation control rather than strength provisions.

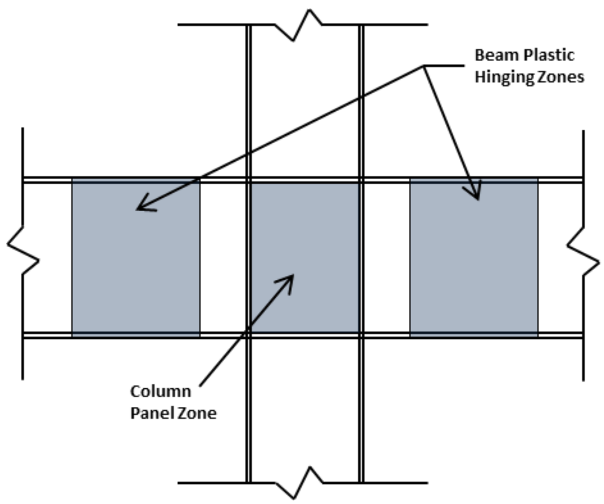


Figure 2.2 - 15 | **SMF Inelastic Deformation Zones**

When designing the beam to column connections, it is important to size members or detail connections to promote inelasticity in the beams and the panel zone, as shown in **Figure 2.2 – 15**. Section K2 of the AISC *Seismic Provisions* requires that at least 75% of the frame deformation must occur at beam hinge locations. This can be done through the use of continuity plates, doubler plates, and increasing the column sizes to encourage hinging away from the column face. As stated by Section 4.3 of the AISC *Seismic Design Manual*, there are two common methods used to force plastic hinging of the beam away from the column face. One way is to specially detail the column-to-beam connection to create enough toughness in the connection to force the elasticity into the beam. Another method is to use a reduced beam section (RBS) connection a short distance away from the column face. The reduced beam section properties will force yielding to occur at this location.

There are two different ways to design for panel zone behaviors in SMF. Generally strong or balanced panel zones are required. In a strong panel zone, the panel strength is greater than the surrounding framing members so that most of the inelastic deformation occurs in the members. A balanced panel allow for a similar inelastic deformation in both the panel zone and the surrounding framing members. AISC Seismic Provisions leads to the design of SMF that behave as either strong or balanced panel. Weak panel designs are possible but are allowed only in intermediate and ordinary moment frames.

One of the most important details of SMF design is the strong column-weak beam concept. The AISC *Seismic Provisions* requires the following equation to be true:

$$\frac{\sum M_{pc}^{*}}{\sum M_{pb}^{*}} > 1.0\ (Provisions\ Eq.E3)$$

$\sum M_{pc}^{*}$ refers to the combined flexural strengths of all the columns coming into a particular joint. $\sum M_{pb}^{*}$ refers to that of the beams coming into the same joint at the plastic hinge locations. The goal of this provision is not to eliminate column yielding; the idea is to eliminate the development of a story mechanism which would cause an entire story to collapse. Beam mechanisms, forced by strong column-weak beam configurations are the preferred mechanism. **Figure 2.2 – 16** illustrates these mechanisms.

This was a factor that was verified by the RAM Frame Seismic Joint checks and seismic strength checks. A few joints were also spot checked to verify that this condition is met. All frames will experience strong column – weak beam behavior, not story mechanism behavior, as required by AISC *Seismic Provisions* § E3.2.

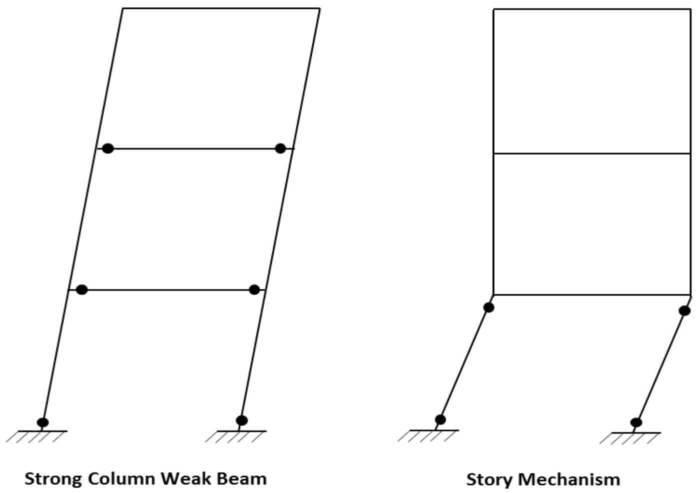


Figure 2.2 - 16 | **Story vs. Beam Mechanism**

SMF are required by AISC to meet the requirements for *highly ductile* members. Therefore, stability bracing of beams must satisfy the requirements for *highly ductile* members in section D1.2b of the AISC *Seismic Provisions*. There are requirements for the lateral bracing of the entire beam length and additional requirements for plastic hinge locations. Some provisions are as follows: (1) Both flanges of beams shall be laterally braced or the beam cross section shall be torsionally braced, (2) Bracing of highly ductile beam members shall have a maximum spacing of $L_b = 0.086 r_y E/F_y$. Additional special bracing at plastic hinge locations must meet the requirements of Section D.2c. This section lists the requirements for the spacing and required strength of the lateral bracing of the plastic hinge regions. Each of these items was verified using the RAM Frame Module on SMF Seismic Provision analysis.

Additional requirements for *highly ductile* members in SMF are as follows. One such provision is that members shall have flanges continuously connected to the web or webs. Members must also not exceed the width-to-thickness ratios listed in Table D1.1 in the AISC *Seismic Provisions*. For example, for I-shaped sections the maximum width-to-thickness ratio, $\lambda_{hd} = 0.30\sqrt{E/F_y}$, must not be exceeded by any member. There are different limiting ratios for different section types. This provision was verified by RAM Frame. Furthermore, no abrupt changes in beam flange area are allowed in plastic hinge regions as well as no flange holes or drilling. These changes will make the determination of the plastic behavior too unpredictable.

Connection design is the next item that requires special detailing. First, groove welds at column splices, welds at column-to-base plate connections, and complete-joint-penetration groove welds of beam flanges and beam webs to columns are must meet the requirements of Section A3.4b of the AISC *Seismic Provisions*. This section lists the mechanical properties of the filler metals for these welds, including required yield and tensile strengths. Beam-to-column connections for seismic force resisting systems (SFRS) are required to accommodate a story drift angle of at least 0.04 radians. Also, the flexural resistance of the connection must be at least $0.8M_p$ of the connected beam at the 0.04 radian story drift angle. In addition, the connections must be proven to conform to the requirements of Section E3.6b by either conforming to a prequalified connection or through performance testing.

Connections are also responsible for handling the shears occurring at column panel zones as shown in **Figure 2.2 – 15**. The shear strength of the panel zone is calculated from the sum of the moments at the column faces. The design shear strength is required to be $\phi_v R_n$, where ϕ_v= 1.0 for LRFD design.

The AISC *Seismic Provisions* also includes specific requirements for the inclusion of doubler plates to increase panel zone thicknesses to meet shear requirements; however, the design of the new special moment frames for La Jolla Commons did not include doubler plates because column sizes were increased until the column web thickness met the shear strength requirements of Section E3.5e. However, a comparison of the clean column option versus a doubler plate design is shown in **Figure 2.2 – 17**. This section shows a symmetrical doubler plate layout with fillet welds as required by Section E3.6e (3).

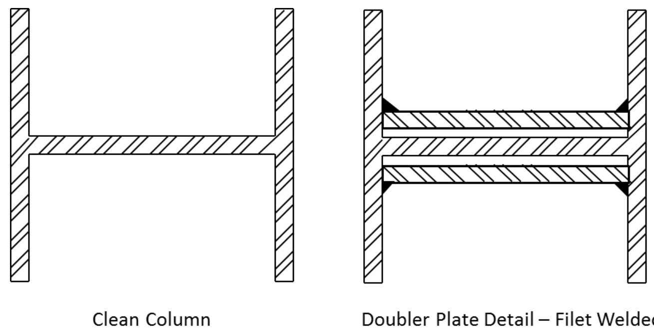


Figure 2.2 - 17 | Clean Column vs Doubler Plate

Continuity plates are another common item used in detailing of SMF. Yet again, the moment frames for La Jolla Commons were designed to not require continuity plates. This is possible when the beam flange is welded to a wide-flange column, if the column is of sufficient thickness, as determined by Equations E3-8 and E3-9 in the AISC *Steel Seismic Provisions*. Although continuity plates were not required for the design of the moment frames for this project, **Figure 2.2 – 18** shows what the continuity plate detail would look like if required.

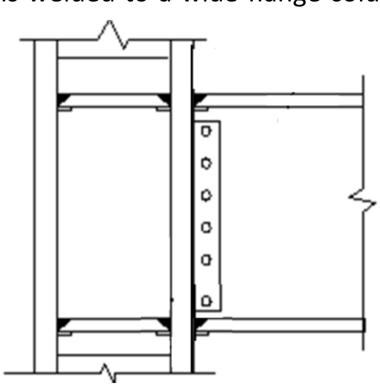


Figure 2.2 - 18 | Continuity Plate Detail

The final item to be addressed is column splicing. As was discussed previously in this report, the columns in the new steel redesign are spliced every two stories. RAM Frame does not analyze columns splices or connections so these would need to be designed and detailed separately. According to the AISC *Seismic Provisions*, when welds are used to make a splice, they must be complete-joint-penetration groove welds. Bolted connections are permitted but the connection must meet specific flexural and shear strength requirements of Section E3.6g.

2.3 – MAE Requirements

Graduate level coursework was used throughout the design and analysis of the new structural system. AE 530 – Advanced Computer Modeling of Building Structures was utilized in the creation and verification of a RAM Structural System gravity and lateral model. Also, material from AE 538 – Earthquake Resistant Design for Buildings was used to design and determine required detailing for the new steel moment frames, as well as, the verification and design modification of the original shear walls. In addition, an investigation was done on the vibrations of the new steel floor system. This required expansion of knowledge beyond completed undergraduate and graduate coursework.

