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Senior Thesis Final Report

Structural Emphasis

Spring 2007

The Pennsylvania State University

555 12th Street - Oakland, CA

Advisor: Dr. Lepage

555 12th Street - Oakland City Center

Project Overview
- Downtown Oakland, California
- 21 Story Class A Office Building
- 487,000 Sq. Ft.
- Constructed May 2000 – April 2002
- $75,000,000 Project Cost
- Design – Build Delivery

Project Team
- Developer/Owner – The Shorenstein Co.
- Architect – Korth Sunseri Hagay Architects
- General Contractor – Charles Pankow Builders
- Structural Engineer – Nishkian Menninger Inc
- Mechanical – Acco Air
- Electrical – Schwartz and Lindheim
- Plumbing – L.J. Kurse Co.
- Lighting – Auerbach and Glasow

Structural
- Structural Steel-Frame Building with composite floors
- Lateral system - Dual system with steel braced frame core and special moment resisting frames at the perimeter. Concrete shear walls stiffen braced frame from basement to 2nd floor.
- Two story below grade parking garage is constructed Of precast concrete columns, beams, and girders and Reinforced concrete slabs

Architectural
- Base – Granite and glazed store-fronts, textured wall panels
- Middle - Clad in warm limestone-colored precast concrete Blue-green glass curtain wall on all sides
- Top - Conceptualized as an expansive curved lantern that will Glow on the Oakland skyline in the evenings.

Mechanical/Plumbing
- HVAC is "built up" system with mechanical equipment in penthouse
- Conditioned air distributed up/down lined shafts to each floor, then to main duct loop above ceiling - Diesel Fuel No. 2
- "Warm" shell with VAV boxes to control supply and reheating
- "Purple-pipe" system – dual plumbing system uses recycled H20

Lighting/Electrical
- 1600 Amp, 277/480V, 3 Phase, 4 Wire Switchboard (East) Primary
- 1000 Amp, 277/480V, 3 Phase, 4 Wire Switchboard (West) Primary
- 480-120/280V, 3 Phase, 4 Wire Switchboard/Floor typical - Tenant
- 1100 KW, 3 Phase, 4 Wire 277/480V Emergency Generator

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www.arche.psu.edu/thesis/eportfolio/2007/portfolios/erm159

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I would like to thank the following companies and professionals for their help throughout the past two semesters of thesis study. Your time and advice was a crucial component of this completed work. Thanks to Jim Dick of Charles Pankow Builders for getting me the drawings, and construction management issues. Thanks to Kevin Menninger for the structural design explanations. And also, thanks to John Dolby of Shorenstein Company for permission to use the building for my thesis.

I would also like to extend my appreciation to the entire AE structural faculty for their help throughout the years. Thank you to Dr. Andres Lepage for meeting with me and my fellow ETABS adventurers on a weekly basis, sometimes more.

I would like to thank my fellow AE's for their help and friendship over the years in and out of class. Not all of you went structural, but I forgive you.

Lastly, I want to thank my family and loved ones for their support throughout this past semester and my entire college career. I don't know what I would've done without you all.

EXECUTIVE SUMMARY

555 12TH Street is a 21 Story, 487,000 square foot complex that features class-A office space, retail space, and dining in one location. The majority of framing is structural steel W-shapes with a composite metal deck. The typical floor has an area of 24000 sq. ft. and provides a column free floor plan. All vertical structural elements are centered at the core, or perimeter frame. This open floor plan allows for the tenant to set up their office space to their own desire. The lateral system is a combination of eccentric braced frames at the core, and special moment resisting frames on the perimeter. This is a dual system acting in both major axes directions.

The Depth work conducted for this report focused on the redesign of the lateral force resisting system from a dual system to a reinforced concrete shear wall core. The proposed design exceeds the height limit of 240 ft, for shear wall systems in a high seismic region, so this design would be subject to a peer review. In order to ensure the most likelihood of acceptance of design in the professional world, 555 12th Street was designed based on performance under dynamic loading. This design included prescribing its own design criteria to exceed the code requirements of the IBC-2006, ASCE 7-05, and ACI 318 -05. With the implementation of shear walls, the perimeter special moment frames would be redesigned as gravity members only.

The structural program ETABS was used to perform the static and dynamic analysis of the building. Forces calculated from the equivalent lateral force procedure in ASCE7 were used in the model and design. The modeling process was iterative, and required a great deal of time to find the best design possible. After analysis and design it was found that the R/C shear walls in the core performed excellent under dynamic loading. All design criteria defined in the report were checked and passed. PCA column and RAM Structural System were used to check the shear wall designs and steel gravity columns, respectively. Hand calculations and design of coupling beams was also performed. The symmetrical layout of the floor and lateral force resisting elements limited the inherent torsion on the building. The removal of perimeter moment frames was thought to possibly cause excessive torsional forces, but the shear walls were found to have more than enough capacity and stiffness to account for this. The final layout consisted of four shear wall piers connected with coupling beams.

Breadth work investigating speech privacy of an open office layout, and construction management issues were also performed. It was determined that the speech privacy required for an open office space can be met with proper partitions, sound absorbing finish materials, and masking sound producers.

After completing the redesign and analyzing the cost, schedule, and constructability of the new system, it is the opinion of the writer that the new design is a feasible and economically advantageous alternative to the original design. With steel prices rising over the past decade, it is even more to owner's benefit, today, to consider the core only lateral system. It is imperative, that the peer reviewer be brought into the design phase at the beginning, and has periodic meetings to discuss/converse. If this is not done, then the building permit could be put on hold, which could greatly impact the start date of construction. This could mean a loss of thousands of dollars if tenant space was reserved for an opening date, and the building is not complete.

INTRODUCTION

555 12th Street is a 21 story, 487,000 square foot complex that features Class-A office space, retail space, and dining in one covenant location. Located in the heart of downtown Oakland, California, the building provides great views of the San Francisco Bay, as well as the East Bay Hills. It is one of several buildings that make up what is known as the Oakland City Center.

The building was completed in April 2002 after two years of construction, and is owned and managed by the Shorenstein Company. Korth Sunseri Hagay Architects was hired to lead the architectural design of the building while Nishkian Menninger Inc. was in charge of structural systems design. Charles Pankow Builders were the general contractor in charge of the $75,000,000 design-build project. The remaining firms involved with the project are listed below.

Trade	Firm
Mechanical	Acco Air
Electrical	Schwartz and Lindhelm
Plumbing	L.J. Kurse Co.
Lighting	Auerbach and Glasow
Landscape Arch	Guzzardo and Associates Inc
Fire Protection	Allied Fire Protection

There are two levels of underground parking available on site, a ground floor plaza, nineteen elevated office floors, and a mechanical floor. Each office floor has a gross area of 24000 square feet with the stairs, elevators, and HVAC towers located in the core of the building. The main support columns occupy the core and the perimeter walls which allow for a column free work space for tenants.

Architecture:

The building is expressed in a contemporary way, yet the fundamental architecture expression of base, middle and top is incorporated into the design. An important design objective for the 555 12th Street was to create a building base that would be an open and welcoming extension of the existing Clay Street pedestrian corridor. To accomplish this, Korth's design provides a base that meets the sidewalks along Clay Street, 11th Street and Jefferson Street while attracting pedestrian traffic on the north side along 12th Street. "This base incorporates glazed storefronts which recall the scale and rhythm of the older traditional storefront systems found in the area," Korth said. He added that delicate glass canopies, textured wall panels, custom wall sconces and a granite base will provide the detailing needed to match the nearby buildings.

The East/West curtain walls consist of mainly precast concrete spandrels, with blue vision glass mixed in. Along the N/S elevations, which are considered the glazed walls, spandrel and blue vision glass are used and supported by the floor slabs. Precast panels are also located on these faces. The entire top of the building was conceptualized as an expansive curved lantern that will glow on the Oakland skyline in the evenings.

Mechanical:

The HVAC system is a "built-up" system with the air handlers, cooling towers, chillers and boilers on the top two stories. Conditioned air is distributed down lined drywall shafts to each floor. At each floor air is distributed into the main duct loop above the ceiling. Return air is distributed up through the ceiling plenum and then into the core vertical gypboard shaft. Halfway through the project, the Owner requested to "warm" the shell. This included adding supply ducts off of the main duct loop and VAV boxes that control the air supply and re-heats the air if required.

There are two boilers and two chillers which supply 6,000,000 BTU's/boiler and 415/650 Tons/chiller. The building is fueled by natural gas, and is 100% sprinkled. The building utilizes a purple pipe system for efficiency. It is a dual plumbing system that recycles water throughout the building, cutting operating costs.

Electrical/Lighting:

Electrical circuits are split up to serve each half of the building. One set controls the west wing, and the other controls the east. Electrical service is 277/480V at 1600A and 1000A, for the east and west primary switchboard. This power is then routed to 480-120/280V switchboards for each floor, for each tenants use. There is an 1100kW emergency generator on site, for use when power is lost to the building. Lighting for the typical office floors consists of fluorescent lights recessed into the drop ceiling.

EXISTING STRUCTURAL SYSTEMS

Foundation:

The foundation was designed based on soil reports by URS Greiner Woodward Clyde, dated April 13, 2000. The soil bearing capacity was found for three different load combinations. For dead load, dead + live load, and dead + live + earthquake, the capacities are 5000, 7500, and 10000 PSF respectively. A surcharge load at street side was calculated as 150 PSF. All concrete for the foundation has a 28-day strength of f'_c = 4000 PSI. The reinforcing steel is ASTM A615 GR 60 deformed bars.

Over 650 truckloads of concrete – 24 Million lbs.- were required to pour the mat foundation. The foundation has a 5 foot thickness near the exterior walls, and transitions to 7 feet thick as it approaches the interior core. The entire mat is reinforced with #9 @ 8" Top EW and #10 @ 8" Bottom EW.

Spread and continuous footings are used to support the columns of the parking garage and first floor columns that extend beyond the footprint of the elevated floors. Spread footings, 3' thick and reinforced with #5 @ 12" Top EW and #9 @ 8" Bot. bars EW are used to support the interior columns of the parking garage. There sizes range from 10' to 20' for both length and width. Typical exterior and interior wall footings are continuous and 2'-6" thick. They are reinforced with #6 @ 14" bars T&B EW, unless otherwise noted.

Columns:

Most of the columns in the building are part of lateral resisting frames. They will be described in that section. They attach to base plates and anchor to the top of concrete columns that run from the foundation up to the first floor. The EBF concrete column is 4’x4’ reinforced with (40) 1 3/8” diameter DYWIDAG treaded bar(ASTM722) and #5 @ 3” Ties Baugrid. The SMRF frame concrete columns vary in size from 3’ to 3’-9” square. They larger columns are reinforced with (20) 1” diameter DYWIDAG with #5 ties @ 3”. The 3’ columns are reinforced with (20) #11 vertical and #5 ties @ 3”. TS 8x8x3/8 are used typically as columns for the 21st floor up to the roof for the mechanical floor. All frame concrete columns are required to have a 56 day strength of f'_c = 7500 PSI.

All non-frame gravity steel columns range from W14x109 at the 21st floor, up to W14x500 at ground level. The canopy columns at the 1st floor are W14x53 and W14x48. The base plates are 30”x30”x3” with (4) 1 ½” Anchor bolts with 24” embedment.

Floor Systems:

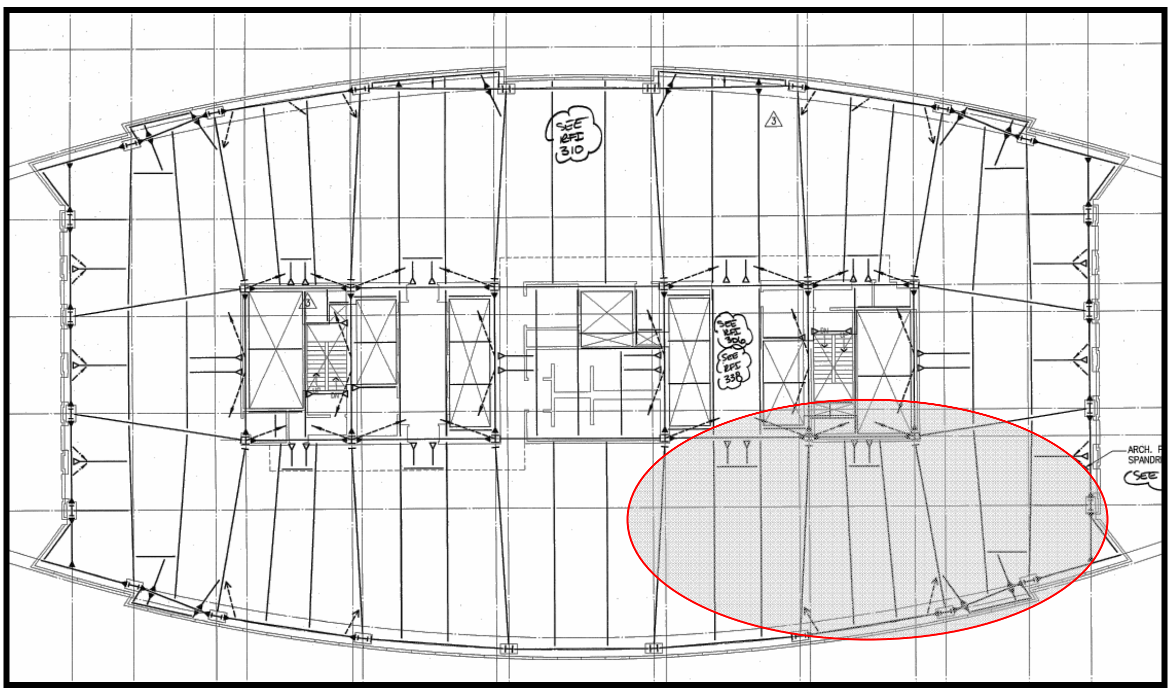


Figure 1: Typical Elevated Floor Framing and Plan (Circled – see Figure 2)

A 4” slab on grade(SOG) reinforced with #4 @ 18” EW is placed over a layer of class 2 aggregate fill, over the mat footing. All other SOG is 6” thick, reinforced with #4 @ 12” EW typically. All concrete for the SOG has a strength of f’c = 4000 PSI. Floors for level P1 and the 1st floor are cast-in-place(CIP) one way slab systems, supported by precast and CIP members. The thickness ranges from 6-12”, depending on location, and reinforcing varies from #4-#7 bars @ 12” T&B.

The majority of the structural system is designated as ASTM A992, Gr 50 steel, unless otherwise noted. The building takes advantage of two lines of symmetry, one in the N-S direction, and the other in the E-W direction. The typical floors, 2-21, have the same framing, unless otherwise noted. The elevated slabs are supported by wide flange beams with varying lengths, the longest being about 44', because of the curved exterior wall. They are typically W18x35 up to W18x55, unless otherwise noted and have cambers of up to 1 3/8" on the longest spans.