AE 538 – Earthquake Resistant Design for Buildings

> *"The objective of this course is to provide students with an understanding of damaging aspects of earthquake phenomenon and how to analyze and design buildings to satisfy model building code provisions." – Penn State Department of Architectural Engineering*

AE 530 – Advanced Computer Modeling of Building Structures

> *"The course is designed to provide students with the ability to create computer models representative of actual building response and in line with prevalent modeling techniques implemented using commercial structural analysis software. Primary objectives include developing an understanding of the process used by computers to solve structural systems, with emphasis placed on the use of computer models in the analysis and design process to satisfy building code requirements." – Penn State Department of Architectural Engineering*

Chapter 3 – Architectural Breadth

The architecture of La Jolla Commons will be impacted by the change in building structure from concrete to steel. Although the original column and wall locations were maintained, the building height will be impacted along with the building fire protection requirements. The chapter of the report analyzes the building height impact. It also looks into the fire protection requirements for the new structure. Designs for the required fire-resistance ratings have also been determined.

3.1 – Floor-to-Ceiling Height and Building Height Analysis

The change from a concrete structure to a steel structure will have a significant impact on the overall height of the building. La Jolla Commons Phase II Office Tower's current height is 198'-8", which is firmly restricted by a flight path controlled by the Federal Aviation Administration. As a result, an increase in building height would be unacceptable. As expected, the steel structure was, on average, about 15 inches deeper per level than the original concrete flat plate slab. Two options are to be considered. Either the original building height will be maintained and the floor-to-ceiling heights will decrease, or the original floor-to-ceiling heights will be maintained, causing the building height to increase, ignoring the height restriction. **Table 3.1 – 1** shows a comparison of the depth of the structure at each level for the concrete and steel systems, and **Table 3.1 – 2** shows the difference in the cumulative floor depths of the two systems.

Table 3.1 – 1 | Structural Depths

Level	Deepest Member	Depth of Deepest Member (in)	Total Slab Depth (in)	Depth of Structure at Level (in)	Original Depth of Structure (in)
2	W24x76	23.9	5.75	29.65	14
3	W24x76	23.9	5.75	29.65	14
4	W24x76	23.9	5.75	29.65	14
5	W24x76	23.9	5.75	29.65	14
6	W24x76	23.9	5.75	29.65	14
7	W24x76	23.9	5.75	29.65	14
8	W24x76	23.9	5.75	29.65	14
9	W24x84	24.1	5.75	29.85	14
10	W24x84	24.1	5.75	29.85	14
11	W24x84	24.1	5.75	29.85	14
12	W24x84	24.1	5.75	29.85	14
13	W24x84	24.1	5.75	29.85	14
PH	W24x62	23.7	5.75	29.45	14
Roof	W16x26	15.7	5.75	21.45	14

Table 3.1 –2 | Cumulative Structural Floor Depths

Concrete Floor Structure Total Depth	196	in
Steel Floor Structure Total Depth	407.7	in
Difference in Structural Depth	211.7	in
	17.64	ft

It obviously is unfavorable to increase the overall height of the building to maintain ceiling heights; therefore, an analysis was done to determine the impact of the decreased ceiling heights. The office space finishes are unknown; however, typical office conditions were assumed for this analysis. A drop ceiling was assumed. Also, it is known that there is a 12 inch raised floor system on each level of the building's lease space; the raised floor creates a plenum for the under-floor air distribution system. For the original concrete building structure, it was assumed that the drop ceiling hung 18 inches below the slab in order to conceal concrete beams. For the redesigned steel structure, the ceiling is assumed to hang 36 inches below the structural slab in order to conceal the floor structure. The raised floor-to-ceiling heights for the steel and concrete structures are compared in **Table 3.1 – 3** and **Table 3.1 – 4** respectively, assuming the original building height of 198 feet is to be maintained.

Table 3.1-3 | *Steel Floor System* - Building Height at 198 ft

Story	Original Floor To Floor Height (ft)	Raised Floor System (in)	Depth of Floor Structure (in)	Drop Ceiling Required to Enclose Structure (in)	Floor to Ceiling Height (ft)
2	15.00	12.0	29.7	36.0	9.17
3	13.17	12.0	29.7	36.0	11.00
4	13.17	12.0	29.7	36.0	9.17
5	13.17	12.0	29.7	36.0	9.17
6	13.17	12.0	29.7	36.0	9.17
7	13.17	12.0	29.7	36.0	9.17
8	13.17	12.0	29.7	36.0	9.17
9	13.17	12.0	29.9	36.0	9.17
10	13.17	12.0	29.9	36.0	9.17
11	13.17	12.0	29.9	36.0	9.17
12	13.17	12.0	29.9	36.0	9.17
13	13.17	12.0	29.9	36.0	9.17
Penthouse	14.50	0.0	29.5	0.0	14.50
Roof	24.33	0.0	21.5	0.0	24.33

Table 3.1-4 | *Original Concrete Floor System* - Building Height at 198 ft

Level	Original Floor To Floor Height (ft)	Raised Floor System (in)	Depth of Floor Structure (in)	Drop Ceiling Required to Enclose Structure (in)	Floor to Ceiling Height (ft)
2	15.00	12.0	14.0	18.0	12.50
3	13.17	12.0	14.0	18.0	10.67
4	13.17	12.0	14.0	18.0	10.67
5	13.17	12.0	14.0	18.0	10.67
6	13.17	12.0	14.0	18.0	10.67
7	13.17	12.0	14.0	18.0	10.67
8	13.17	12.0	14.0	18.0	10.67
9	13.17	12.0	14.0	18.0	10.67
10	13.17	12.0	14.0	18.0	10.67
11	13.17	12.0	14.0	18.0	10.67
12	13.17	12.0	14.0	18.0	10.67
13	13.17	12.0	14.0	18.0	10.67
Penthouse	14.50	0.0	14.0	0.0	14.50
Penthouse Roof	24.33	0.0	14.0	0.0	24.33

As can be seen, the original floor-to-ceiling height is 10’-8”, and the new floor-to-ceiling height is 9’-2” in the office space. **Figure 3.1 – 1** shows the differences between the original concrete system and new steel system floor-to-ceiling heights.

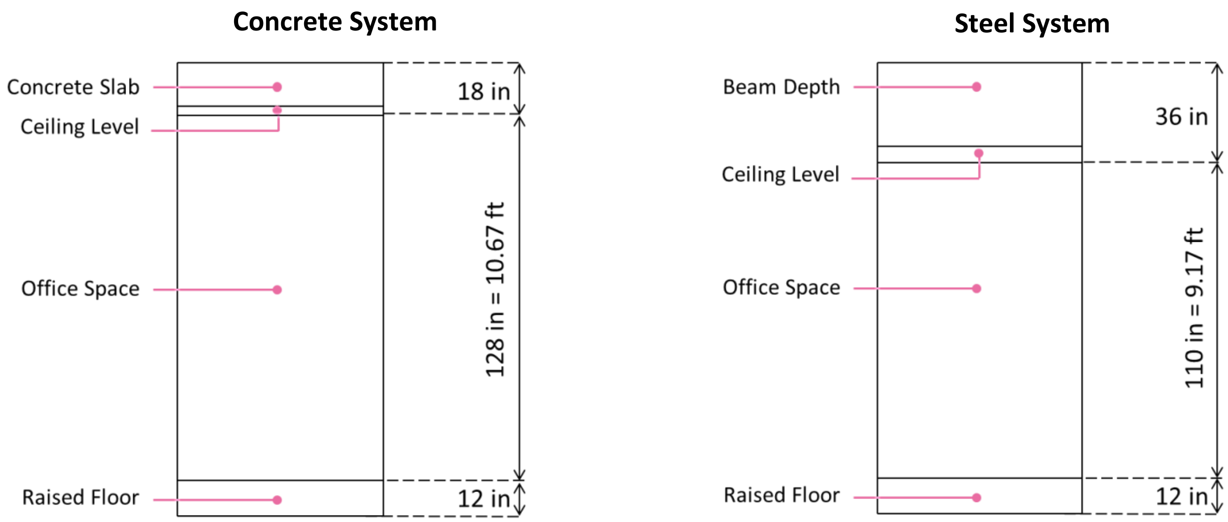


Figure 3.1 – 1 | Floor-to-Ceiling Heights – Concrete vs. Steel

Next, using the floor-to-ceiling heights of the original concrete structure, the new required floor-to-floor heights were determined for the steel structure. In order to maintain the original floor-to-ceiling heights, the building's overall height must increase from 198'-8" to 216'-8". This is an increase in the overall building height of 18'-0". **Table 3.1 – 5** shows this calculation.

Table 3.1-5 | Steel Floor System – Original Floor to Ceiling Heights maintained

Level	Raised Floor To Ceiling Height Required (ft)	Raised Floor System (in)	Depth of Floor Structure (in)	Drop Ceiling Required to Enclose Structure (in)	New Floor to Floor Height (ft)
2	12.5	12.0	29.7	36.0	16.5
3	10.7	12.0	29.7	36.0	14.7
4	10.7	12.0	29.7	36.0	14.7
5	10.7	12.0	29.7	36.0	14.7
6	10.7	12.0	29.7	36.0	14.7
7	10.7	12.0	29.7	36.0	14.7
8	10.7	12.0	29.7	36.0	14.7
9	10.7	12.0	29.9	36.0	14.7
10	10.7	12.0	29.9	36.0	14.7
11	10.7	12.0	29.9	36.0	14.7
12	10.7	12.0	29.9	36.0	14.7
13	10.7	12.0	29.9	36.0	14.7
Penthouse	14.5	0.0	29.5	0.0	14.5
PH Roof	24.3	0.0	21.5	0.0	24.3
				New Building Height	216.7
				Original Building Height	198.7
				Increase in Building Height	18.00

As a result of this significant increase in building height, it has been decided that the original building height should be maintained in order to agree with the FAA height restriction. Although the floor-to-ceiling heights will decrease to 9'-2" in the office spaces, this is not a significant architectural change or concern. According to several design guides and industry standards, office spaces typically have a floor-to-ceiling height of 8'-6" to 9'-0". Therefore, the office space, following precedent office buildings, will still maintain a comfortable and functional atmosphere.

3.2 – Fire Protection Analysis and Design

General Information

La Jolla Commons Phase II Office Tower was originally an entirely concrete structure; as such, the structure had inherent fire-resistive abilities. Changing the building structure to steel has the disadvantage of requiring some sort of fire protection for the building's structural elements. An analysis of IBC 2012 has been performed to determine the requirements for the new structure's fire protection. Also, designs for the fire protection of the building elements have been selected and illustrated, utilizing the Underwriter Laboratories' approved fire protection details. The original building design featured a full automatic sprinkler system. This system will remain in the new building design.

The first step in the analysis was to determine the requirements for the fire protection of the structural elements based on the building height and use. The Use and Occupancy Classification is Business Group B, which is defined as typical office buildings. As shown in **Figure 3.2 – 1**, in order to construct a 13 story, 198 foot tall, and 32,085 square foot per story office building, Type 1A construction must be utilized, according to Table 503 of IBC 2012. However, according to Section 403.2.2.1, because the height of LJC II is less than 420 feet, the fire resistance ratings of the building elements in Type 1A construction are able to be reduced to the minimum fire ratings for the building elements in Type 1B. This excludes columns which will remain Type 1A construction.

TABLE 503
ALLOWABLE BUILDING HEIGHTS AND AREAS[a, b]
Building height limitations shown in feet above grade plane. Story limitations shown as stories above grade plane.
Building area limitations shown in square feet, as determined by the definition of "Area, building," per story

GROUP		TYPE OF CONSTRUCTION								
		TYPE I		TYPE II		TYPE III		TYPE IV	TYPE V	
		A	B	A	B	A	B	HT	A	B
	HEIGHT (feet)	UL	160	65	55	65	55	65	50	40
		STORIES(S) AREA (A)								
A-1	S	UL	5	3	2	3	2	3	2	1
	A	UL	UL	15,500	8,500	14,000	8,500	15,000	11,500	5,500
A-2	S	UL	11	3	2	3	2	3	2	1
	A	UL	UL	15,500	9,500	14,000	9,500	15,000	11,500	6,000
A-3	S	UL	11	3	2	3	2	3	2	1
	A	UL	UL	15,500	9,500	14,000	9,500	15,000	11,500	6,000
A-4	S	UL	11	3	2	3	2	3	2	1
	A	UL	UL	15,500	9,500	14,000	9,500	15,000	11,500	6,000
A-5	S	UL	UL	UL	UL	UL	UL	UL	UL	UL
	A	UL	UL	UL	UL	UL	UL	UL	UL	UL
B	S	UL	11	5	3	5	3	5	3	2
	A	UL	UL	37,500	23,000	28,500	19,000	36,000	18,000	9,000
E	S	UL	5	3	2	3	2	3	1	1
	A	UL	UL	26,500	14,500	23,500	14,500	25,500	18,500	9,500
F-1	S	UL	11	4	2	3	2	4	2	1
	A	UL	UL	25,000	15,500	19,000	12,000	33,500	14,000	8,500
F-2	S	UL	11	5	3	4	3	5	3	2
	A	UL	UL	37,500	23,000	28,500	18,000	50,500	21,000	13,000
H-1	S	1	1	1	1	1	1	1	1	NP
	A	21,000	16,500	11,000	7,000	9,500	7,000	10,500	7,500	NP
H-2	S	UL	3	2	1	2	1	2	1	1
	A	21,000	16,500	11,000	7,000	9,500	7,000	10,500	7,500	3,000
H-3	S	UL	6	4	2	4	2	4	2	1
	A	UL	60,000	26,500	14,000	17,500	13,000	25,500	10,000	5,000
H-4	S	UL	7	5	3	5	3	5	3	2
	A	UL	UL	37,500	17,500	28,500	17,500	36,000	18,000	6,500
H-5	S	4	4	3	3	3	3	3	3	2
	A	UL	UL	37,500	23,000	28,500	19,000	36,000	18,000	9,000
I-1	S	UL	9	4	3	4	3	4	3	2
	A	UL	55,000	19,000	10,000	16,500	10,000	18,000	10,500	4,500
I-2	S	UL	4	2	1	1	NP	1	1	NP
	A	UL	UL	15,000	11,000	12,000	NP	12,000	9,500	NP
I-3	S	UL	4	2	1	2	1	2	2	1
	A	UL	UL	15,000	10,000	10,500	7,500	12,000	7,500	5,000
I-4	S	UL	5	3	2	3	2	3	1	1
	A	UL	60,500	26,500	13,000	23,500	13,000	25,500	18,500	9,000

Figure 3.2 - 1 | **Required Construction Types, IBC 2012 Table 503**

According to Table 601 of IBC 2012, shown in **Figure 3.2 – 2**, the required fire-resistance rating for the primary structural elements of Type 1A construction is 3 hours. For Type 1B construction, the rating is 2 hours. Therefore, the columns will be Type 1A construction and will, therefore, require a 3 hour rating. The floor framing members will require a 2 hour rating in accordance with Type 1B construction. **Table 3.2 – 1** shows the summary of the required fire-ratings for the structural members.

TABLE 601
FIRE-RESISTANCE RATING REQUIREMENTS FOR BUILDING ELEMENTS (HOURS)

BUILDING ELEMENT	TYPE I		TYPE II		TYPE III		TYPE IV	TYPE V	
	A	B	A[d]	B	A[d]	B	HT	A[d]	B
Primary structural frame[g] (see Section 202)	3[a]	2[a]	1	0	1	0	HT	1	0
Bearing walls Exterior[f, g] Interior	 3 3[a]	 2 2[a]	 1 1	 0 0	 2 1	 2 0	 2 1/HT	 1 1	 0 0
Nonbearing walls and partitions Exterior	See Table 602								
Nonbearing walls and partitions Interior[e]	0	0	0	0	0	0	See Section 602.4.6	0	0
Floor construction and associated secondary members (see Section 202)	2	2	1	0	1	0	HT	1	0
Roof construction and associated secondary members (see Section 202)	$1^1/_2$[b]	1[b,c]	1[b,c]	0[c]	1[b,c]	0	HT	1[b,c]	0

For SI: 1 foot = 304.8 mm.

a. Roof supports: Fire-resistance ratings of primary structural frame and bearing walls are permitted to be reduced by 1 hour where supporting a roof only.

b. Except in Group F-1, H, M and S-1 occupancies, fire protection of structural members shall not be required, including protection of roof framing and decking where every part of the roof construction is 20 feet or more above any floor immediately below. Fire-retardant-treated wood members shall be allowed to be used for such unprotected members.

c. In all occupancies, heavy timber shall be allowed where a 1-hour or less fire-resistance rating is required.

d. An approved automatic sprinkler system in accordance with Section 903.3.1.1 shall be allowed to be substituted for 1-hour fire-resistance-rated construction, provided such system is not otherwise required by other provisions of the code or used for an allowable area increase in accordance with Section 506.3 or an allowable height increase in accordance with Section 504.2. The 1-hour substitution for the fire resistance of exterior walls shall not be permitted.

Figure 3.2 - 2 | Fire Resistance Ratings for Building Elements, IBC 2012 Table 601

Table 3.2 – 1 | Required Fire-Resistance Ratings

Element	Construction Type	Required Rating (hours)
Primary Floor Framing Members	Type 1B	2
Secondary Floor Framing Members	Type 1B	2
Structural Columns	Type 1A	3

Designs were selected based on these requirements as well as those of IBC 2012 Section 704. Columns must be individually encased on all four sides for the full column length. This includes the connections to other structural members. When the column extends through the ceiling, the encasement must be continuous from the floor assembly below through the ceiling space to the top of the column. For floor framing members, all members are required to have individual encasement on all sides for the full length of the member. This again includes the connections to other members.

Floor Framing Fire Protection Design

Very little is known about the interior architecture and finishes of La Jolla Commons. The interior build out information is held by LPL Financial and are confidential. Therefore, in order to create the most conservative design that will work with all finishes, the ceiling materials were assumed to not create an adequate fire-resistance rating for the floor framing members. Therefore, other means of protection of floor framing members will be required. This also allows for the educational opportunity of learning how the design of other methods of fire-protection would work.

First, it was decided that the most efficient choice for the fire protection of floor framing members was Sprayed Fire-Resistant Materials (SFRM) without any welded wire fabric, slab reinforcing, or metal lath application. This was selected because it can be done quickly and efficiently as compared to systems using gypsum board or SFRM with metal lath application. With the large number of members that need to be protected, the quickest method needed to be selected.