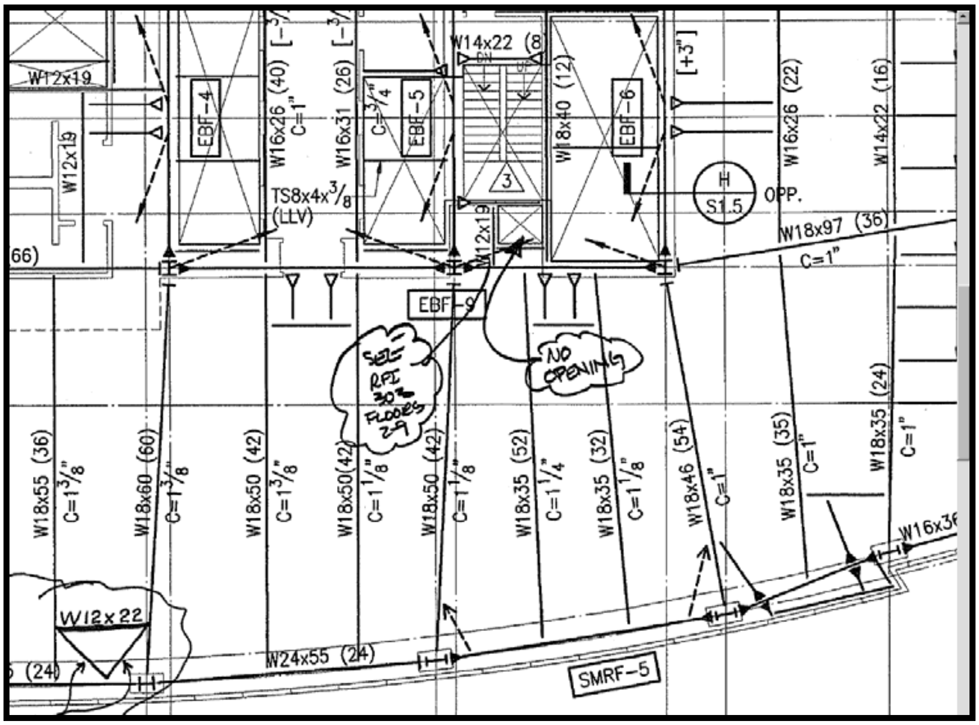


Figure 2 : South East typical steel layout

The girders, which are not part of the moment frames, are sized from W24x55 up to W27x84 and span at the greatest, 35'. Smaller W-shapes are used on the interior core area to support the slabs. The 22nd floor-mechanical floor has the same location of beams and girders, but different sizes. The typical beam is a W24x55 up to a W24x94. The typical girder is slightly larger, being a W27x84 on the exterior wall, and W30x124 on the interior core. The roof uses W12x22 up to W21x44 for its beams and girders, along with TS shapes for exterior beams, sized as TS10x8.

The elevated floors, starting from level 2, are composite metal deck systems. The 2nd floor is 3" 18 gage composite decking with 4" of normal weight concrete cover. It is reinforced with #4 @ 16" EW. Typical floors 3-21 are 3" 18 gage composite deck with 2 ½" of normal weight concrete cover. The slabs are reinforced by either #6 @ 13" EW or WWF6x6 W1.9. The mechanical room on the 22nd floor, along with its mezzanine level, uses a variety of composite decking. There is either 3"-16 or 18 gage composite deck with up to 7" of normal weight concrete over it. Reinforcement is typically #4 @ 12" unless otherwise noted. The roof uses 3"-18 gage composite decking with 2 ½" of lightweight fill. It is reinforced with #3 @ 16" EW.

Lateral System:

The lateral system of 555 12th Street is considered a dual system in the N/S and E/W directions. Dual systems are systems with shear walls and/or braced frames and moment frames working in parallel to resist lateral forces. Moment frames are designed to resist 25% of the total base shear in the direction they act, as dictated by code. The building has a steel braced frame core and Special Moment Resistant Frames (SMRF) at the perimeter. From the basement to the 2nd floor, a concrete shear wall core was utilized to help stiffen the structure at the first floor, which has a high floor-to-floor height of 24 feet. A steel braced frame was used from level 2 through the roof. The steel braced frame "jamb" columns extended into the concrete shear wall. A more detailed description of each component of the lateral system is provided on the next few pages.

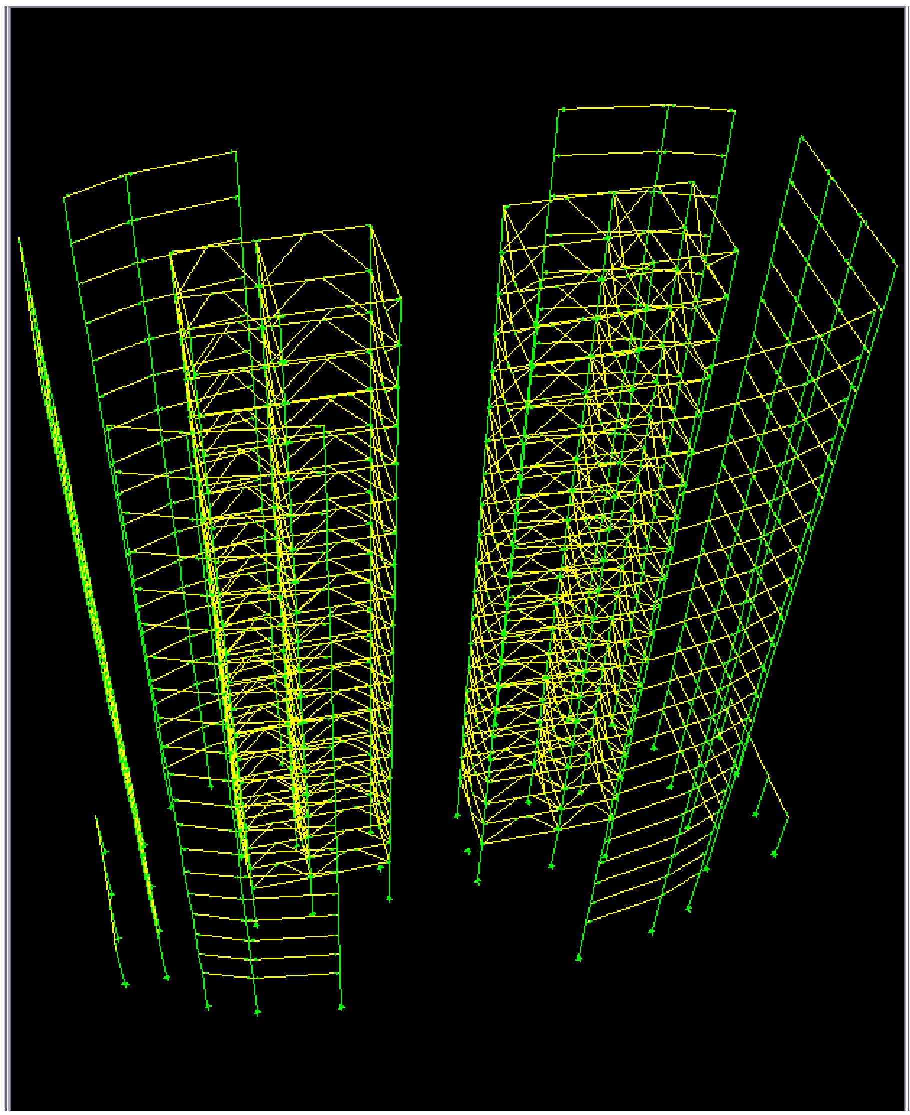
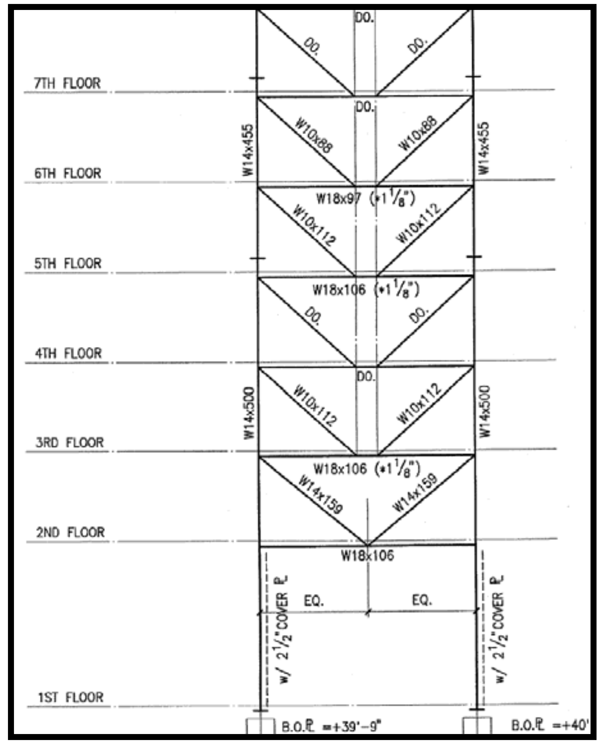


Figure 3: Existing Lateral Force Resisting Elements and Frames

Eccentric Braced Frames (EBF)

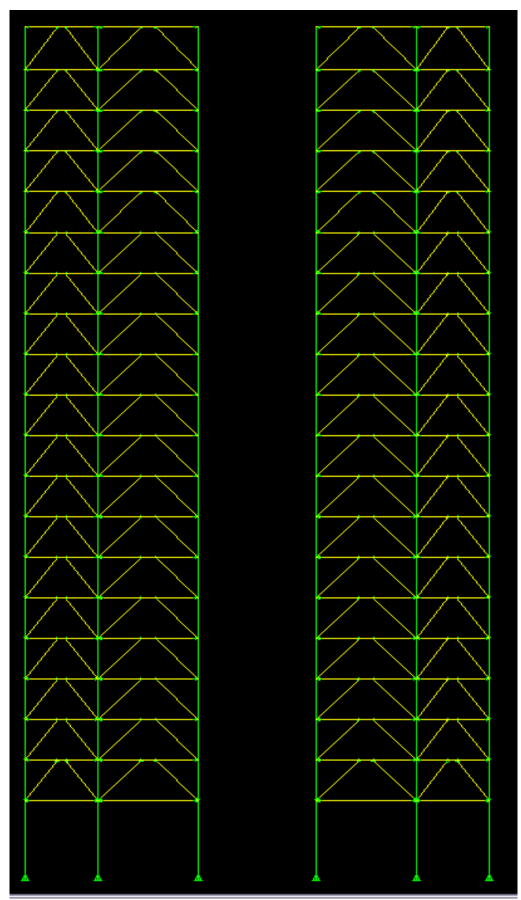
- *North-South direction*



These frames are fairly typical, and run from just below the first floor, up to the roof. They occupy one bay width, 31'-4", from B.8-D.2, and there are six of them. The following frames are similar; EBF 1 and 6, EBF 2 and 5, and EBF 3 and 4. The heaviest column members are located at the bottom, and are a robust W14x665. They progressively get smaller as they reach the roof, where they have fell to W14x106 or W12x159, depending on the gridline. The beams spanning the brace also depend on which EBF it is, but range from W18x71 to W21x122, from top to bottom. Lastly is the bracing, which makes it an eccentric braced frame. These members form an upside down trapezoid with the columns and beams. Their sizes range from W10x88 up to W14x159 at the bottom. On all EBF's, a distance of 3' in the middle creates the eccentricity. This 3 foot section allows for energy dissipation due to cyclical loading from lateral forces.

- *East-West Direction*

There are four of these frames in the E-W direction. They are all similar, and use the same sized members. Two of them are located between 3.3 and 4.9 on gridlines B8 and D2, and the other two are located between 6.1 and 7.8 on gridlines B8 and D2. Columns for these frames are shared with the EBF's in the N-S direction. Beam sizes range from W16x57 to W18x97, and brace sizes range from W8x58 to W14x159. These braces form right-side up trapezoids between columns, the opposite as the N-S. The collector portions of the frames are 2'-6" and 4' and allow for energy absorption from cyclical loading. This eccentricity also allows for doorway and elevator openings in the walls.



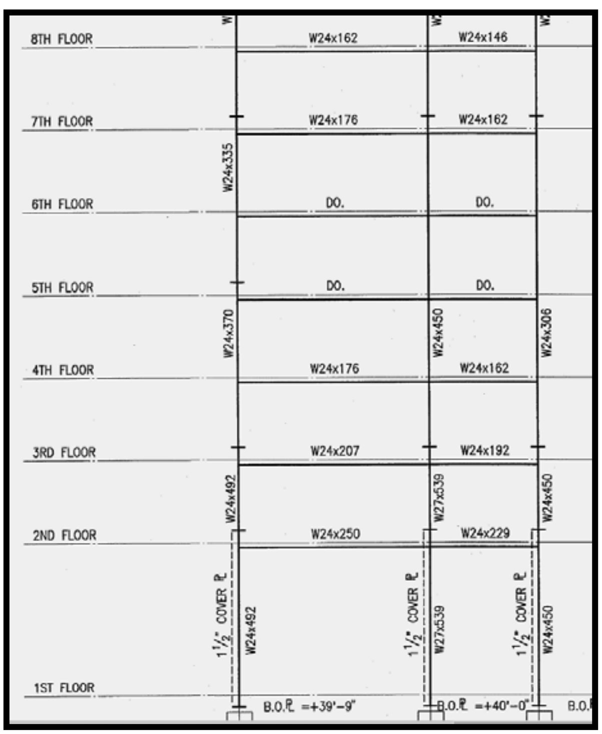
Top: EBF (E/W Faces)
Bottom: EBF (N/S Faces)

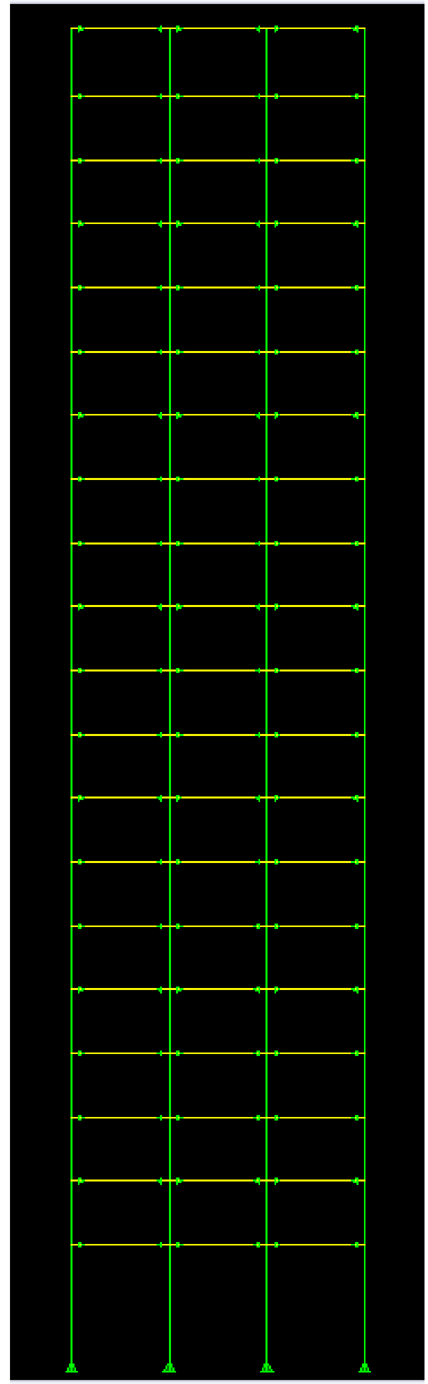
Special Moment Resisting Frames (SMRF)

Moment frames have good ductility and are more flexible than braced frames. All connections within the frames are moment connections. These frames are located on the perimeter walls of the building. Four of these frames are located on the curved portions of the North and South faces, and the other four are on the East and West faces. Two on the E-W faces only go from the first to second floor, as represented by the larger first floor footprint, compared to the upper levels. The other two go all the way to the roof. These frames use only W shapes for beams and columns. Beams for the N-S faces are W24's and the columns range from W24-W33. On the E-W faces, for the frames that reach the roof, there beams are W33's and columns are sized W36's.

Shear Walls

The shear walls provide stiffness to the eccentric frames of the core at the first floor level. The shear walls are located directly under the EBF frames, and occupy the same gridlines. They run from the mat foundation up to the second floor, where they meet the beams of the frames. They are typically 24" thick and reinforced with #6 @ 12" each face each way, unless otherwise noted. 25" thick walls exist on the grid lines D2 and B8. All core shear walls are required to have a f'_c = 5000 psi





Above: SMRF(E/WFaces)
Left: SMRF (N/S Faces)

DEPTH - STRUCTURAL REDESIGN

Proposal:

555 12th Street in Oakland, California is located near a major fault line on the west coast. The design spectral response acceleration parameters for short and long period range are 1.6063 and 0.815 respectively. This creates great demand on the lateral force resisting system; much more than from wind. The dual system used to distribute lateral loads in 555 12th Street gives it a high response modification factor of 8. The use of eccentrically braced frames and moment frames decreases the seismic response coefficient, and in turn, the overall base shear to the building. This type of dual system is required by ASCE code for any building in seismic design category D or E over 160 feet tall. This does not require a peer review during the design phase to take place. This is an effective solution that the designers used based on code requirements and location of the building.