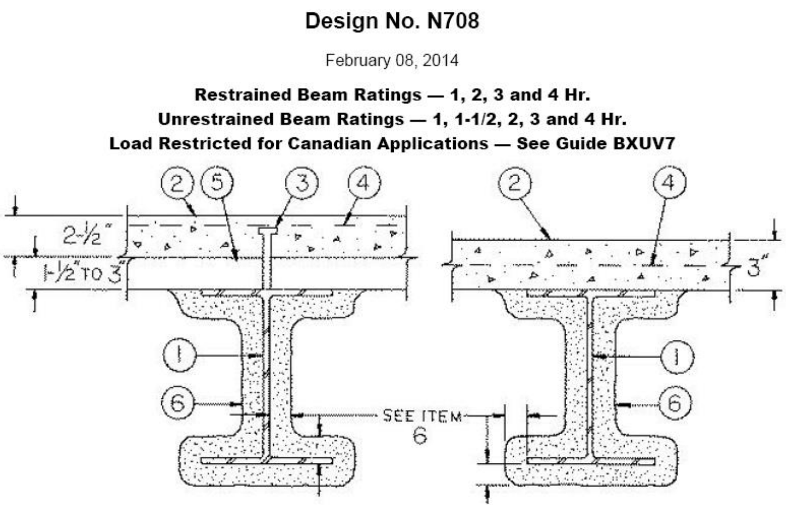


Figure 3.2 - 3 | **UL Design No. N708 – Steel Beam Protection Assembly**

Building framing members must be protected with an *approved* fire-resistance-rated assembly as provided by the Underwriters Laboratories Database. Here, assemblies may be selected based on the system's configuration and required fire-resistance rating. The UL gives many possible assemblies for SFRM protection. However, the possible assemblies were limited to one because of the elimination of welded wire fabric, slab reinforcing, and metal lath application. As a result, Design No. N708 was selected. For this assembly, the minimum beam size is listed as W8x28, shear connectors are optional, and welded wire fabric is optional. **Figure 3.2 – 3** shows a schematic of the assembly. The assembly also requires 1.5 – 3 inch fluted deck with a minimum of 2.5 inches of concrete topping. This requirement is met by the 1.5VLR20 deck with 4.25" LW topping used on each level of the office space.

	Min Thkns In.	
Rating Hr	Restrained Beam Rating Hr	Unrestrained Beam Rating Hr
1	7/16	7/16
1-1/2	1/2	3/4
2	13/16	1
3	1-5/16	1-5/16
4	1-5/8	1-5/8

Figure 3.2 – 4 | **UL Design No. N708 – Required Thickness of Spray Foam**

The next concern is whether to treat the structural members as Restrained or Unrestrained. A professional source was consulted to make this decision – *STRUCTURE Magazine: Fireproofing Steel Structures* from the February 2007 issue. It is recommended by this article to treat members as thermally unrestrained unless the engineer is confident that the member will behave otherwise. The unrestrained assembly will require more spray fire proofing; therefore, if the beam does behave as somewhat restrained, the increase in fire proofing thickness was a conservative assumption. Using the unrestrained assumption, it was determined that 1 inch of spray fire proofing is required to achieve a 2 hour fire rating. This was determined using the tables for N708 as shown in **Figure 3.2 – 4** on the previous page.

However, the minimum beam size required by N708 is a W8x28 which has a weight to heated perimeter (W/D) ratio of 0.819. The minimum beam size used in the structural design was a W12x14 which has a W/D ratio of 0.405. As a result, the above assembly cannot be used without modifying the required spray fire proofing thicknesses for the reduced W/D ratio. The required thickness was modified based on the procedure of Section 722.5.2.1 of IBC 2012, according to Equation 7-17, as shown in **Figure 3.2 – 5**. After using the modification equation, it was determined that a thickness of 1.5 inches is required on a W12x14 member to achieve a 2 hour fire rating. This calculation can be seen in **Table 3.2 – 2**.

$$h_2 = h_1\,[(W_1/D_1) + 0.60] / [(W_2/D_2) + 0.60]$$

(Equation 7-17)

where:

h = Thickness of sprayed fire-resistant material in inches.

W = Weight of the structural steel beam or girder in pounds per linear foot.

D = Heated perimeter of the structural steel beam in inches.

Subscript 1 refers to the beam and fire-resistant material thickness in the *approved* assembly.

Figure 3.2 - 5 | SFRM Thickness Modifier – Eq. 7-17, IBC 2012

Table 3.2 – 2 | Required Spray Fireproofing Thickness

722.5.2.2.1 Requirements					
Min W/D for Substitute Beam:	0.37	OK			
Min Thickness of Protection:	0.375	in			
Unrestrained/restrained?	Unrestrained (to be conservative)				
Min Fire Rating:	1 hour				
Required Fire Rating:	2 hour				
Minimum Beam Size:	W12x14				
Heated Perimeter:	0.405				
Assembly Tested	**Min Beam Size**	**h1**	**W1/D1**	**W2/D2**	**h2**
N708	W8x28	1.00	0.819	0.405	**1.412**

For the final design of the fire protection for the floor framing members, 1.5 inches of spray fire proofing will be applied to all floor members in accordance with Underwriter Laboratories Design No. N708 to create a 2 hour fire-resistance rating. No metal lath, slab reinforcing, or welded wire fabric is required. All members are to be sprayed for over the full exposed surface, as well as, all connections between floor framing members. Connections to columns will be protected within the fire protection assembly of the columns. This assembly analysis and selection will be discussed in the following section.

Structural Column Fire Protection Design

The design of the fire protection for gravity and lateral columns will vary depending on the column size and dimensions. For the purposes of this report, the fire protection is designed for the smallest and lightest column used in the design. The fire protection selected for the smallest member will also work for larger members.

The smallest column size used for the steel redesign of La Jolla Commons is a W10X49 at the roof level and is a gravity member. As the above analysis shows, the columns will require a 3 hour fire-resistance rating. As with the steel beam elements, approved assemblies must be used and can be found in the Underwriters Laboratory Database. To maintain the original architecture of the building, the columns will be encased in Type X gypsum wall board so that they will resemble the original concrete columns that they are replacing. The design selected is UL Design No. X508. **Figure 3.2 – 6** shows a schematic of this assembly, and the list of required items are listed below.

Design No. X508

February 22, 2014

Rating — 3 Hr.

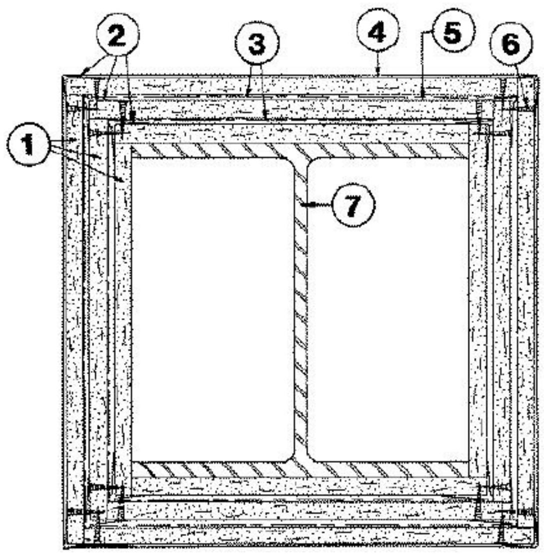


Figure 3.2 - 6 | UL Design No. X508 – Column Fire Protection

1. The outer layer must be 5/8 inches thick. The inner layers will be 5/8 inch thick wall board as well. The wallboard is installed without any horizontal joints. 1 inch long self-drilling screws shall be spaced as required for the installation of the first layer of wall board.
2. 28 MSG galvanized metal corner bead
3. 18 SWG annealed wire, space 6 inches from each end and at 1'-9" intervals
4. May be finished with 3/32" thick gypsum veneer plaster. Joints reinforced.
5. Laminated with joint cement.
6. 1 inch long self-drilling screws spaced at 12" center to center
7. Minimum column size of W10X49. 9/16 flange thickness and 5/16 inch web thickness. 14.4 square inch area.

Floor-to-Floor Protection

La Jolla Commons Phase II Office Tower is designed to function as a multi-tenant office building, although it is currently being utilized by only LPL Financial. As a result, the building will be divided into separate fire areas for each floor; although all levels will be Occupancy Category B, a fire resistance rating between the groups will be required in order to isolate each fire area. According to IBC 2012 Section 711.3, when a floor assembly separates a single occupancy into different fire areas, the assembly will have a fire-resistance-rating according to Table 707.3.10 which can be seen in **Figure 3.2 – 7**. This requirement is achieved by the deck and slab configuration selected for the floor framing system. According to the Vulcraft Deck Catalogue, 3.25 inches of lightweight concrete topping is required to achieve a 2 hour fire-rating for unprotected deck, as shown in **Figure 3.2 – 8**. 1.5VLR20 deck with 4.25" LW topping has been provided. Therefore, the fire-rating of the floor system will meet the required 2 hour rating required between business occupancies.

TABLE 707.3.10
FIRE-RESISTANCE RATING REQUIREMENTS FOR FIRE BARRIER ASSEMBLIES OR HORIZONTAL ASSEMBLIES BETWEEN FIRE AREAS

OCCUPANCY GROUP	FIRE-RESISTANCE RATING (hours)
H-1, H-2	4
F-1, H-3, S-1	3
A, B, E, F-2, H-4, H-5, I, M, R, S-2	2
U	1

Figure 3.2 - 7 | Resistance between Occupancies – IBC 2012 Table 707.3.10

Restrained Assembly Rating	Type of Protection	Concrete Thickness & Type (1)
2 Hr. (continued)	Sprayed Fiber	2" NW&LW
		2 1/2" NW&LW
		2 1/2" LW
		2 1/2" NW
		3 1/4" LW
	Unprotected Deck	3 1/4" LW
		4 1/2" NW

Figure 3.2 - 8 | Vulcraft Deck Catalogue Fire Ratings

It is important to remember that a separation of occupancies is not required according to Table 508.4 in IBC 2012. The required occupancy separation is N – No separation requirement. It has been a design decision to separate the separate tenant levels into separate fire areas.

However, the interface between Level 1 and Lower Level 1 of the parking garage poses a different problem. According to Table 508.4, the separation of S-2, an enclosed parking garage, and B occupancies is required to have a fire-resistance-rating of 1 hour. This is not a concern as the original concrete system will be utilized for the parking garage beginning at Level 1. The existing concrete floor has a minimum of a 10 inch thickness. Therefore, the 1 hour requirement is met.