Moment connections for the moment frames and EBF's are costly and labor intensive. Also, the columns and girders in these frames are designed to take large moments along with gravity loads, which create massive members in the frames. It is proposed that a core-only reinforced concrete shear wall design be investigated to eliminate these large members and connections. The aim is to reduce labor/material cost and schedule length. A dynamic analysis will be performed, which will account for the majority of work performed for the redesign. This new system will be designed based on the most recent codes of the IBC, ACI 318, and ASCE-7, as well as supplementary material on seismic design.

Since this building exceeds the height limits of code, it will be designed based on how it performs under certain load applications and analysis procedures. This method of "Performance Based Design" is being pushed by engineers on the west coast, particularly the California Bay area.

LATERAL FORCE RESISTING SYSTEM

Background:

It was proposed to design 555 12th Street with reinforced concrete core-only shear walls. There is a height limitation, for this construction, on seismic design category E buildings to 240 feet which the ASCE7-05, section 12.2.5.4 allows for specially reinforced concrete shear walls. 555 12th Street is 301' tall, so in order to design the building with reinforced concrete shear walls, a design peer review would be required. This peer review would include, but is not limited to:

1. Review of acceptance criteria used to demonstrate the adequacy of structural elements and systems to withstand calculated force and deformation demands, together with that laboratory and other data used to substantiate these criteria.
2. Review of the preliminary design including the selection of structural system and the configuration of structural elements.
3. Review of the final design of the entire structural system and all supporting analyses.

555 12th Street is a high rise structure in a high seismic region. The best analysis for such a building is to perform both a static and dynamic analysis using a computer program. For this analysis, ETABS will be used along with PCA Column to check the shear walls.

Alternative Procedure for Seismic Analysis

There has been a push recently, on the west coast, to adopt an alternative design procedure for tall buildings in high seismic areas. This methodology is performance based and tries to justify a building frame that goes against prescriptive code requirements. Alternative lateral force procedures using rational analyses based on well established principles of mechanics may be used in lieu of those prescribed by code. Any system or method of construction to be used shall be based on a rational analysis in accordance with well-established principles of mechanics. Such analysis shall result in a system that provides a complete load path capable of transferring all loads and forces from their point of origin to the load resisting elements. These criteria are spoken out in sections 1629.10.1 and 1605.2 in the 2001 California Building Code.

The performance objectives that are required by the Alternative Design of tall buildings are provided in the SEAOC BlueBook (C101.1.1 of the 1999 Bluebook) and complimented by the ASCE7-05, 2006-IBC, and FEMA-356 documents. There are three levels of ground motion and performance that are considered is this process. They are summarized in the chart on the following page.

Table 2: Performance Objectives

Level of Earthquake	Earthquake Performance Objectives
Frequent 50% probability of exceedance in 30 years (43year return)	**Serviceability** minimal structural damage; repairable
Design Basis Earthquake (DBE) 10% probability of exceedance in 50 years (475 year return)	**Code Level - Life Safety** moderate structural damage; extensive repairs may be required
Maximum Considered Earthquake (MCE) 2% probability of exceedance in 50 years (2,475 year return) with deterministic limit	**Collapse Prevention** extensive structural damage; repairs are required and may not be economically feasible

These performance objectives are achieved through a 3 step analysis and design procedure summarized in table 3. More detail can be found online or obtaining information from the City and County of San Francisco's Department of Building Inspection.

Table 3: Basic Requirements

Evaluation Step	Ground Motion	Analysis Type	Reduction Factor R	Accidental Torsion	Material Reduction Factors	Material Strength
1	DBE	3D-LDP	2001CBC Table 16-N	yes	2001 CBC	Specified
2	50%/30 years	3D-LDP	1	no	1	Expected
3	MCE	3D-NLRH	N/A	no	1	Expected

LDP-linear dynamic procedure(response spectrum analysis)
NLRH-nonlinear response history

After the analysis and design, the project would be submitted to a peer review panel. An advisory board would need to be implemented to ensure that there is consistency in reviews between all projects that came in.

These guidelines for structural design, analysis, and plan check review of new tall buildings are trying to be established and implemented in code at this very moment in the San Francisco Bay area. The depth work performed in this report can be viewed as a Performance based design and analysis, since it does not prescribe to the code limit on height of 240 feet.

Design Criteria:

As mentioned in the previous section, the design of this building would require it to pass a peer review by an independent company/firm. In order to ensure the highest likelihood of acceptance, 555 12th Street was designed using the following criteria, which go beyond those required by Chapter 21 of ACI-318 2005 and Chapter 12 of ASCE 7-05.

ACI: 21.7.4.4 – For shear walls, V_n shall not be taken larger than $10A_{cv}\sqrt{f'_c}$ for an individual pier, and $8A_{cv}\sqrt{f'_c}$ for all wall piers sharing a common lateral force.

Designed As: V_n *shall not exceed* $4A_{cv}\sqrt{f'_c}$ *for all shear walls*

For coupling beams, V_n shall not be taken larger than $10A_{cv}\sqrt{f'_c}$

Designed As: V_n *shall not be taken larger than* $8A_{cv}\sqrt{f'_c}$

ASCE: 12.12.1 – The design story drift shall not exceed .020hsx from floor to floor and base to roof.

Designed As: The design story drift shall not exceed $0.015h_{sx}$, *and deflection shall not exceed* $0.01h_{sx}$ *from base to roof.*

Preliminary Design:

The first step in design was to remove the EBF's in the core and SMRF's along the perimeter. The building is symmetric about both orthogonal axis, and already utilized this in the original design. Shear walls were placed in the north-south direction in the core where EBF frames 2, 3, 4, and 5 were located. Shear walls in the east-west orientation were then placed to form four similar concrete core I beams. These walls would be connected with coupling beams.

An arbitrary thickness of 24" was selected as a starting point for the walls and coupling beams. This produced an area of 668 sq ft of shear walls per floor, for a total of 2.7% of the total floor area. A pound per square foot of the shear walls was determined and added to the weight of a typical floor. Through Equivalent Lateral Force procedures of ASCE 7-05, the weight of the structure was determined, along with an overall base shear for the building. This base shear was then substituted into the limiting shear stress criteria equation, shown below with an $f'_c = 10000$psi. It was assumed that each wall in the N/S direction took total base shear/4 since they had equal length and thickness.

$$K = \frac{V_u / \Phi \times 1000}{A_{cv}\sqrt{f'_c}} \leq 4$$

This resulted in K < 4, so this design was investigated further. The same equation was used to test walls in the E/W direction which also passed the criteria.

Another preliminary check that was performed was the determination of a target period of vibration and moment of inertia required for the building. A T = 2.7 seconds was determined through calculation and plugged into the following equation to find a required moment of inertia (I) in the north/south direction.

$$T = \frac{2\pi}{3.52}\sqrt{\frac{\mu \times H^4}{EI}}$$

An I_{req} was determined and compared to that supplied by the trial sized shear walls. The $I_g > I_{req}$, so the preliminary design was pushed into the analysis phase. It should be noted that an increase in period, will ultimately decrease the dynamic design forces the building will see. This can be accomplished by decreasing the moment of inertia from shear walls.

The preliminary design assumed a 4 foot depth for the coupling beams to give 9 foot doorway openings in the shear walls.

ETABS ANALYSIS

After preliminary shear wall sizes and locations were determined, an ETABS model was set up for analysis. The purpose of the model was to determine how shear was actually distributed to each lateral element, including the coupling beams. From this, the beams and walls can be detailed with hand calculations.

Static Forces and Analysis:

The final lateral forces determined with correct floor and shear wall weights were determined using the equivalent lateral force procedure in ASCE7-05 Chapter 12. The variables involved and results are presented below. Calculations are Available in appendix.

SEISMIC	
Ss	2.4095
Sl	0.9405
Site Class	C
Fa , Fv	1.0 , 1.3
Importance Factor	I = 1.0
Sds	1.6063
Sdl	0.815
Seismic Design	E
R	5
Cd	5
Ω_o	2.5
Cu	1.4
Ta	1.445
T	2.02
k	1.76
Cs	0.094

Total Weight (W) = **68454 kips**
Seismic Base Shear (V) = **6435 kips**
Overturning Moment = **1,364,157 ft-kip**

Building located at Latitude: 37.804603
Longitude: -122.275486

The center of mass and center of rigidity of the structure coincide at the same point on the typical floor plan. This fact allows the building to be designed without any inherent torsion in either direction. However, by code, 555 12th Street is designed to account for an accidental torsion of 5% of the diaphragm length in both orthogonal directions. It was then modified to account for Ax, the accidental torsion amplification factor. Ax was determined from checking 4 load cases and taking the max Ax, and multiplying by .05. Ax was determined with the following equation:

$$Ax = \left(\frac{\delta \max}{1.2 \cdot \delta avg} \right)^2$$ Where Ax = 1.07 (.05) = .0535 for accidental torsion

This eccentricity ratio is used for both static and dynamic load cases in each direction.

	Accidental Torsion Amplification					
Floor	Load Case	δ_1	δ_2	$\delta \max$	δavg	Ax
11th	autoEXZ1	2.7975	2.3441	2.7975	2.5708	**0.8223**
	autoEXZ2	2.3441	2.7975	2.7975	2.5708	**0.8223**
	autoEYZ1	3.656	6.0164	6.0164	4.8362	**1.0747**
	autoEYZ2	6.0164	3.656	6.0164	4.8362	**1.0747**
Roof	autoEXZ1	5.3013	4.6068	5.3013	4.95405	**0.7952**
	autoEXZ2	4.6068	5.3013	5.3013	4.95405	**0.7952**
	autoEYZ1	10.2216	14.4305	14.4305	12.3261	**0.9518**
	autoEYZ2	14.4305	10.2216	14.4305	12.3261	**0.9518**

Load Cases:

Several load cases were investigated during the static analysis of the structure:

<u>ELFX /ELFY:</u> These correspond to the story forces found by manual calculation using the equivalent lateral force procedure. Story forces are input acting in the x and y directions for ELFX and ELFY respectfully. They are input to act at the center of mass, with accidental torsion ratio of .0535.

Story	Forces(k)	Story	Forces(k)
Roof	265	11	267
mezz	133	10	229
mech	939	9	193
19	663	8	160
18	605	7	129
17	550	6	102
16	497	5	77
15	446	4	55
14	398	3	37
13	352	2	22
12	308	1	12

<u>AUTO:</u> There are four such cases that relate to the AUTO name. They are load cases that use the IBC 2000 and user input values for eccentricity, period, response modification factor, long and short spectral response parameters, and site coefficients. They account for the following earthquake loading conditions:

EXZ1 – X direction loading with positive eccentricity in the Y direction
EXZ2 - X direction loading with negative eccentricity in the Y direction
EYZ1 - Y direction loading with positive eccentricity in the X direction
EYZ2 - Y direction loading with negative eccentricity in the X direction

The values of ELFX and ELFY were used to scale the dynamic load cases in the next section. No design was based solely off the static load cases and forces.

Wind Loads

It was determined in previous work that wind is not the governing lateral force. When the total base shear from wind was calculated and multiplied by a factor of 1.4, it was still less than the total base shear from seismic. Loads were calculated in both the North/South and East/West directions. The building has slight irregularities, so several assumptions were made in determining base shear:

Assumptions:

- Assume building does not have curved façade, that is is rectangular.
- Assume no canopy and 1st floor is same footprint as all floors
- Height is 306 feet, North and South face = 227’, East and West face = 125’
- Parapet at roof was ignored, and made into the top of the roof @ 306’

Velocity Pressure, q_z was calculated = $0.00256*k_z*k_{zt}*k_d*(V^2)*I$

Final Pressure, P was calculated = $q_z(GC_p)-q_i(GC_{pi})$

Building:		555 12th Street	Reference
Location:		Oakland, CA	
Basic Wind Speed(mph):	V	85	Fig. 6-1
Exposure Category:		C	6.5.6.3
Enclosure Class		Enclosed	Sect. 6.2
Building Category		II	Table 1-1
Importance Factor:	Iw	1.0	Table 6-1
Topography Factor:	Kzt	1	Sect. 6.5.7
Directionality Factor:	Kd	0.85	Table 6-4
Internal Pressure Coefficient:	Gcpi	0.18	Fig. 6-5
Gust Factor- assume rigid	G	0.85	6.5.8.1
Building Height:	h (feet)	306	
Length Parallel to wind:	L (feet)	227'	
Length Perpendicular to wind:	B (feet)	125'	

From the total pressures, shear at story heights, base shear, and overturning moment were found.

East/West:	Base shear = V = 1105 kips
	Overturning Moment = 182,196 ft-kips
North/South:	Base Shear = V = 2286 kips
	Overturning Moment = 375,334 ft-kips

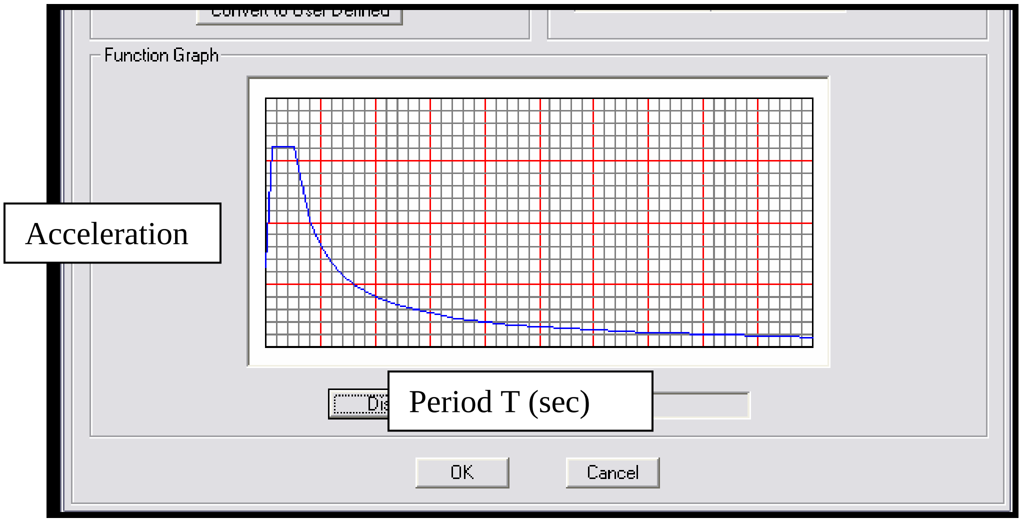
These are merely for comparison purposes only, and not used in any design.

Dynamic Forces and Analysis:

A modal response spectrum analysis was conducted on the building to determine the natural modes of vibration for the structure. The analysis took into account 12 modes so as to obtain a combined modal mass participation of at least 90% of the actual mass in each of the orthogonal directions. The combined response parameter used in this analysis, by the computer, for the modes was the complete quadratic combination equation (CQC).

Response Spectrum Load Cases

Four load cases were defined for dynamic analysis. Each load case scaled design values of the combined response differently, depending on its purpose. Each load case uses a user defined response spectrum based on the IBC and values of S_{ds} and S_{dl}. In this case they are 1.6063 and 0.815 respectively. This creates a function graph which will be used for analysis.



Response Spectrum Function for Dynamic Analysis

DFX: This load case was used to determine shear and moment forces on individual beams or piers when an earthquake acted in the X – direction. This load case accounted for an eccentricity of 0.0535 and damping of 5%. This was originally scaled to 386, before it was scaled down to output the same base shear as the ELFX static load case. The 386 is acceleration due to gravity (in/sec^2) = g. After the down scale, it was multiplied by a factor of 0.85. This is the advantage to using a dynamic analysis, in that you can decrease your total base shear by 15% when analyzing. The final scale factor used was 66.32.

$$\text{ASCE7 – section 12.9.4: Scale Factor} = \left(\frac{Velf}{Vdyn}\right) \times g \times 0.85$$

DFY: This is the same as DFX, except the earthquake is acting in the Y – direction. The final scale factor used was 73.51.

DDX: This load case was used to determine drift and deflection of the entire structure while subject to an earthquake in the X direction. It uses an eccentricity of 0.0535, and damping of 5%. The response spectra function is scaled as follows:

$$\text{Scale Factor} = g \times \left(\frac{C_d}{R \cdot I}\right) = 386$$

DDY: This is the same as DDX, except the earthquake is acting in the Y – direction. The scale factor is 386. The dynamic drift found from these load cases is comparable to performing a static analysis to find deflection, then multiplying by the dynamic amplification factor (C_d). However, the actual dynamic analysis will give you the real response of the structure.

P- Delta Effects: The increased story forces and deflections from vertical loads were automatically accounted for in this analysis with the computer program.

Modeling Process

The model used for analysis consisted of the lateral force resisting elements, rigid diaphragms, and lumped masses. The shear walls and coupling beams were added first in their preliminary locations, and later changed according to new calculations. They were input as having zero mass properties A point mass was placed at the center of mass of each floor and included all dead load acting on that floor from shear walls, composite floor, and curtain wall. Also accounted for at this center of mass was a rotation about the Z moment of inertia, calculated as follows:

$$J = \left(\frac{\sum (Ix + Iy)}{mass} \right) \times Area$$ Where all variables are based off the floor plate

Rotation About Z axis					
	Ix	Iy	Area	Mass	J
roof	9.86E+10	1.03E+12	1955527	2.581	1489450
mezz	9.86E+10	1.03E+12	1955527	1.374	792912
mech	4.78E+11	1.61E+12	3291192	10.964	6955787
typ	4.78E+11	1.61E+12	3291192	8.48	5379887
2nd	7.20E+11	4.16E+12	4598151	9.723	10321435

This will have an effect on torsion and the first mode shape of the building. There are two underground levels of parking that were modeled to resist all movement in the X and Y direction and rotation about the Z axis. A spring force = 1e+30 k/in was used for each of these directions. The rigid diaphragm for each floor extended the area and shape of the floor plate on that floor. From this model, all dynamic forces and moments were found and used in calculations after numerous variations of the model were run.

Modes of Vibration for structure

1st: Movement in Y Direction : T = 2.31 sec

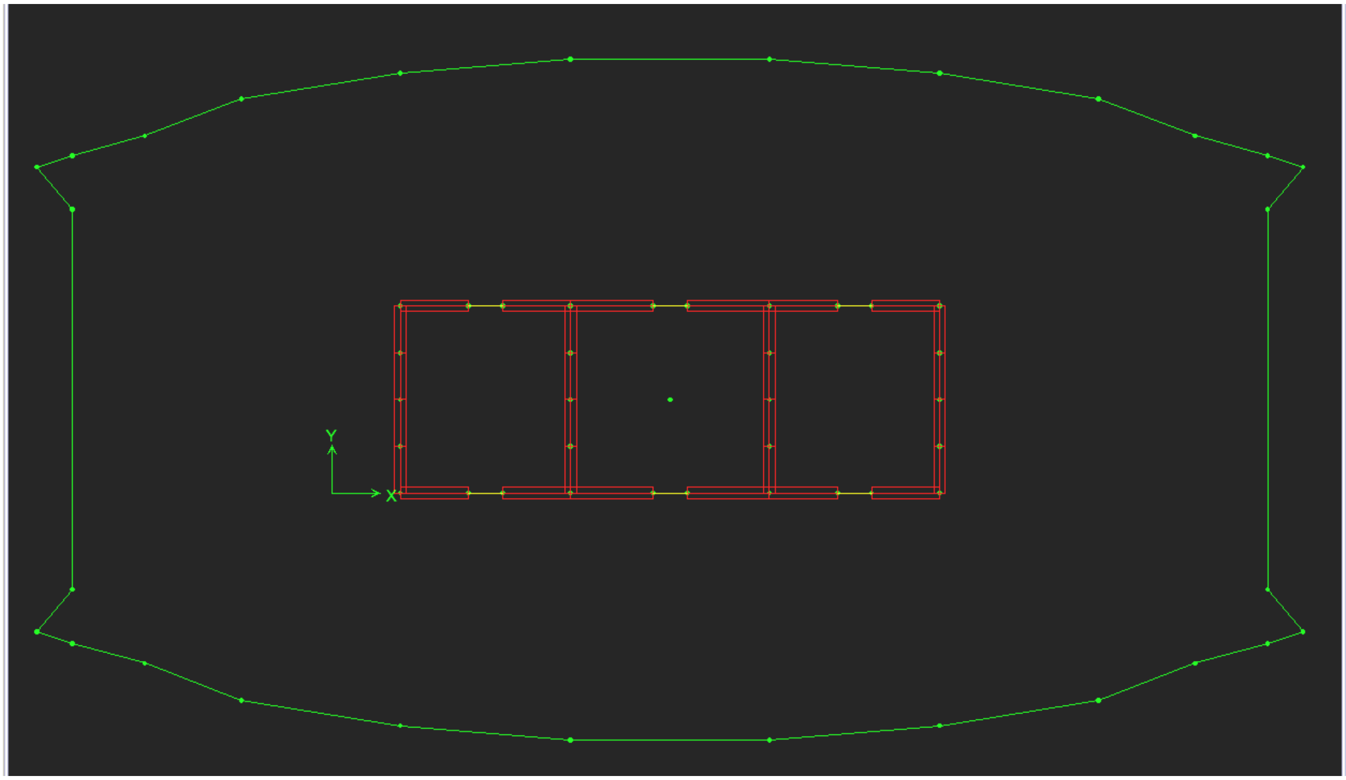
2nd: Torsional rotation : T = 2.01 sec

3rd : Movement in X direction T = 1.58 sec

4th : Torsional rotation : T = 0.54 sec

The first mode of vibration will have the greatest effect on forces and displacement, and each successive mode has less impact on total force and displacement. The target period from the ASCE, was found to be 2.02 sec. The target period from the first preliminary design was 2.7 seconds.

Finalized Dynamic Loads and Layout



The proposed concrete shear wall system was designed to provide adequate strength, stiffness, and energy dissipation capacity to withstand the design ground motions within the prescribed limits of deformation and strength demand. Collector elements were designed to adequately transfer seismic forces originating in other portions of the structure to the shear walls, which provide resistance to those forces. As previously mentioned, numerous models were run until the layout of the lateral system met all of the design criteria. Below is a summary of the major aspects of design.

Coupling Beams

- All have 6 foot span
- All are 3 feet deep
- All have Effective moment of inertia of 0.11 I gross.
- All have f'_c = 9000 psi

Shear Walls

- All are 24 inches thick
- All have f'_c = 9000 psi
- 2 similar I beam walls with Area = 111 ft^2 each
- 2 similar C shape walls with Area = 82 ft^2
- f_{22} = 0.5 to accounting for cracked section properties

Also determined from this final design was the redundancy factor of the system. A more redundant structure implies a more ductile structure. The redundancy factor depends on:

• Number of lateral-load-resisting-elements in the building
• Plan area of the building
• The ratio of the lateral force in a lateral-load-resisting element to the total lateral shear at a particular story level

For Seismic design category D, E, or F: $\rho = 2.0 - \left(\frac{20}{r_{\max}\sqrt{A_i}}\right) \geq 1.0 \leq 1.5$

r_{max} = maximum element-to-story shear ratio = $\left(\frac{V_{\max}}{V_{total}}\right) \times \left(\frac{10}{L_{wall}}\right)$

This calc would be taken from a story in the bottom 2/3 of the building. A calculation was taken from the 2nd story where V_{max} = 1961 kips, V_{total} =7684 kips, L_w = 33' and A_i = 24000ft^2.

ρ = 0.33 :: use ρ = 1.0 for all calculations.

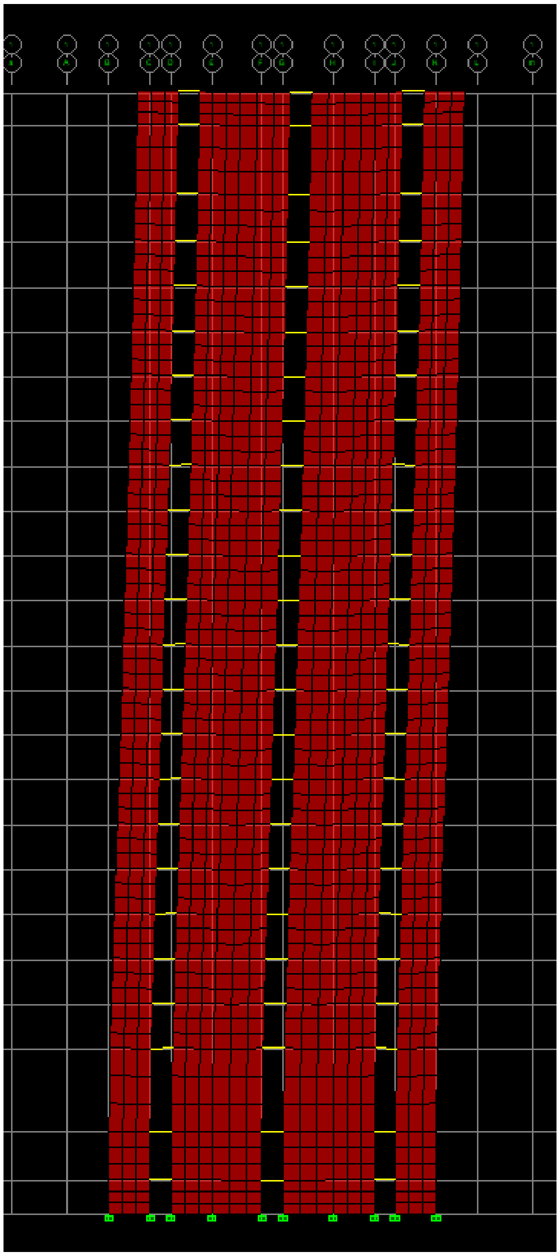
This means the system is redundant enough that if on of the elements failed, the other elements would still be able to hold up the structure. A guideline is usually if a lateral force resisting element takes more than 33% of the load on one direction, than a higher redundancy factor must be used. This provision in the IBC encourages designers to use a greater number of LFR elements, or be penalized.

Seismic Story Drifts:

From the load cases DDX and DDY, dynamic drift and deflection were found for each story, and entire building. The most critical direction for drift was in the Y direction (N-S). Results were taken from the center of mass of each floor as described in section 12.8.6 of ASCE7-05. The results are shown below:

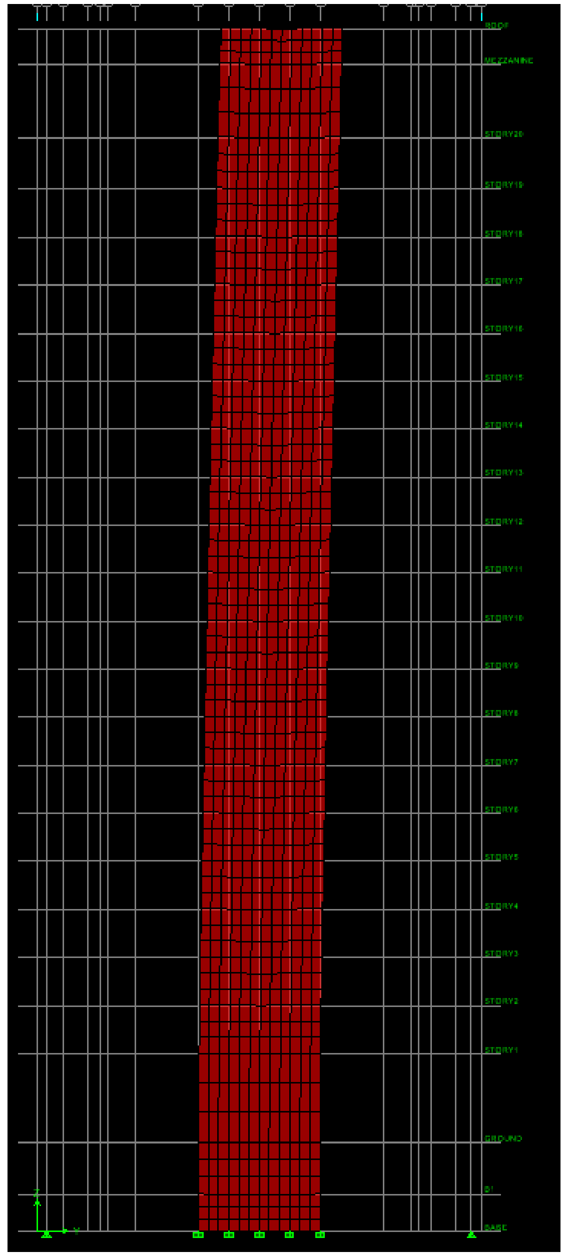
From Earthquake in X direction

STORY	DISP-X	DRIFT-X
ROOF	18.837292	0.004031
MEZZ	18.38748	0.004237
STORY20	17.418563	0.004561
STORY19	16.711318	0.004863
STORY18	16.00557	0.005162
STORY17	15.262037	0.005439
STORY16	14.481937	0.005681
STORY15	13.667972	0.005883
STORY14	12.823541	0.006044
STORY13	11.95222	0.006165
STORY12	11.05746	0.006252
STORY11	10.14244	0.00631
STORY10	9.210172	0.006344
STORY9	8.263686	0.006355
STORY8	7.306413	0.006342
STORY7	6.342678	0.006298
STORY6	5.378318	0.00621
STORY5	4.421474	0.006058
STORY4	3.483636	0.00581
STORY3	2.581051	0.005424
STORY2	1.736653	0.004839
STORY1	0.982415	0.003411
GROUND	0	0
B1	0	0



From Earthquake in Y direction

STORY	DISP-Y	DRIFT-Y
ROOF	30.735	0.01096
MEZZ	29.4792	0.01095
STORY20	26.8858	0.01098
STORY19	25.1039	0.01097
STORY18	23.4235	0.01094
STORY17	21.7514	0.01087
STORY16	20.0919	0.01076
STORY15	18.4503	0.01061
STORY14	16.8319	0.01041
STORY13	15.2422	0.01018
STORY12	13.6872	0.00989
STORY11	12.1728	0.00957
STORY10	10.7052	0.0092
STORY9	9.2909	0.00879
STORY8	7.93682	0.00833
STORY7	6.65039	0.00782
STORY6	5.43972	0.00726
STORY5	4.31368	0.00664
STORY4	3.28208	0.00596
STORY3	2.35569	0.0052
STORY2	1.54637	0.00436
STORY1	0.86679	0.00301
GROUND	0	0
B1	0	0



Design criteria set earlier put the overall building deflection to .01 x height = 36" base to roof. The max deflection found was 30". Also, the max inter-story drift was set to .015 x h_{sx}. The maximum drift found was 0.011. Both criteria were passed for design.