Incidental Uses Concerns

Table 509 of IBC 2012 lists requirements for the separation required for different spaces within a particular occupancy group. For example, there are mechanical rooms at the core of each level in LJC II within the office space. Do these rooms need to be separated or protected differently than the rest of the office space? As stated before, the new building design will utilize the full automatic sprinkler system used in the original design. According to Table 509, separation of furnace, refrigerant machinery, and waste and linen collection spaces need not be fire rated if an automatic sprinkler system is present. Table 509 can be viewed below in **Figure 3.2 – 9**. As a result, the shear walls and partitions used to enclose these spaces will be sufficient according to IBC 2012.

TABLE 509
INCIDENTAL USES

ROOM OR AREA	SEPARATION AND/OR PROTECTION
Furnace room where any piece of equipment is over 400,000 Btu per hour input	1 hour or provide automatic sprinkler system
Rooms with boilers where the largest piece of equipment is over 15 psi and 10 horsepower	1 hour or provide automatic sprinkler system
Refrigerant machinery room	1 hour or provide automatic sprinkler system
Hydrogen cutoff rooms, not classified as Group H	1 hour in Group B, F, M, S and U occupancies; 2 hours in Group A, E, I and R occupancies.
Incinerator rooms	2 hours and automatic sprinkler system
Paint shops, not classified as Group H, located in occupancies other than Group F	2 hours; or 1 hour and provide automatic sprinkler system
Laboratories and vocational shops, not classified as Group H, located in a Group E or I-2 occupancy	1 hour or provide automatic sprinkler system
Laundry rooms over 100 square feet	1 hour or provide automatic sprinkler system
Group I-3 cells equipped with padded surfaces	1 hour
Waste and linen collection rooms located in either Group I-2 occupancies or ambulatory care facilities	1 hour
Waste and linen collection rooms over 100 square feet	1 hour or provide automatic sprinkler system
Stationary storage battery systems having a liquid electrolyte capacity of more than 50 gallons for flooded lead-acid, nickel cadmium or VRLA, or more than 1,000 pounds for lithium-ion and lithium metal polymer used for facility standby power, emergency power or uninterruptable power supplies	1 hour in Group B, F, M, S and U occupancies; 2 hours in Group A, E, I and R occupancies.

For SI: 1 square foot = 0.0929 m², 1 pound per square inch (psi) = 6.9 kPa, 1 British thermal unit (Btu) per hour = 0.293 watts, 1 horsepower = 746 watts, 1 gallon = 3.785 L.

Figure 3.2 – 9 | **Incidental Uses – Table 509 IBC 2012**

Exterior Wall Protection

Now that the interior of the building has been designed to properly resist fire, the exterior walls are the next concern. Both exterior projections and the exterior wall materials need to be analyzed for fire-resistant performance. The existing façade system will be analyzed to determine if it meets the requirements of IBC 2012 for fire protection. Section 705 – Exterior Walls of IBC 2012 will be used to perform this analysis.

Projections

According to IBC 2012 Section 705.2, projections extending beyond the building's exterior wall must be of Type I and Type II construction. La Jolla Commons Phase II Office Tower features two cantilevered ends that will extend beyond the exterior wall below them. Also, the main lobby area on Level 1 is stepped back from the façade above; see **Figure 3.2 – 10** for clarification.

Figure 3.2 – 10 | Building Projections under Investigation

The projections under consideration have all been protected using Type 1A or Type 1B construction. Also, the requirements of 1406.3 and 1406.4 are met because the exterior wall material is noncombustible. Therefore, the projections meet the requirement of Section 705.2 and are properly protected from fire.

Exterior Wall Requirements

The next area of concern with the building's overall fire-resistance and protection is the fire-resistive-rating of the exterior wall system. According to Section 705 of IBC 2012, the rating depends on the fire separation distance, occupancy group, and type of construction to determine the required fire-rating of the exterior wall system.

First to be determined is the fire separation distance, as shown in **Figure 3.2 – 11**. In accordance with IBC 2012, the fire separation distance is the minimum of the following:

1. The distance from the face of the building to the closest interior lot line
2. The distance from the face of the building to the centerline of a street
3. The distance from the face of the building to an imaginary line between two buildings on the property.

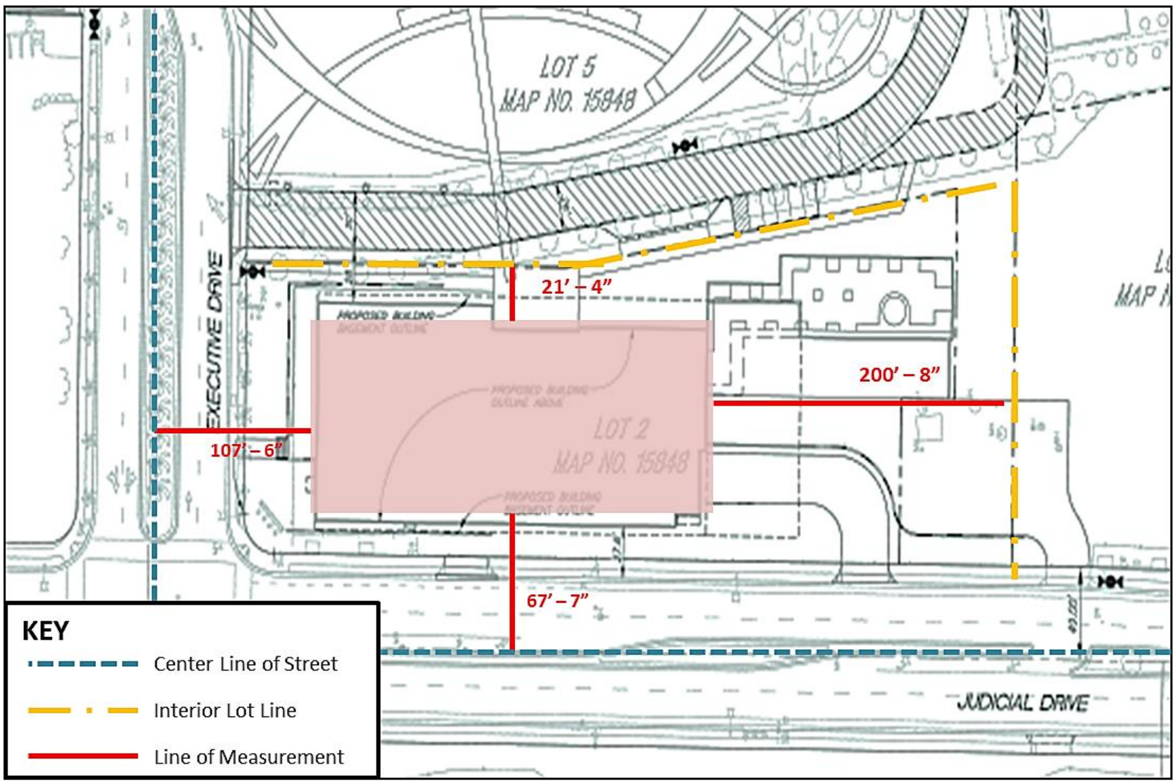


Figure 3.2 – 11 | **Fire Separation Distance Diagram**

With the minimum fire separation distance of 21' – 4", occupancy category B, and construction Type 1A, a 1 hour fire rating is required for the exterior walls. This is based on Table 602 from IBC 2012 as can be seen in **Figure 3.2 – 12**.

TABLE 602
FIRE-RESISTANCE RATING REQUIREMENTS FOR EXTERIOR WALLS BASED ON FIRE SEPARATION DISTANCE[a, e, h]

FIRE SEPARATION DISTANCE = X (feet)	TYPE OF CONSTRUCTION	OCCUPANCY GROUP H[f]	OCCUPANCY GROUP F-1, M, S-1[g]	OCCUPANCY GROUP A, B, E, F-2, I, R, S-2[g], U[b]
$X < 5^c$	All	3	2	1
$5 \le X < 10$	IA	3	2	1
	Others	2	1	1
$10 \le X < 30$	IA, IB	2	1	1[d]
	IIB, VB	1	0	0
	Others	1	1	1[d]
$X \ge 30$	All	0	0	0

For SI: 1 foot = 304.8 mm.
a. Load-bearing exterior walls shall also comply with the fire-resistance rating requirements of Table 601.
b. For special requirements for Group U occupancies, see Section 406.3.
c. See Section 706.1.1 for party walls.
d. Open parking garages complying with Section 406 shall not be required to have a fire-resistance rating.
e. The fire-resistance rating of an exterior wall is determined based upon the fire separation distance of the exterior wall and the story in which the wall is located.
f. For special requirements for Group H occupancies, see Section 415.5.
g. For special requirements for Group S aircraft hangars, see Section 412.4.1.
h. Where Table 705.8 permits nonbearing exterior walls with unlimited area of unprotected openings, the required fire-resistance rating for the exterior walls is 0 hours.

Figure 3.2 – 12 | **Exterior Wall Required Rating – IBC 2012 Table 602**

The façade of La Jolla Commons consists of a unitized curtain wall system with nominal 2 ¾" by 6" deep framing members. Dependent upon the area of the building, the curtain wall is glazed by either conventional, capture glazing on all sides or a combination of capture glazing on the horizontals or structural silicone glazing on the verticals. The glass is typically 1" insulating (double pane) Low Emissivity high performance glazing. The exterior curtain wall system may be used as part of the fire-rated assembly, according to Section 716.2 of the IBC, but the assembly must be verified by testing to meet to 1 hour fire resistance rating. The curtain wall system for La Jolla Commons Phase II Office Tower has a tested and verified fire resistance rating of 1 hour as a minimum. Along with a perimeter fire protection system, the curtain wall system meets the requirements of IBC 2012 Section 715.