Coupling Beams:

The coupling beams connecting structural walls provide stiffness and energy dissipation. They are designed to crack before the shear walls and act as plastic hinges in the building. To account for this, a lower limit of the effective moment of inertia was found based on the following equation:

$$I_{eff} = \frac{0.2}{\left(1+3\left(\frac{t}{l_n}\right)^2\right)} \times I_{gross} \quad = \text{ 1/9 } I_{gross} \text{ for beams used.}$$

The lower the I_{eff} used, the less shear the coupling beams would see as reaction forces. By cracking the beams, they begin to act plastically, which produces a probable flexural strength, M_{pr}. The yielding strength of the steel increases by a factor of 1.25, from f_y=60ksi to f_y=75ksi. Thus, the M_{pr} is greater than the original capacity. This in turn, increases the shear experienced by the beam at the plastic hinge points. The beam must be designed to account for this increase in forces. Dynamic analysis provided the shear in each coupling beam at every story. Beams 7,9,10 and 12 were shown to take the same loading while beams 8 and 11 showed the same loading.

The coupling beams were first designed as 4 feet thick, but were changed to 3 feet to make detailing simpler. ACI 318 Ch 21.7.7 requires any coupling beam with (span/depth) < 2 and Vu > $4A_{cv}\sqrt{f'_c}$ to be reinforced with two intersecting groups of diagonal reinforcement. Decreasing the thickness put the beam right at a ratio of 2.0, so it was then designed as a beam member in a moment frame. Also, the diagonal reinforcement is most effective if placed at a steep angle. If diagonal reinforcement were used in this design, it would only be at an angle of 26 degrees. So, each bar would be taking less than half the capacity, if the shear reinforcement were placed vertically. Another reason to not use diagonal reinforcing is under a peer review, the rotation of these bars under max loading would be closely looked at if rotation angle was large.

Beams were then designed for shear based on the assumption of load sharing. An average shear was found for a group of stories and compared to 0.8 * Vmax of that group. This set of coupling beams was then designed based on 0.8 *Vmax. The 20% reduction is allowed by code if this procedure is followed. Conservatively, each set was eventually designed based on the max of the average or 0.8Vmax.

Beam 7,9,10,12			**Beam 8,11**		
Floors	**0.8*Vmax (kips)**	**Vavg**	**Floors**	**0.8*Vmax (kips)**	**Vavg**
16-Roof	**144**	121	16-Roof	**206**	183
8 to 15	225	**241**	8 to 15	306	**334**
1 to 7	244	**290**	1 to 7	322	**381**

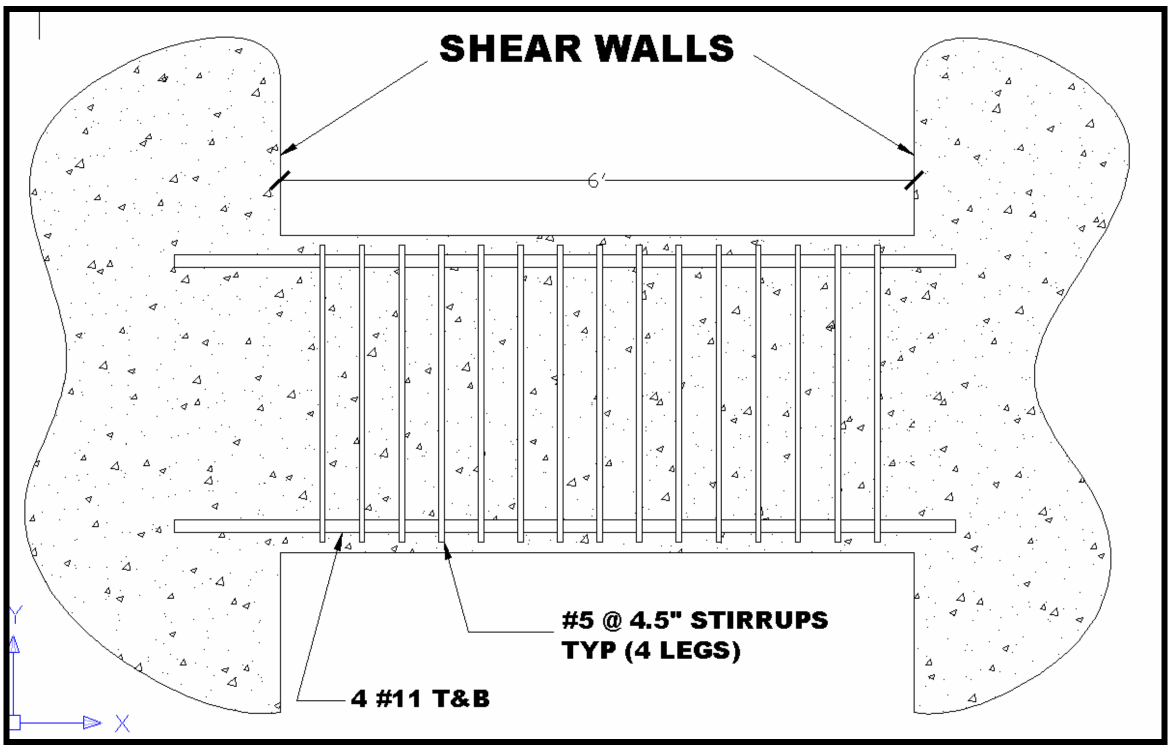
A summary of the beams with reinforcing is placed below. Beams did not require skin reinforcing because they were not greater than 36" deep.

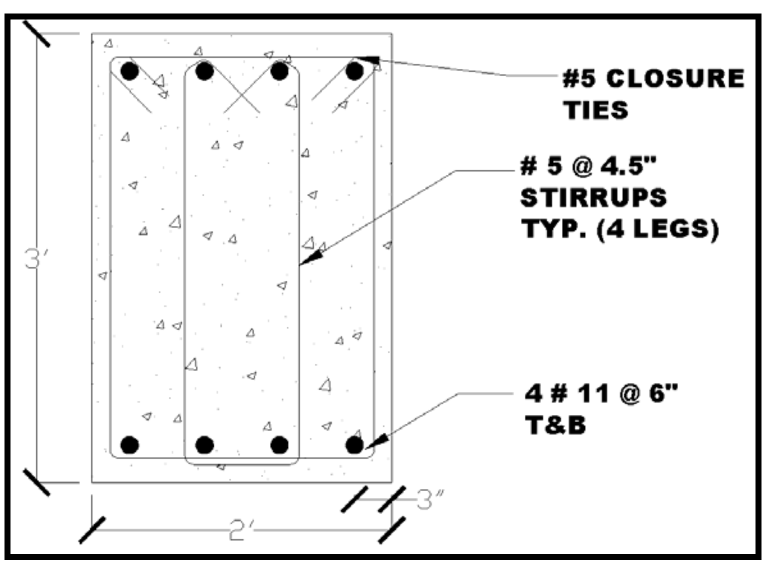
COUPLING BEAM SCHEDULE

Member	Width (in)	Depth (in)	Long. Reinf Top	Long. Reinf. Bottom	Skin Reinf.	Shear Reinf.
Floors 1-7						
B7	24	33	4-#11	4-#11	NONE	4 legs #5 ties @ 4.5"
B8	24	33	5-#11	5-#11	NONE	5 legs #5 ties @ 4.5"
B9	24	33	4-#11	4-#11	NONE	4 legs #5 ties @ 4.5"
B10	24	33	4-#11	4-#11	NONE	4 legs #5 ties @ 4.5"
B11	24	33	5-#11	5-#11	NONE	5 legs #5 ties @ 4.5"
B12	24	33	4-#11	4-#11	NONE	4 legs #5 ties @ 4.5"
Floors 8-15						
B7	24	33	4-#10	4-#10	NONE	4 legs #5 ties @ 5.5"
B8	24	33	5-#11	5-#11	NONE	5 legs #5 ties @ 4.5"
B9	24	33	4-#10	4-#10	NONE	4 legs #5 ties @ 5.5"
B10	24	33	4-#10	4-#10	NONE	4 legs #5 ties @ 5.5"
B11	24	33	5-#11	5-#11	NONE	5 legs #5 ties @ 4.5"
B12	24	33	4-#10	4-#10	NONE	4 legs #5 ties @ 5.5"
Floors 16-Roof						
B7	24	33	4-#8	4-#8	NONE	4 legs #5 ties @ 8"
B8	24	33	4-#9	4-#9	NONE	4 legs #5 ties @ 7"
B9	24	33	4-#8	4-#8	NONE	4 legs #5 ties @ 8"
B10	24	33	4-#8	4-#8	NONE	4 legs #5 ties @ 8"
B11	24	33	4-#9	4-#9	NONE	4 legs #5 ties @ 7"
B12	24	33	4-#8	4-#8	NONE	4 legs #5 ties @ 8"

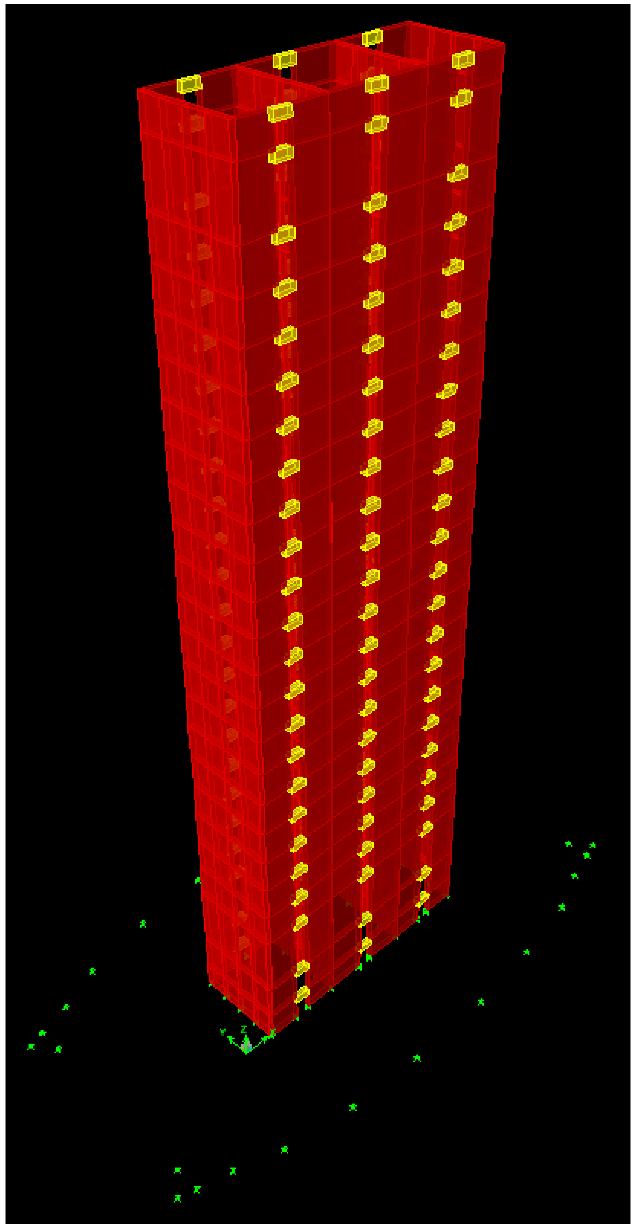
Coupling Beams are the most important element designed in this lateral force resisting system. When they begin to act plastically, their shear and moment demand increase dramatically. If they are not detailed accordingly, then these members will lose structural integrity and discontinue performing their intended purpose. The system will lose the rigidity that the members brought to the core unit, and could result in failure of the structure under extreme lateral loads.

A typical detail of reinforcing in Beam 7 from the 1st – 7th floor is presented to show confinement and spacing of shear and flexural reinforcement.





Shear Wall Design:

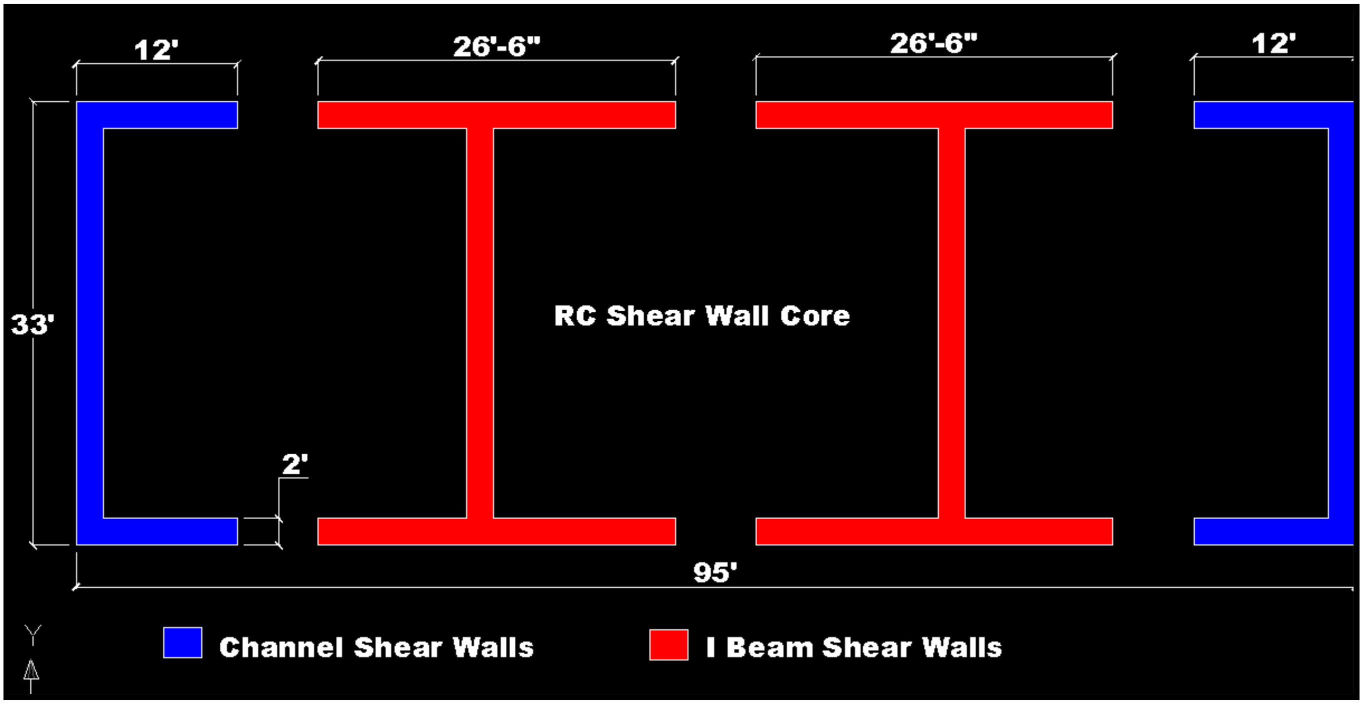


Shear walls were made 2 feet thick for every wall. The final design utilized two similar C shapes and 2 similar I shapes. All shear walls are load bearing, so an axial – moment interaction analysis was performed for each shape. Tributary areas were determined for each shape, and dead loads from self weight and floor area were added together. Moment at the seismic base in the X and Y direction were determined through dynamic analysis.