Perimeter Fire Containment

The curtain wall assembly will leave a void between the exterior wall and the floor system. According to IBC Section 715.4, the void must be sealed with an approved system to prevent interior spread of fire. The assembly must be tested and meet the required fire-resistance-rating of the floor assembly, which for the building redesign is 2 hours minimum. **Figure 3.2 – 13** shows the differences between an unprotected void and a protected void.

Perimeter fire containment is used to keep fire from spreading to the next floor. Since, the floor assembly is typically fire rated, a possible fire route is for fire to spread through and up the exterior curtain wall system. As a result, the exterior curtain wall system must eliminate the opening between the floor and wall system, and it must provide a fire containment barrier to keep flames from exiting the building and igniting materials on the floor above.

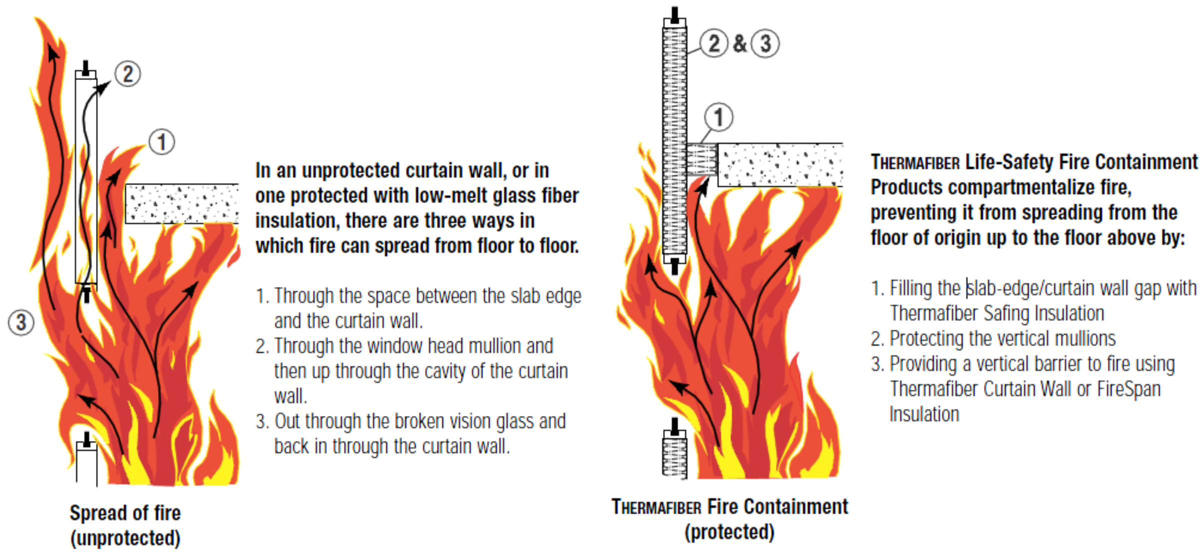


Figure 3.2 – 13 | **Perimeter Fire Containment vs. No Protection – From Thermafiber Catalogue**

The system chosen for the protection of the perimeter of the La Jolla Commons redesign is designed by Thermafiber Insulation systems. Thermafiber assemblies have been approved and tested by the Underwriters Laboratories; they are approved for both fire and smoke containment. The Thermafiber Fire Containment Curtain Wall system is mechanically attached to mullions using impaling pins, screws or other positive mechanical attachment. The exposed aluminum mullions must be protected with Thermafiber Curtain Wall mullion covers. A light steel angle or channel is placed horizontally at the safing line, attached to the vertical mullions. This will prevent bowing due to the compression fit of the safing insulation. To resist the passage of smoke, the Thermafiber Safing Insulation must be foil-faced.

The approved Aluminum Spandrel Curtain Wall Fire Containment system follows UL Design CW-S-2002. This will provide a 2 hour fire rating as required by the IBC. The aluminum spandrel panels are secured to the aluminum mullions. 2 inch thick foil-faced Thermafiber FireSpan Insulation installed between mullions. The inside face of all mullions are to be covered with 2 inch thick strips of foil-faced Thermafiber FireSpan Insulation. The curtain wall insulation and mullion covers are to be held in place on impaling pins by clinch shields. Thermafiber Safing Insulation, 4 inches thick will be placed between the concrete floor and the curtain wall insulation. It must be recessed 1 inch below the top surface of the concrete floor. A stiff back steel channel will be installed behind the curtain wall insulation to provide lateral support. Furthermore, FireCode Compound must be applied over the safing insulation at a thickness of 1 inch. See **Figure 3.2 – 14** for the Thermafiber assembly diagram.

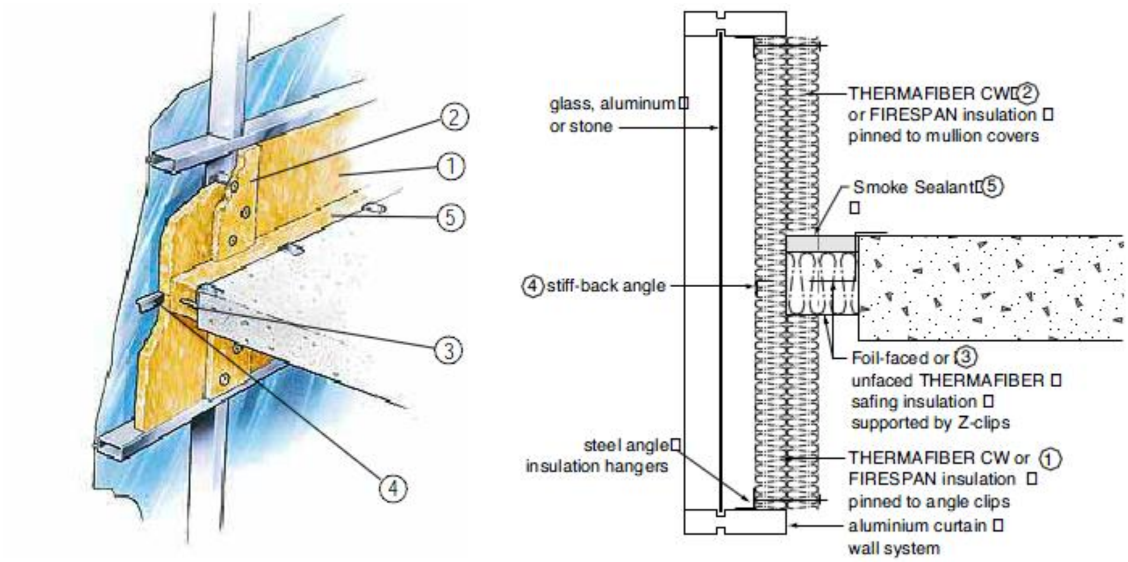


Figure 3.2 – 14 | **Thermafiber Perimeter Protection Assembly – from Thermafiber Catalogue**

Conclusion

The fire protection system for La Jolla Commons Phase II Office Tower has been detailed in this section. The change of the building structure from concrete to steel required an adjustment to the fire-rated assemblies of the floor systems, structural members, and exterior walls. As can be imagined, the change to steel requires significantly more fire resistive material application than the original concrete system. As will be showed in the construction cost analysis later in this report, the fire protection requirements will contribute to the new structure cost and schedule duration.

Chapter 4 – Construction Breadth

4.1 – Cost Analysis

Concrete Estimate

The structural engineer of La Jolla Commons Phase II Office Tower indicated that the building was designed in concrete because in California at the time of construction the concrete system was cheaper than the steel system. As part of the analysis of changing structural systems, this claim was analyzed as part of the second breadth.

The cost of the original building structure was provided by the property developer, Hines. The cost of the building structure with general conditions and overhead was found to be $61.46 per square foot including basement levels. This estimate included lower garage levels, foundations, shear walls, slabs, concrete columns, and rebar. The cost of the concrete structural system, at about $24.5 million, was about 30% of the total building cost.

Steel Estimate

A detailed cost analysis was performed on the new building structural system utilizing R.S. Means Building Construction Cost Data 2009 Edition. A location modifier of 1.051 was utilized to account for the increased cost in the San Diego area. Also, a time modifier of 1.13 was applied to account for the increase in cost between 2009 and April 2014, assuming 3% inflation per year.

Table 4.1-1 | Steel Structure Cost Analysis

Total Steel Structure Cost (Based on 2009 RS Means)		
Item	Cost	% Total Cost
Concrete on Metal Deck	$ 3,049,983.64	12.01%
Structural Steel Framing	$ 9,052,267.61	35.65%
Shear Walls	$ 4,309,712.97	16.97%
Foundation Walls	$ 1,929,048.98	7.60%
Lower Level Concrete Slabs	$ 2,796,418.47	11.01%
Lower Level Concrete Columns	$ 198,415.62	0.78%
Mat Foundation	$ 4,054,749.44	15.97%
Total Cost	$ 25,390,596.74	100.00%

Location Modifier	105.1	1.051
Time Multiplier		1.13

Final Modified Total Cost	$ 30,072,276.51

This steel system estimate includes all the steel and fireproofing that makes up the superstructure and lateral moment frames. It also includes the existing concrete shear walls, foundation walls, mat foundation, and lower level slabs and concrete columns. The concrete estimate included formwork, reinforcing steel, concrete material and placement, and concrete finishing. Formwork, rebar, and concrete were increased by 10%, 5% and 5% respectively to account for waste. All concrete take-offs were performed based on the original building drawings. The steel take-offs were performed using RAM Structural System output.

Table 4.1 – 1 shows a breakdown of the detailed cost analysis performed on the steel structure. As expected, the structural steel framing makes up a majority of the building cost. The next highest percentage, as expected, came from the special concrete shear walls located at the building core. For a detailed breakdown of the cost analysis performed for the steel and concrete structures, see spreadsheets located in **Appendix L**.