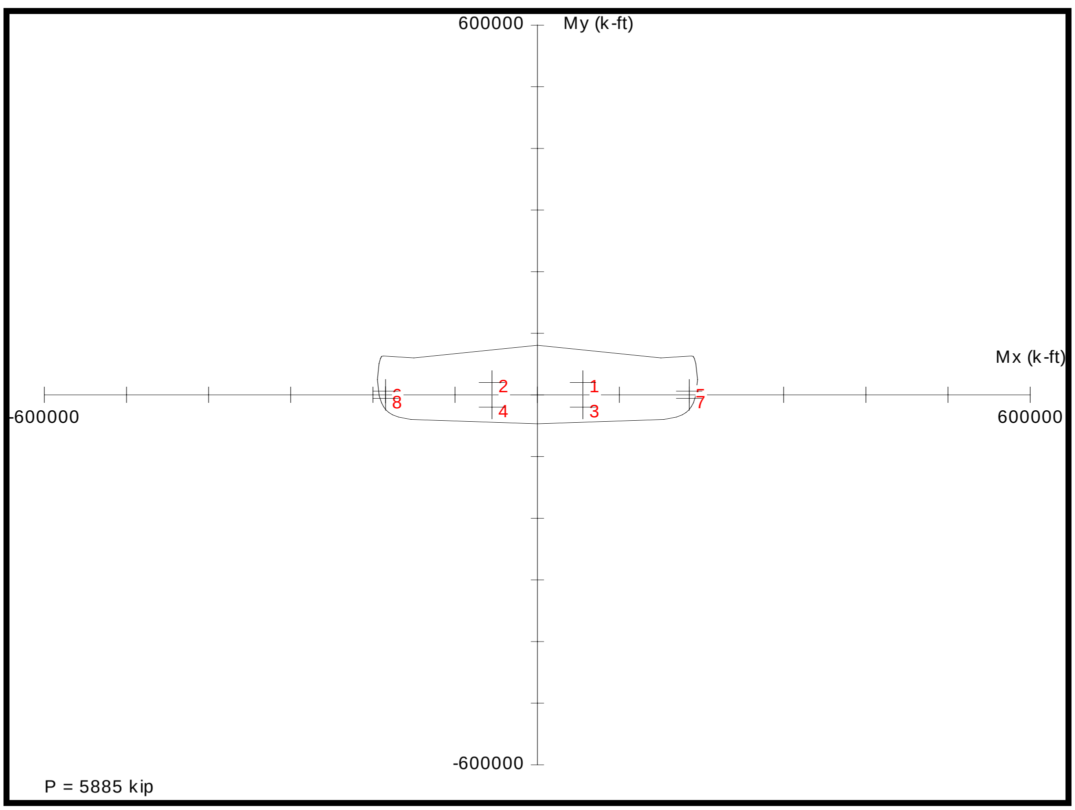
With this information, PCA column was used to determine the amount of rebar needed to resist bending about both axes. The following load cases were input and checked in the program:

1) 0.7D + 1.0Ex + 0.3Ey
2) 0.7D + 1.0Ex - 0.3Ey
3) 0.7D - 1.0Ex + 0.3Ey
4) 0.7D - 1.0Ex - 0.3Ey
5) 0.7D + 1.0Ey + 0.3Ex
6) 0.7D + 1.0Ey - 0.3Ex
7) 0.7D - 1.0Ey + 0.3Ex
8) 0.7D - 1.0Ey - 0.3Ex

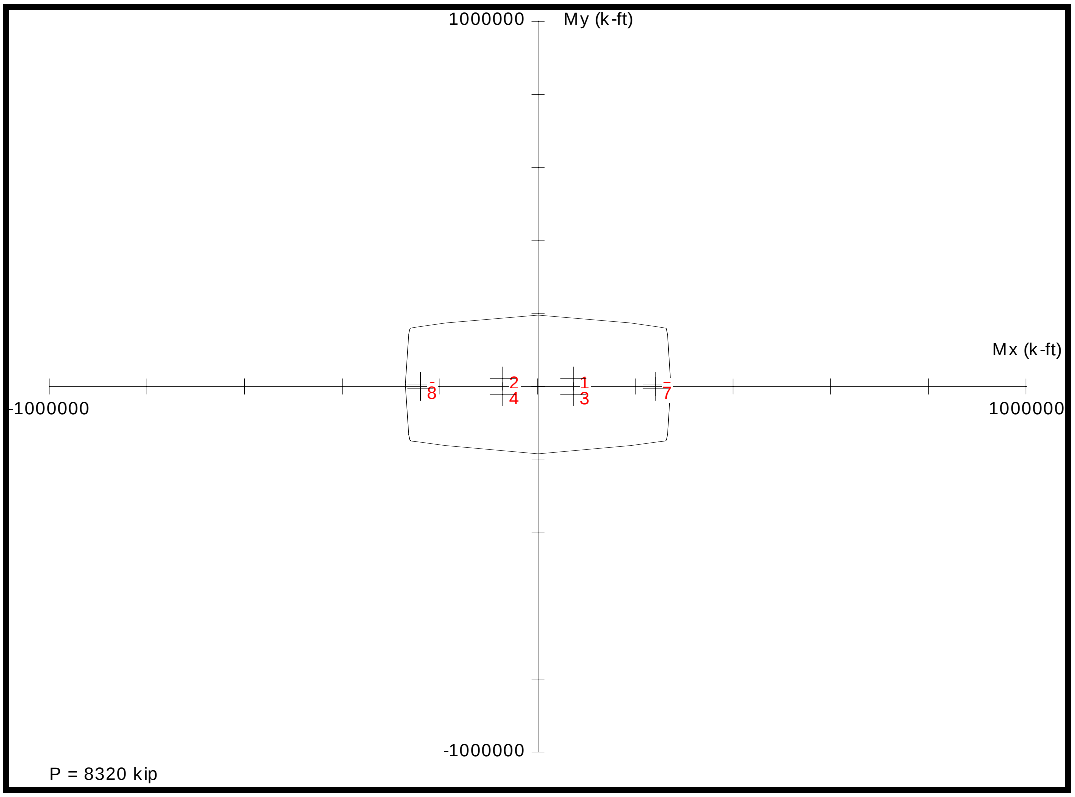
The Ex refers to the moment about the Y axis (My), and Ey refers to the moment about the X axis (Mx). The 0.7D was taken from 0.9D – Sds*D. This is a conservative design approach because accounting for all the dead weight on the wall will effectively reduce the moment induced, and amount of rebar needed for flexure. P-M and moment interaction diagrams are shown on the next few pages for both types of shear walls.



The RC shear wall core is laid out above, with various dimensions. Both I Beam Walls are the same and both Channel Walls are the same dimensions and properties.



Mx-My diagram for C shear walls show all 8 load cases within the "envelope" of moment capacity of the entire cross section. This is for an axial load of 5885 kips.



Mx-My diagram for I beam walls show all 8 load cases within the "envelope" of moment capacity of the entire cross section. This is for an axial load of 5885 kips.

The rebar needed for flexure will be placed in the boundary zones of the walls, which are located at the ends and intersections with perpendicular walls. These areas were not designed in detail, but an area of steel needed is provided.

Section	Total As (in^2)	Web (in^2)	Flanges (in^2)
I beam	158	22	68 each
Channel	117	33	42 each

Shear reinforcing for each wall was checked using the equation for nominal shear capacity. Each individual length of wall was looked at in each direction.

$$V_n = A_{cv}\left(\alpha_c\sqrt{f'_c} + \rho_t f_y\right) \text{ where } \alpha_c = 2.0 \text{ for h/l > 2.0}$$

Each wall required only the minimum amount of shear reinforcing of ρ_t = .0025 each way, both faces except both C channel webs, which required ρ_t =.003. However, all walls will use #6 @ 12" each way, each face to satisfy required steel ratio for shrinkage and temperature, and strength.

GRAVITY SYSTEM REDESIGN

The implementation of a reinforced concrete shear wall core led to the elimination of all eccentrically braced frames in the core, and perimeter moment frames. These members were originally designed to take gravity loads and lateral loads, and were much larger in weight/size than their gravity only counterparts. Using RAM Structural System, a model of the building was created to redesign the columns and girders in question. Gravity loads were the only loads input into the model, because these members were no longer required to resist lateral loads. The following design gravity loads were used in the model:

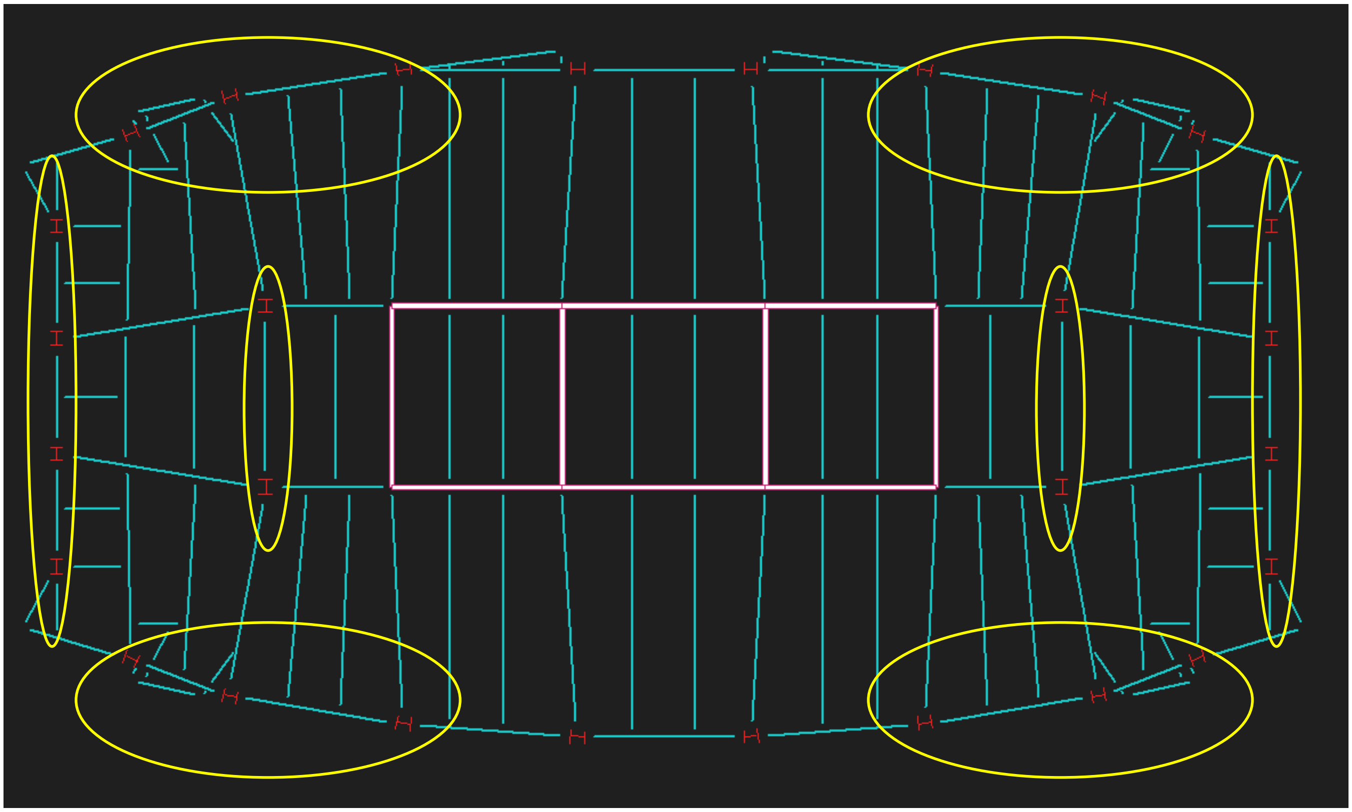
Dead Loads:

MEP	10psf
Structural Framing	15psf
Collateral	5psf
Typical Floor Concrete/Deck	50psf
Roof LWT Concrete/Deck	39psf
2nd Floor Concrete/Deck	69psf
Mechanical Floor/Deck	100psf + Equipment Pads
Mezzanine	100psf
Curtain Wall Line Load	0.325 klf

Live Loads: Taken from table 1607.1 from IBC 2003

Roof:	20psf
Office Floor:	80 psf
Partitions:	20 psf
Plaza, Lobby, Corridors, Stairs:	100 psf
Parking:	50 psf
Loading Dock/Court:	250 psf
Storage:	125 psf

Several load combinations were also input, with the governing being 1.2Dead + 1.6Live. Members were designed based on the LRFD method, 3rd edition, and designed to act compositely with the deck and slab, just like the original system. Live load reductions were accounted for, as allowed by IBC code section 1607.9. Deflection criteria were based off of l/240 and l/360 for total load and live load for serviceability.



Typical floor: Circled elements are columns and girders that were redesigned

Moment frames on north and south perimeter have the same member sizes, while moment frames on the east and west perimeter are similar as well. The two bays of interior columns that changed are also similar, and use the same sized members in the redesign.

EBF1 and EBF6: These are the two interior core frames. Columns in these bays ranged from W14x43 up to W14x370 at the base. Girders changed to W16x26 with 60 shear studs on typical floors.

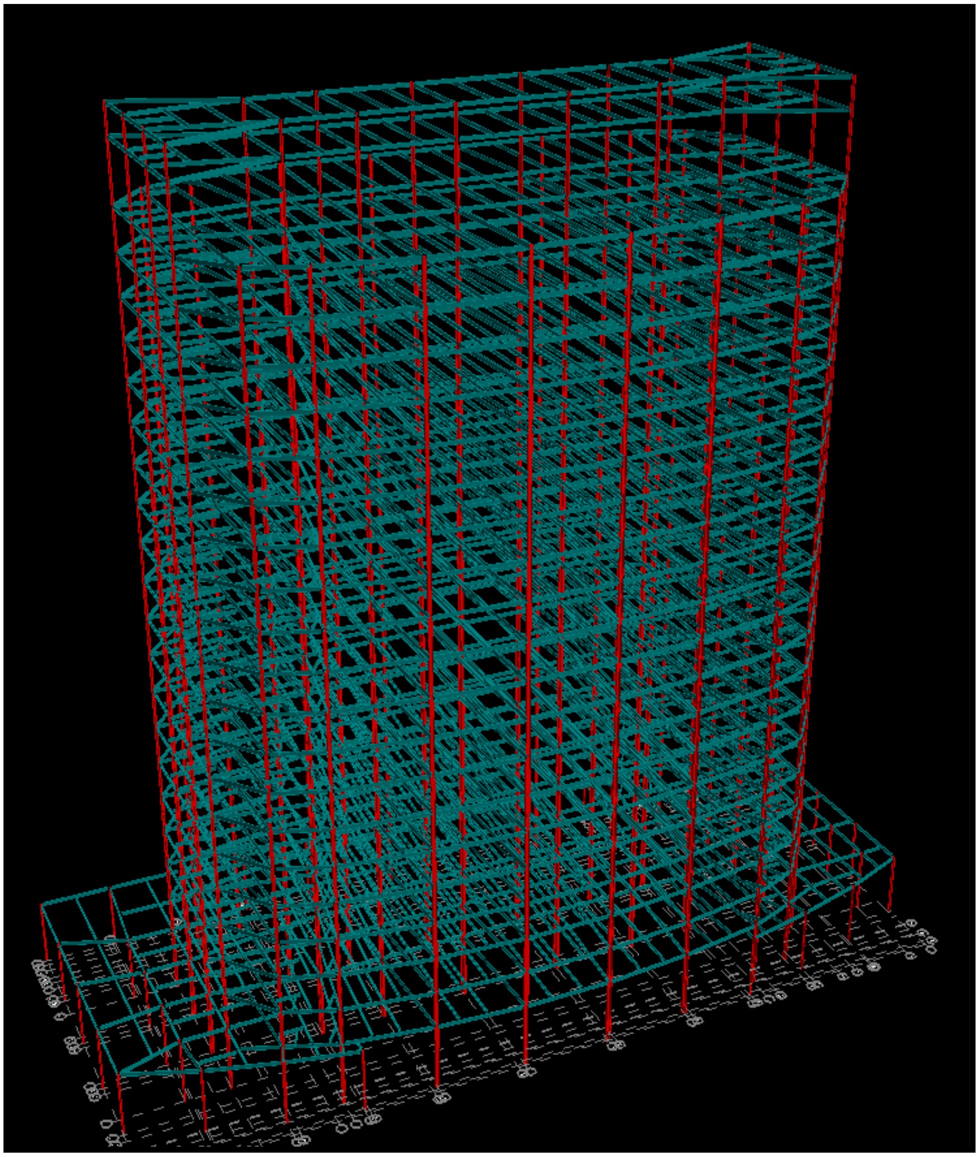
SMRF1, SMRF2: These are the frames on the east and west perimeter. Columns ranged from W12x40 up to W12x106 for the two exterior bay columns and from W14x43 up to W14x233 on the interior columns. Girders changed to W10x12 with 18 shear studs on typical floors.

SMRF3,4,5 and 6: These frames span two bays on the perimeter on the north and south walls. All three sets of columns for each frame were W14's ranging in weight from 43 up to 283. For a typical floor, there were two girder sizes used. For the longer span, W21x44 with 25 shear studs were used, while W16x31 with 12 shear studs were used for the shorter span.

SMRF7, and 8: Located on the first floor only, supporting the larger roof. All columns became W10x33's and all girders became W8x10 with 7 shear studs.

Shear Walls: The reinforced concrete shear walls designed for the lateral system were also designed to carry dead and live load from the structure. The axial – moment interaction of these walls is presented in the lateral system redesign section of this report.

The redesign of the EBF's and SMRF's to gravity only frames yielded much smaller members for girders and columns. A complete spreadsheet of all column and girder sizes is available upon request. Also, all moment connections that the original frames had were replaced with simple shear connections. The cost and schedule impact will be investigated in the breadth sections.



Three Dimensional Steel superstructure from RAM Structural System

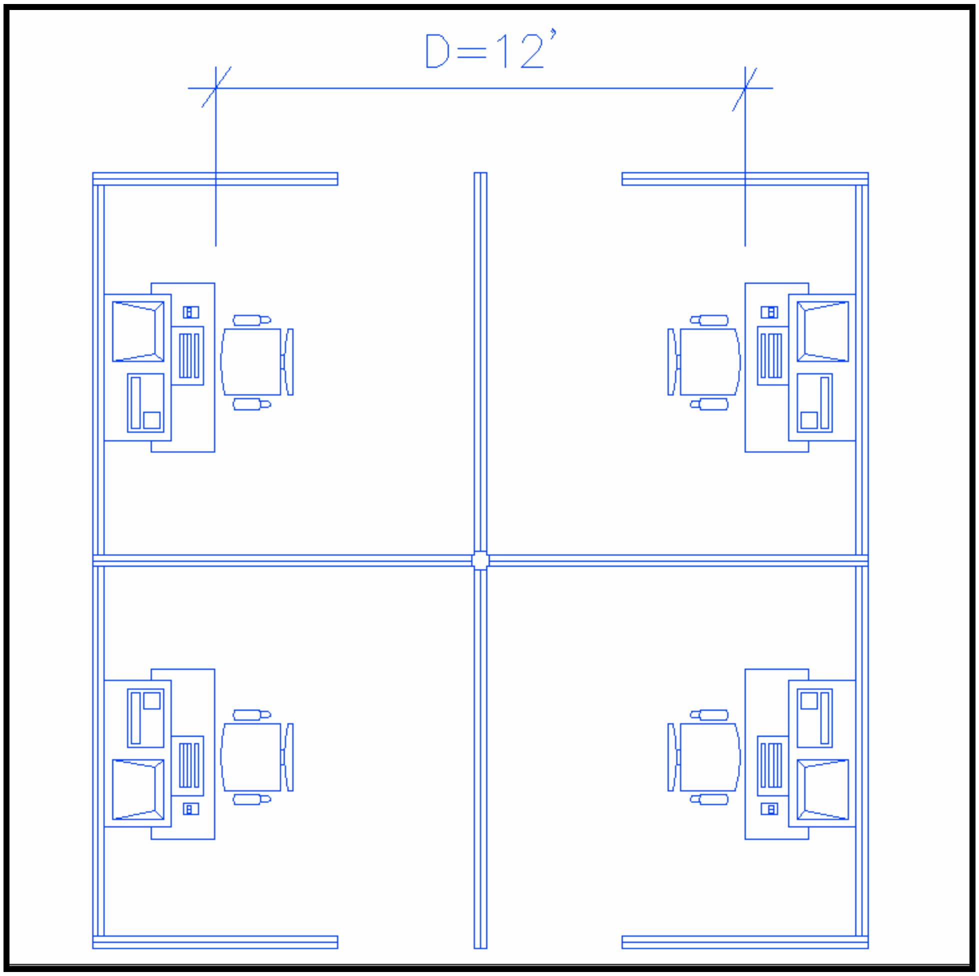
ARCHITECTURAL ACOUSTICS

Speech privacy

The main tenant space of each floor is set up as open plan, where the length and width are much greater than height, and full-height barriers are not used. This plan provides flexibility in layout and ease in arranging and rearranging workstations in offices. In the open plan, one of the most important design problems is to provide the occupants with acoustical privacy from speech transmitted between workstations.

Using the speech privacy analysis method, as outlined in Architectural Acoustics (Egan), an evaluation and design of an open office space was conducted. It is assumed that an engineering firm is to take over the floor, and set up cubicles. The analysis is a step by step procedure that will ultimately give you a speech privacy rating and degree of satisfaction. Satisfactory conditions are anticipated when speech privacy rating number is zero or less.

This evaluation is based on the following workstation set up as shown below. For view of open



Partial height partitions with sound absorbing surfaces on both sides and sound isolating septum are used to divide cubicles. These allow sound energy to be absorbed and block transmission of sound through the barrier.

First Step: Speech Effort (dBA): This describes how people are going to be talking in the room. The scale is from loud to low from 72 to 54 dBA. A-weighted decibels (dBA) refers to the permissible upper limit of noise exposure that will occur in an office environment. For this office, we would want persons within close proximity to each other to speak in a conversational voice level so that they will not disturb other workers. This gives a 60 dBA on the speech effort scale.

Second Step: Privacy Allowance: This describes the amount of privacy that is desired in the open office environment. This can be split into two categories, normal or confidential. Usually, most spaces are designated as normal, with the exceptions being financial institutions, government agencies, or other locations where confidential information is passed around. Confidential privacy is hard to obtain in open plans and is not desired for this office space. A normal privacy is usually designated for open offices and a value of 9 will be used here.

- The overall speech rating is 60+9 = 69.

Third Step: Isolation Rating: This rating is determined from a combination of factors in the office space. The first variable is the distance from source to listener which accounts for the attenuation of voice levels with distance. In this case the D = 12 feet. Secondly, the room finishes for floor and ceiling play a major roll in sound absorption and sound level falloff over distance. For this office, it was assumed that all floors will be carpeted, and all ceilings will utilize sound absorbing acoustical ceiling tiles (ACT).

Thirdly, the height of the partitions dividing the cubicles is taken into account. The value H(ft) is the portion of partition above the acoustical line of sight between source and listener. The assumption is that the partition is half way between source and listener, where it is least effective. Also, these partitions should be at least 5 feet tall and extend all the way to the floor to prevent flanking of speech underneath. It is also important, as noted earlier, to provide partitions with sound absorbing materials to control sound reflections at workstations. A mass of ¾ lb/ft^2 for the solid septum is recommended. For this case we will use a barrier height of 5 feet and an H = 1 ft.

Lastly, a room background noise level (dBA) must be specified. This background noise should be designed to mask and cover up the speech at workstations. This noise should be continuous and uniform over the entire office area, to the point that it is barely noticeable. To pick a correct dBA, noise criteria curves should be consulted. For a large office space, a recommended NC rating is NC-35 to NC-40. This is the about the equivalent of a 47 dBA level. Up to 50 dBA is the prescribed limit, as this is where the background noise can become noticeable, and sometimes annoying to the point where people talk louder to be heard. This in essence defeats the purpose of the masking system. For this office space, a 50 dBA noise level will be implemented.

From all these values, a speech privacy rating number is determined along with a degree of satisfaction. A completed analysis sheet is provided, which yields a speech privacy rating of 3. This value is plotted on the anticipated response to privacy situation curve, and is right on the border of apparent satisfaction and mild dissatisfaction.

In order to ensure an apparent satisfaction response, the partitions could be increased to 6 feet, or the distance between workstations could be increased. A calculation for 6 foot high partitions is also performed on the analysis sheet. This yielded a speech privacy rating of zero, which is

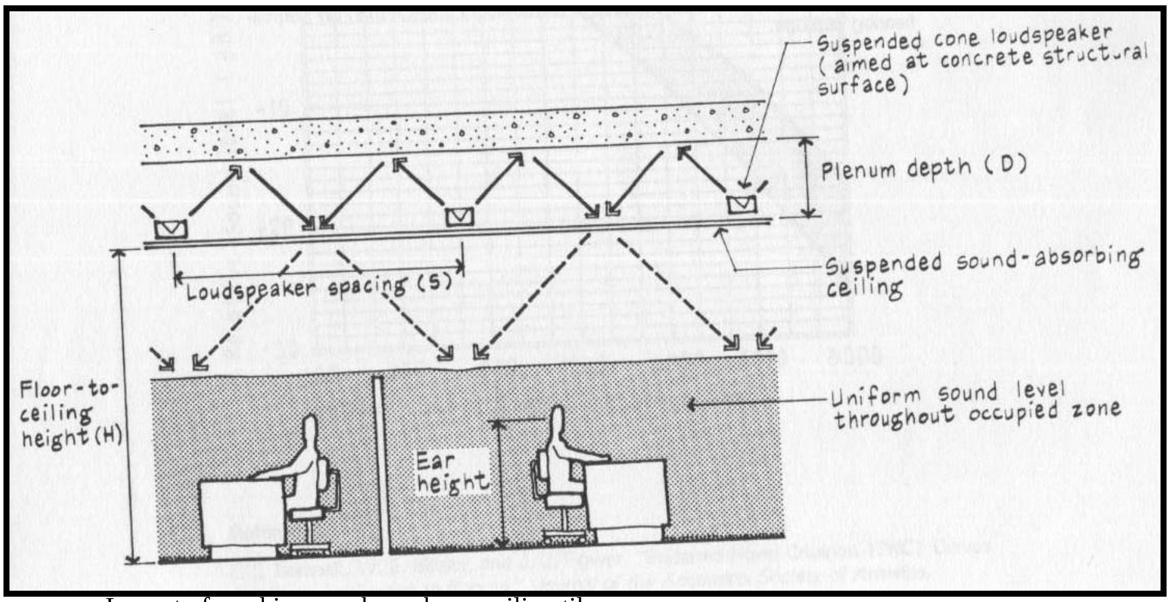
The required sound energy needed as a masking system is rarely produced by an air distribution system by itself. The uniformity, smooth frequency, and proper decibel level needed is hard to achieve for successful speech privacy in an open office plan. However, there is a reliable method of installing electronic masking systems with loudspeakers above the acoustical ceiling tiles. This system should be utilized in all parts of the structure to ensure uniformity throughout the entire floor.

A calculation of preliminary spacing of these types of speakers is performed below.

S = 1.4 (2D + H – 4)

Where S = spacing between loudspeakers (ft)
D = plenum depth (ft) = 13'-6"-9' = 3.5'
H = floor-to-ceiling height (ft) = 9'

S = 1.4 (2 (3.5')+ 9' – 4) = 16.8' ~ **17' spacing**

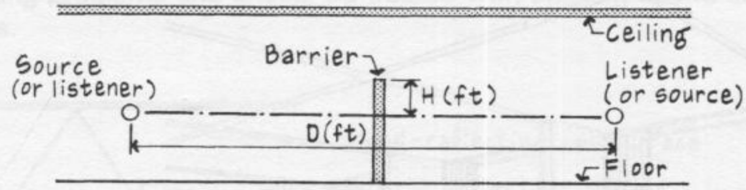


Layout of masking speakers above ceiling tile.

ANALYSIS SHEET (OPEN PLAN)

Analysis Sheet (Open Plan)

Open-plan dimensions



Examples

A B C

Speech rating

1. **Speech effort:** how people talk in room

Loud Raised Conversational Low

72 66 60 54

60 | 60

2. **Privacy allowance:** degree of privacy desired

Confidential Normal

15 9

9 | 9

■ Speech rating total

69 | 69

Isolation rating

3. **Distance from source to listener:** table approximates effect of room sound absorption and sound level falloff with distance D from source to listener

Room finishes		Distance D (ft)				
Ceiling	Floor	3	6	12	24	48
Reflecting	Reflecting	0	3	6	9	12
Reflecting	Absorbing	0	4	8	12	16
Absorbing	Reflecting	0	5	10	15	20
Absorbing	Absorbing	0	6	12	18	24

12 | 12

4. **Partial-height barrier:** table accounts for attenuation from barrier with ceiling absorption at NRC of 0.80. (Barrier width should be at least twice its total height.)

Barrier height H (ft)	Distance D (ft) 3	6	12	24	48
1	11	7	4	2	0
2	14	10	7	4	3
3	15	11	8	5	4
4	16	12	9	6	5

4 | 7

5. **Room background noise level (dBA):** masking sound available

50 | 50

■ Isolation rating total

66 | 69

Speech privacy rating number

3 | 0

Find speech privacy rating number by subtracting isolation total from speech rating total. Then use graph at top of p. 329 to assess degree of satisfaction. Satisfactory conditions are anticipated when speech privacy rating number is 0 or less.

Apparent satisfaction / mild dissatisfaction (5 foot partitions)

A

B

Apparent Satisfaction (6 foot partitions)

C

CONSTRUCTION MANAGEMENT

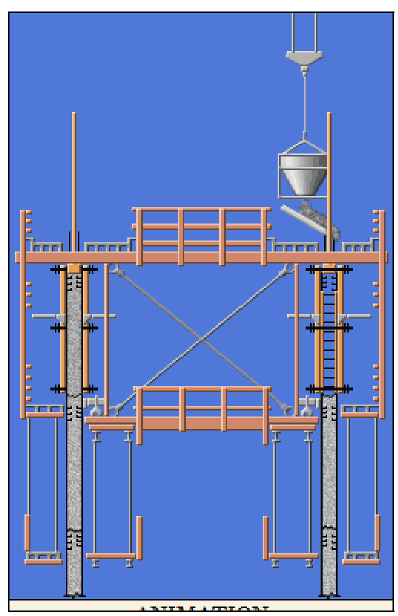
Constructability

The elimination of moment frames from the perimeter walls makes construction much easier. All connections are now shear connections which only require them to be bolted into place, instead of welded. Also, member sizes are now much smaller which allows more members to be shipped in one load. Floor can be put up faster, and man hours can be saved. Changing the core to concrete has some issues that must be addressed. Inherently, using an all steel system or all concrete system will be easier to construct than a mixture. If connections on the shear walls are placed slightly off location issues arise with trying to connect a prefabricated steel girder or beam. There is less room for error, and therefore tolerances are less. Also, there will be congestion in the core area of elevators, stairs, and cranes.

A jump form system will be utilized in the new shear wall core design. In a jump form, a frame is constructed from structural steel members over the central core. Steel formwork panels are hung from this frame, some supported on rollers. After the concrete walls are poured, the formwork is released and rolled back from the concrete face. Jacks then lift or climb the whole frame up one level. All the formwork panels are attached to the frame. This process takes approximately one and a half hours.

Once the climbing formwork is in position, the formwork panels are closed and the next concrete wall is poured. The cycle continues, which is normally four days. Faster times have been achieved. However, the limiting factor to faster times is usually the construction of the floor slabs, which are done as a separate process.

Jump Form In Use
-utilizes crane and bucket for concrete pour



Schedule

There is a considerable decrease in man hours needed, if moment connections are eliminated from the lateral system. This would decrease time for each floor to be built, thereby moving ahead the opening date of the building. Also, the concrete core would utilize a jump form to construct the shear walls. Using this jump form, one floor can be completed each day because it only needs to be in the form for 12 hours. This allows the core to be finished before floors are connected to it. The core could be topped off before the steel is started to erect. However, in all likelihood, the steel would probably be started soon after the jump form is started, and may even catch up to it because there are no more moment connections on the perimeter. Elevator towers could be installed quicker, along with stair wells. There is also a decrease in lead time, as there is not as much steel fabrication and preassembled connection work to be completed. However, as noted in the structural redesign, a peer review would be needed for this project to even take place. Peer reviews take place at the preliminary phase, and periodically through the design phase, and at completion. These can take anywhere from a few months, up to a few years in certain instances. If a project wants to make sure they will be able to start building on schedule, they could apply for a permit earlier in the design phase, or bring the peer reviewer in earlier.