Cost Comparison

Table 4.1-2 | Cost Comparison Analysis

Cost Comparison		
Original Concrete Structure Cost	$	24,435,196.74
New Steel Structure Cost	$	30,072,276.51
% Increase in Cost		23%

After this study was performed, the claim that the steel system would be more expensive than the steel alternative was verified. As expected, the steel system was more expensive, at $30 million, than the concrete system, at $24.5 million. This is about a 23% increase in the cost of the building structure. It is important to note that some detail was not taken into account in the steel estimate. Things such as connections from steel-to-steel and steel-to-concrete were not analyzed. Also, the steel system would have an impact on the mat and the foundation wall designs. These things were also not taken into account. Therefore, the difference in cost could be slightly different than that shown in **Table 4.1 – 2**. The square foot cost of the original concrete system was $61.46 and the new steel system is $65.05. Considering these results, it is apparent why the structure was designed in concrete and not steel.

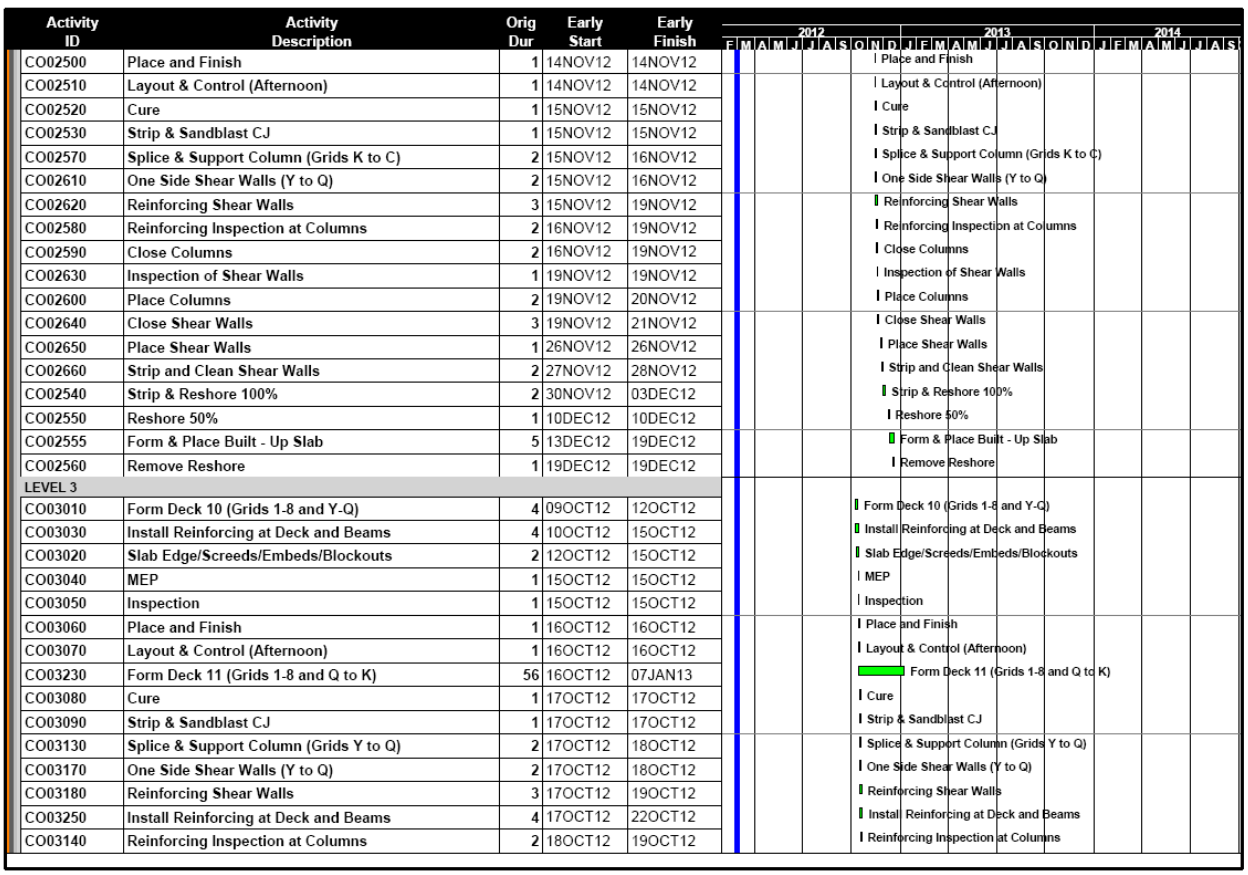
4.2 – Schedule Analysis

Concrete Schedule

The schedule for the existing concrete system was obtained from the project manager and developer, Hines, for La Jolla Commons Phase II Office Tower. The original building superstructure construction began on September 12th, 2012 and ended on May 24th, 2013. This results in a total duration of 240 work days or 34 weeks for the building superstructure. The original schedule critical tasks included the concrete slabs, concrete columns, and concrete shear wall placement. Inspection of the special shear walls also increased the project duration. Tasks such as MEP rough in were found to not impact the construction schedule because they were not predecessors to any activities. The shear walls were found to have the greatest impact on the building schedule. In order in increase the pace of slab forming and placement, the building was separated into three zones that were formed and placed with three separate crews. Finishing of concrete occurred simultaneously on multiple levels. Two cranes were used during construction. **Table 4.2 – 1** shows the superstructure durations, and **Figure 4.2 – 1** shows a portion of the original concrete schedule.

Table 4.2 - 1 | Original Schedule Duration

Original Schedule Structure Duration	
Structure Begin	26-Sep-12
Structure Complete	24-May-13
Total Duration (years)	0.66
Total Duration (weeks)	34
Total Duration (days)	240



Activity ID	Activity Description	Orig Dur	Early Start	Early Finish
CO02500	Place and Finish	1	14NOV12	14NOV12
CO02510	Layout & Control (Afternoon)	1	14NOV12	14NOV12
CO02520	Cure	1	15NOV12	15NOV12
CO02530	Strip & Sandblast CJ	1	15NOV12	15NOV12
CO02570	Splice & Support Column (Grids K to C)	2	15NOV12	16NOV12
CO02610	One Side Shear Walls (Y to Q)	2	15NOV12	16NOV12
CO02620	Reinforcing Shear Walls	3	15NOV12	19NOV12
CO02580	Reinforcing Inspection at Columns	2	16NOV12	19NOV12
CO02590	Close Columns	2	16NOV12	19NOV12
CO02630	Inspection of Shear Walls	1	19NOV12	19NOV12
CO02600	Place Columns	2	19NOV12	20NOV12
CO02640	Close Shear Walls	3	19NOV12	21NOV12
CO02650	Place Shear Walls	1	26NOV12	26NOV12
CO02660	Strip and Clean Shear Walls	2	27NOV12	28NOV12
CO02540	Strip & Reshore 100%	2	30NOV12	03DEC12
CO02550	Reshore 50%	1	10DEC12	10DEC12
CO02555	Form & Place Built - Up Slab	5	13DEC12	19DEC12
CO02560	Remove Reshore	1	19DEC12	19DEC12
LEVEL 3				
CO03010	Form Deck 10 (Grids 1-8 and Y-Q)	4	09OCT12	12OCT12
CO03030	Install Reinforcing at Deck and Beams	4	10OCT12	15OCT12
CO03020	Slab Edge/Screeds/Embeds/Blockouts	2	12OCT12	15OCT12
CO03040	MEP	1	15OCT12	15OCT12
CO03050	Inspection	1	15OCT12	15OCT12
CO03060	Place and Finish	1	16OCT12	16OCT12
CO03070	Layout & Control (Afternoon)	1	16OCT12	16OCT12
CO03230	Form Deck 11 (Grids 1-8 and Q to K)	56	16OCT12	07JAN13
CO03080	Cure	1	17OCT12	17OCT12
CO03090	Strip & Sandblast CJ	1	17OCT12	17OCT12
CO03130	Splice & Support Column (Grids Y to Q)	2	17OCT12	18OCT12
CO03170	One Side Shear Walls (Y to Q)	2	17OCT12	18OCT12
CO03180	Reinforcing Shear Walls	3	17OCT12	19OCT12
CO03250	Install Reinforcing at Deck and Beams	4	17OCT12	22OCT12
CO03140	Reinforcing Inspection at Columns	2	18OCT12	19OCT12

Figure 4.2 - 1 | Portion of Original Concrete Schedule – Provided by HINES

Steel Schedule

The new steel schedule was developed using a repeating process of tasks for each level using Microsoft Project. Levels were done in sets of two, due to column splicing every two stories; therefore, work can occur simultaneously on these two levels within reason based on crane usage. It was assumed in the calculation of durations that two cranes would be used on the project; one at each end of the site as shown in **Figure 4.2 – 2**.

Figure 4.2 - 2 | **Site Crane Placement**

First, steel columns need to be erected on the two levels under construction. Next the shear walls need to be constructed at these levels. Rebar for the shear walls will be placed, the walls formed and placed, and then allowed to cure. While this is occurring, steel framing can begin to be placed for the two levels at the building perimeter bays. The interior bays can only be placed after the shear walls are complete. This must occur because the shear walls will act as one supporting end for the beams in the interior bays. After the framing of the two levels has begun, the deck placement of these levels may also begin. A start-to-start lag of 1 day is required in order for the placement of enough framing members to allow for the installation of decking. The deck must be laid and welded, concrete placed and allowed to cure, and the floor level needs to be finished. After the completion of the deck for levels 4 and 5, fireproofing of the floor framing members can begin. See **Figure 4.2 – 3** for a sample of the schedule. For the entire schedule and duration calculations, see **Appendix M**.

	Task Name	Duration	Start	Finish	Prede
9	Decking - Level 3	9.8 days	Tue 4/29/14	Mon 5/12/14	8
10	Fireproofing - Levels 2 and 3	8 days	Mon 5/12/14	Thu 5/22/14	7,9
11	**Levels 4 and 5**	**42.1 days**	**Mon 5/12/14**	**Thu 7/10/14**	
12	Column Erection - Levels 4 and 5	3 days	Mon 5/12/14	Thu 5/15/14	9
13	Rebar SW - Level 4 and 5	4 days	Mon 5/12/14	Fri 5/16/14	5,9
14	Form SW - Level 4 and 5	12 days	Fri 5/16/14	Tue 6/3/14	13
15	Place SW - Level 4 and 5	1.5 days	Tue 6/3/14	Thu 6/5/14	14
16	Floor Framing- Level 4	2.8 days	Thu 6/5/14	Tue 6/10/14	15,12
17	Decking - Level 4	7.5 days	Tue 6/10/14	Thu 6/19/14	16
18	Floor Framing - Level 5	4 days	Tue 6/10/14	Mon 6/16/14	16
19	Decking - Level 5	9.8 days	Mon 6/16/14	Mon 6/30/14	18
20	Fireproofing - Levels 4 and 5	8 days	Mon 6/30/14	Thu 7/10/14	17,19
21	**Levels 6 and 7**	**43.3 days**	**Mon 6/30/14**	**Thu 8/28/14**	
22	Column Erection - Levels 6 and 7	3 days	Mon 6/30/14	Thu 7/3/14	19
23	Rebar SW - Level 6 and 7	4 days	Mon 6/30/14	Fri 7/4/14	15,19
24	Form SW - Level 6 and 7	12 days	Fri 7/4/14	Tue 7/22/14	23
25	Place SW - Level 6 and 7	1.5 days	Tue 7/22/14	Wed 7/23/14	24
26	Floor Framing - Level 6	4 days	Wed 7/23/14	Tue 7/29/14	25

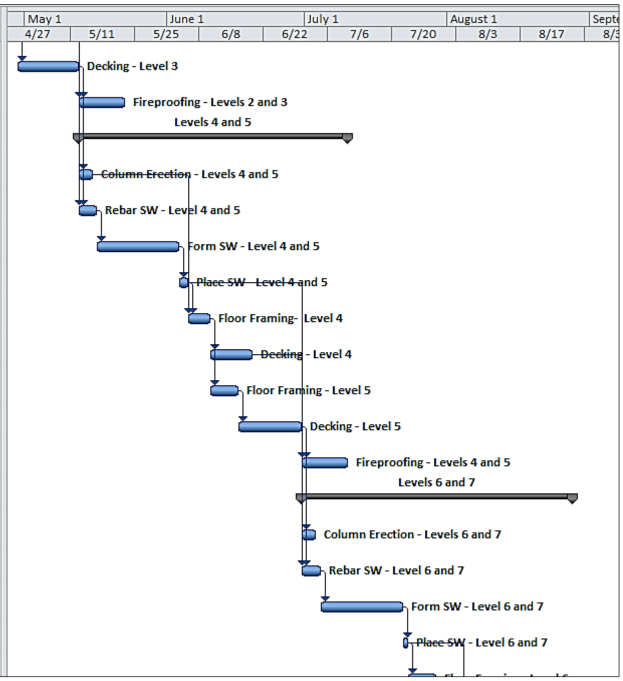


Figure 4.2 - 3 | **Portion of Steel Schedule from Microsoft Project**

This process will proceed up to the next two levels, requiring that the framing of the previous levels be complete and the shear walls at those levels placed and cured. The process will then be similar to that already described. This will repeat until the structure is topped out at the roof level. A typical two level process takes about 43 days to complete.

The durations for this schedule were determined using daily output values from RS Means. The project schedule was expedited using multiple crews on different tasks. For example, four crews were used or the placement and finishing of the deck. It is assumed that two crews would be on one level and two on the level above. Each crew would be responsible for one of two zones on the given level. For the placement of the structural steel, two crews were assumed; this also stems from the assumption that there will be two cranes on site. As a result, one crew will be responsible for each zone of the building on a given level. Multiple crews were also assumed for the construction of the shear formwork and reinforcing placement. Two crews were assumed for the placement of shear walls, requiring two pumps. The new steel structure schedule was found to take 230 days or 32 weeks.

Schedule Comparison

The total length of the original concrete schedule was 240 days or 34 weeks; the new steel schedule was found to be 230 days or 32 weeks. This is only a difference of 10 weekdays or two weeks. This was surprising; originally, it was expected that the steel system would take significantly less time than the cast-in-place concrete system. However, this was found to not be the case.

One factor that could have led to this outcome was the design decision to support the steel beams on the concrete shear walls. The steel framing of floor levels could not be completed until the shear walls at that particular level were complete; as a result, the time saving benefits of a steel system were offset by the time it takes to form, place, and cure the concrete shear walls. Another item that increased the duration of the steel system was the inclusion of fire protection materials. The original concrete system did not require additional time for fireproofing like the new steel system. Additional time is also required to inspect the new steel moment frames.

This result only further explains why La Jolla Commons was constructed of cast-in-place concrete and not steel. If the steel schedule was greatly reduced, the higher cost of the steel system may have been worth the time savings. However, because the schedule was not reduced by a significant amount, the increased cost does not seem to be worth it.

Overall System Comparison

La Jolla Commons Phase II Office Tower was originally designed to be a concrete structure. This redesign changed the gravity system from concrete to steel and implemented the use of special moment frames as part of the new lateral system. There are some positive and negative aspects to this redesign that will be discussed, and a final recommendation will be made to the building owner.

First, the steel redesign resulted in a significantly lighter structure. This, in turn, considerably reduces seismic forces, which was shown to be the controlling lateral load for the building's lateral system. The lower seismic forces will limit the required shear wall reinforcing and moment frame member sizes. As was shown in the redesign of the concrete shear walls, the reduced forces allowed for a reduction in the reinforcing required in the special concrete shear walls.

One concern that was present early in the steel system design process was the control of walking induced vibrations in the office environment. As was shown in Chapter 2.1 of this report, the vibrations of the steel system were controlled to the limits of AISC Design Guide 11. Vibrations in the steel system will be left unnoticed to most occupants; therefore, this serviceability concern would not be a major factor in deciding whether the building should be steel or concrete.

A downside to the steel system is that fire protective materials will be required on the structural elements, unlike concrete which has inherent fire-resistive abilities. This will add additional cost and time to the schedule. Another disadvantage is the decrease in floor-to-ceiling heights from 10' – 8" to 9' – 2" when switching to a steel structure. Although many office buildings have ceiling heights around 9' – 0", this may still be undesirable for the building owner.

Furthermore, there was a 23% increase in structure cost associated with the steel system. Also, there was only a time savings of about two weeks for the steel construction schedule. As was mentioned in Chapter 4.2, the time savings in the schedule is too minimal to offset the significant increase in cost with the steel system.

Based on all of these factors, although a steel structure is certainly feasible, it was not necessarily the best choice for this particular project. Concrete allows for higher floor-to-ceiling heights, lower costs without a significant schedule increase, and does not require fire-resistive materials. Also, the concrete system will inherently control vibrations. Thus, a concrete structure is probability the most efficient choice for La Jolla Commons Phase II Office Tower.

Conclusion

The report consisted of an analysis of La Jolla Commons Phase II Office Tower. After studying the existing concrete system during the fall semester, it was decided to investigate the redesign of the building structure in steel, using a design scenario as a guide. The gravity system was redesigned using the original column locations and steel composite beams. A preliminary assessment was done to determine a deck configuration and beam spacing to control walking induced vibrations. Next, RAM Structural System was used to analyze and design the most economical cross sections for the beams and columns. These designs were then verified using hand calculations. The final beam layout and design was then analyzed according to AISC *Design Guide 11* and found to be adequate for walking induced vibrations.

The next portion of this report addressed the redesign of the building's lateral system. The original lateral system had an extreme torsional irregularity under seismic loading. Special steel moment frames were added to the perimeter and designed for strength considerations. The designs were further refined to allow for "clean columns" – columns that do not require web plates, flange stiffeners, continuity plates, etc. Upon analyzing the building drift and torsion under seismic loading, drift was found to be under the code maximum, and the torsional irregularity was found to no longer exist.

A breadth in architecture was done to assess the impact of changing the building structure on the building height. It was found that, in order to meet the FAA height limitation of 198'-8", the floor-to-ceiling height would decrease from 10'-8' for the concrete structure to 9'-2" for the steel structure. Although many office buildings have ceiling heights around 9'-0", this may be undesirable for the owner. This breadth was further expanded to determine the fire-resistive requirements and designs for the new steel structure. It was determined that Type 1B construction is required for the structural elements except for columns which must be Type 1A. The columns required a 3 hour fire rating, which was achieved using a layered gypsum encasement. The structural floor framing members required a 2 hour fire rating, which was achieved using 1.5" of spray fire proofing materials. A perimeter fire containment assembly was also selected.

The second breadth analyzed the schedule length and cost of the building structure. It was found that the new steel system costs about 23% more than the existing concrete system. This analysis was based on original cost information provided by Hines, and a cost analysis performed using RS Means 2009. A schedule analysis was also performed. It was found that the steel system only takes about 2 weeks less to construct than the concrete system.

Although there are benefits to the steel system, including reduced seismic loads and no torsional irregularities, the steel system may not be the most effective design for this particular project. The concrete system will allow for higher floor-to-ceiling heights, lower costs without a significant schedule increase, and does not require additional fire-resistive materials. Therefore, the concrete structure is probably the most efficient choice for La Jolla Commons Phase II Office Tower.

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NOTE: In addition to the sources listed above, course notes were utilized from many courses in the Penn State Architectural Engineering Department.