Cost

A cost comparison of the two lateral systems was performed to find any potential savings. The total tonnage of steel from the original dual system, including columns, girders, and braces, was calculated by adding up linear feet of steel and multiplying by its associated plf. This total was 3285 tons for the lateral system. This was then compared to the redesigned gravity only columns and girders, where a tonnage of steel was calculated in the same manner, and found to be only 465 tons. This large discrepancy is due to the fact that the eccentric braced frames in the core are now concrete. Also, the cubic yardage of concrete needed for the shear walls and coupling beams was calculated as 6036 cubic yards needed.

Consultation with the project manager at Pankow Builders, and the fabricator, Herrick Corp, yielded some estimate prices to be used. Applying reductions for inflation over the years, the costs of 1 CY of concrete was around 800-850 dollars including form, labor, material, etc. The price of steel at that time, directly from Pankow, was around 1650 dollars per ton. Material cost $650, fabrication cost $635, and erection cost $365 per ton. If this building were to be built now, the cost savings would be much greater than when it was originally built because steel is around $3000/ton and concrete is around $1000/CY. A summary expenses is provided below

In 2000 Dollars
Original Steel = $5,420,250 Net: +$159,750 new design
New Steel = $480,000
New Concrete = $5,100,000

In 2007 Dollars Net: -$2,520,000 new design
Original Steel = $9,900,000
New Steel = $1,380,000
New Concrete = $6,000,000

Another factor that must be taken into account is the amount of time on site the workers are and the cost of fabricating the moment connections in shops. Data from the fabricator of the project was obtained that compared the average cost of a moment connection to a shear connection. A moment connection was found to cost an additional $700 per connection for fabrication and $800 per connection for field welding. With around 600 moment connections, the cost for fabrication is $420,000 and welding is $480,000.

Moment Connection Cost = $900,000 for fabrication and field welding

Using RS Means (2000) an E9 crew of workers working on a rigid frame building was chosen as the appropriate crew for moment connections/frames. Assuming 2 E9 crews working on site, at $9,200 / day including O&P, this cost is compared to the cost of field welding. A total of 46 days, divided by two crews, leaves 23 days of man hour's savings from not needing moment connections.

A peer review has no set price, or schedule duration. However, they can be estimated as being an hourly paid expense, or hourly not to exceed. A good range for a project is $20,000 to $50000. In some instances, several peer reviews for the same project will be imposed, which could potentially double, triple, etc. the design costs. For this estimate, we will use $50000.

Total Cost comparison (in year 2000 dollars)

Original Steel	-$5420250
New Steel	+$480000
New Concrete	+$5100000
Peer Review	+50000
Mom. Conn.	-$900,000
SAVINGS	**$403,000 + fireproofing**

It appears from these preliminary and rough costs that the redesigned system will save just over $400,000 from the project cost. This number seems reasonable given the fact that all moment frame members and connections from the original design became smaller and shear connections. The required labor involved with shear connections and moment connections is much less. It is difficult, however, to determine the exact amount of time saved on the project without more information. Such as, the crew size on the original site, and the actual timeline of construction. It does appear, based on conversations with various professionals, that there could be a great deal of time saved with the new design. With more time, a more in depth analysis of coordinating the construction of the concrete shear wall core and the rest of the steel building could be analyzed.

SUMMARY AND CONCLUSIONS

The original lateral force resisting system of this structure utilized a dual system of eccentric braced frames and perimeter moment frames. This type of system is a typical construction for buildings over 240 feet, and in a high seismic region like Oakland, California. The proposal for new design with a reinforced concrete shear wall core is an alternative that is starting to be used more often with tall buildings. This design, however, goes against prescribed code limits of height for shear walls in the ASCE7, chapter 12. The result is a performance based design that requires a peer review process to take place. The depth work performed in this thesis work used design criteria that went beyond those required by code for strength and drift, to increase the likelihood of acceptance.

The end result of this work was the formation of a reinforced concrete core with coupling beams connecting them. The coupling beams added rigidity to the structure, along with energy dissipation when they begin to act plastically with the walls. The core was designed after much iteration was conducted in ETABS. The added weight from the core increased the base shear the building saw, but the dynamic analysis performed reduced this by 15% as allowed by code. There are 4 large shear walls acting in the North-South direction and 8 small to large shear walls acting in the East-West direction. The coupling beams connect the 8 walls in the E/W direction and leave openings for doorways to the stairs, bathrooms, and elevators.

There was a tremendous decrease in tonnage of steel needed by switching lateral systems. Also, all moment connections that were part of the perimeter moment frames were removed. This decreased the amount of time needed for welding on site, and thus decreased schedule. Nearly all columns along the perimeter became W14's, a much smaller member than the W24 and W36's that were being used.

The open plan element of each floor was an important consideration when choosing a lateral system. The core only allowed the plan to stay as is, with minimal effect on architecture. Only slight rearranging of the bathrooms was needed. The open office floor plan led to an interesting study into speech privacy and architectural acoustics. From this study it was determined what type of workstation layout works best to keep speech privacy at the recommended level for an office space.

After reviewing cost information, scheduling, constructability, and overall effectiveness of the lateral system, a recommendation was made. This recommendation is that this structural redesign performed, be used instead of the original dual system. Further and more detailed cost estimates could be performed to provide an even clearer answer from an economic perspective.

APPENDICES

APPENDIX A:	**RESOURCES**
APPENDIX B:	**SEISMIC LOADS**
APPENDIX C:	**WIND LOADS**
APPENDIX D:	**COUPLING BEAM DESIGN**
APPENDIX E:	**SHEAR WALL DESIGN**

APPENDIX A:

Resources

ACI 318 – 05: Building Code Requirements for Structural Concrete and Commentary.

ASCE 7-05: Minimum Design Loads for Buildings and Other Structure.

IBC 2006: International Building Code.

Seismic Design Manual, Volume III, November 2000, SEAOC.

Consensus Document, *"An alternative procedure for seismic analysis and design of tall buildings located in the Los Angeles region."* Approved 12/8/05, Los Angeles Tall Buildings Structural Design Council.

Manual of Steel Construction, Load and Resistance Factor Design, Third Edition.

Architectural Acoustics, Egan, David M, 1988.

RS Means Building Construction Data, 2000

APPENDIX B:

Lateral Forces Calculations - Equiv. Lateral Force Procedure

WEIGHT CALCULATIONS (DECK WEIGHT FROM VULCRAFT CATALOG)

ROOF DEAD LOADS: 3" METAL DECK w/ 2½" LWT CONCRETE = 39 PSF
MECH/ELEC/PLUMBING = 10 PSF
STRUCTURAL FRAMING = 15 PSF
COLLATERAL = 5 PSF
69 PSF ≈ 70 PSF

TYPICAL FLOOR: 3" METAL DECK w/ 2½" NWT CONCRETE = 50 PSF
MEP = 10 PSF
FRAMING = 15 PSF
COLLATERAL = 5 PSF
80 PSF

2ND FLOOR: 3" METAL DECK w/ 4" NWT CONCRETE = 69 PSF
MEP = 10 PSF
FRAMING = 15 PSF
COLLATERAL = 5 PSF
99 PSF ≈ 100 PSF

22ND FLOOR: 3" METAL DECK w/ 6" NWT CONCRETE = 100 PSF
MEP = 10 PSF
FRAMING = 15 PSF
COLLATERAL = 5 PSF
130 PSF

22ND (MECHANICAL): 3" METAL DECK w/ 6" NWT CONC. = 100 PSF
MEP = 10 PSF
FRAMING = 15 PSF
COLLATERAL = 5 PSF
130 PSF

6" CONCRETE PADS → 75 PSF × AREA PADS ≈ 4500 ft^2

NOTE: ALLOW 10 PSF FOR PARTITIONS IN SEISMIC WEIGHT.

Bergmann associates

Subject: SEISMIC LOAD CALCULATIONS

USING ASCE 7-05 EQUIVALENT LATERAL FORCE PROCEDURE

USGS WEBSITE: OAKLAND, CA: LATITUDE = 37.804603
LONGITUDE = −122.275486

S_S = 0.2s = SPECTRAL RESPONSE = 240.95
S_1 = 1.0s = SPECTRAL RESPONSE = 94.05

BUILDING SITE CLASS: C
BUILDING OCCUPANCY: OFFICE ⇒ TYPE II ⇒ I = 1.0 (TABLE 11.5-1)

SITE COEFFICIENTS: $F_a = 1.0$ (TABLE 11.4-1 FOR $S_S \geq 1.25$)
$F_v = 1.3$ (TABLE 11.4-2 FOR $S_1 \geq 0.5$)

$S_{MS} = F_a S_S = 1.0(2.4095) = 2.4095 \Rightarrow S_{DS} = \frac{2}{3}(2.4095) = 1.6063$
$S_{M1} = F_v S_1 = 1.3(0.9405) = 1.2227 \Rightarrow S_{D1} = \frac{2}{3}(1.2227) = 0.815$

★ SEISMIC DESIGN CATEGORY: $S_{DS} = 1.6063 > 0.5 \Rightarrow D$
$S_{D1} = 0.815 > 0.2 \Rightarrow D$

★ FROM 11.6 OF ASCE: FOR OCCUPENCY CATEGORY I, II, III, WHERE $S_1 \geq 0.75$, BUILDING TO BE DESIGNED AS CATEGORY E.

TABLE 12.2-1: DESIGN COEFF. AND FACTORS FOR SFRS.

A. BEARING WALL: ① SPECIAL REINFORCED CONCRETE SHEAR WALLS
- DETAILING REQ: 14.2, 14.2.3.6
$R = 5$, $\Omega_o = 2.5$, $C_d = 5$

FUNDAMENTAL PERIOD: $C_u = 1.4$ FOR $S_{D1} \geq 0.4$ (TABLE 12.8-1)

$$T_a = C_t \cdot h_n^x = .02(301)^{0.75} = 1.445 \text{ sec}$$

$T_a C_u = 1.4(1.445) = 2.023$

SEISMIC RESPONSE COEFFICIENTS: $T_L = 12$ sec

$C_S = \frac{S_{DS}}{(R/I)} = \frac{1.6063}{(5/1.0)} = 0.32$

$\leq C_S = \frac{S_{D1}}{T(R/I)} = \frac{0.815}{2.023(5/1)} = 0.081$

$\geq C_S = \frac{0.5 S_1}{(R/I)} = \frac{0.5(.9405)}{5} = \boxed{0.094}$ ← governs

Distribution of Static Lateral Forces

Level	area(ft^2)	story wt(psf)	SW wt(psf)	façade wt(psf)	$w_{x\ (kips)}$	h_x	$w_xh_x^k$	C_{vx}	F_x (kips)	M_x (ft-kips)	Shear(kips)
roof	13500	70	0	3.8	996.3	301.4	22997586	0.0412	265	79923	265
mez	3000	130	43	3.8	530.4	291.75	11561725	0.0207	133	38894	398
mech	23937	130	43	3.8	4232.1	271.75	81412369	0.1459	939	255098	1337
19	23937	90	43	3.8	3274.6	258	57491879	0.1030	663	171030	2000
18	23937	90	43	3.8	3274.6	245	52491396	0.0941	605	148286	2605
17	23937	90	43	3.8	3274.6	232	47688611	0.0855	550	127570	3155
16	23937	90	43	3.8	3274.6	219	43086127	0.0772	497	108800	3652
15	23937	90	43	3.8	3274.6	206	38686739	0.0693	446	91892	4098
14	23937	90	43	3.8	3274.6	193	34493454	0.0618	398	76761	4496
13	23937	90	43	3.8	3274.6	180	30509524	0.0547	352	63322	4847
12	23937	90	43	3.8	3274.6	167	26738486	0.0479	308	51487	5156
11	23937	90	43	3.8	3274.6	154	23184209	0.0415	267	41168	5423
10	23937	90	43	3.8	3274.6	141	19850957	0.0356	229	32274	5652
9	23937	90	43	3.8	3274.6	128	16743466	0.0300	193	24712	5845
8	23937	90	43	3.8	3274.6	115	13867048	0.0249	160	18388	6005
7	23937	90	43	3.8	3274.6	102	11227734	0.0201	129	13205	6134
6	23937	90	43	3.8	3274.6	89	8832471	0.0158	102	9064	6236
5	23937	90	43	3.8	3274.6	76	6689409	0.0120	77	5862	6313
4	23937	90	43	3.8	3274.6	63	4808340	0.0086	55	3493	6369
3	23937	90	43	3.8	3274.6	50	3201423	0.0057	37	1846	6406
2	23937	90	43	3.8	3274.6	37	1884477	0.0034	22	804	6427
1	23937	110	43	3.8	3753.3	24	1008295	0.0018	12	279	6439
					68454.6		5.58E+08	1.0008	6439	1364157	
				V=Cs*W	6434.7						

APPENDIX C:

Lateral Forces Calculations – ASCE 7 MWFRS

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General Building Information		
Building:	555 12th Street	Reference
Location:		Oakland, CA
Basic Wind Speed(mph): V	85	Fig. 6-1
Exposure Category:	C	6.5.6.3
Enclosure Class	Enclosed	Sect. 6.2
Building Category	II	Table 1-1
Importance Factor: Iw	1.0	Table 6-1
Topography Factor: Kzt	1	Sect. 6.5.7
Directionality Factor: Kd	0.85	Table 6-4
Internal Pressure Coefficient: Gcpi	0.18	Fig. 6-5
Gust Factor- assume rigid G	0.85	6.5.8.1
Building Height: h (feet)		306
Length Parallel to wind: L (feet)		227'
Length Perpendicular to wind: B (feet)		125'

L/B		L/B
1.9		0.5
External Pressure (Cp)	E-W	N-S
Windward	0.8	0.8
Leeward	-0.3	-0.5

Results	North-South Wind Loading					East-West Wind Loading		
Height	Kz	qz	P leeward	P windward	P total	P leeward	P windward	P total
0-15	0.85	13.4	-15.1	13.6	**28.7**	-10.9	13.6	**24.5**
20	0.9	14.1	-15.1	14.1	**29.2**	-10.9	14.1	**25.0**
25	0.94	14.8	-15.1	14.5	**29.6**	-10.9	14.5	**25.4**
30	0.98	15.4	-15.1	15.0	**30.1**	-10.9	15.0	**25.9**
40	1.04	16.4	-15.1	15.6	**30.7**	-10.9	15.6	**26.5**
50	1.09	17.1	-15.1	16.2	**31.3**	-10.9	16.2	**27.1**
60	1.13	17.8	-15.1	16.6	**31.7**	-10.9	16.6	**27.5**
70	1.17	18.4	-15.1	17.0	**32.1**	-10.9	17.0	**27.9**
80	1.21	19.0	-15.1	17.4	**32.5**	-10.9	17.4	**28.3**
90	1.24	19.5	-15.1	17.8	**32.9**	-10.9	17.8	**28.7**
100	1.26	19.8	-15.1	18.0	**33.1**	-10.9	18.0	**28.9**
120	1.31	20.6	-15.1	18.5	**33.6**	-10.9	18.5	**29.4**
140	1.36	21.4	-15.1	19.0	**34.1**	-10.9	19.0	**29.9**
160	1.39	21.9	-15.1	19.4	**34.5**	-10.9	19.4	**30.3**
180	1.43	22.5	-15.1	19.8	**34.9**	-10.9	19.8	**30.7**
200	1.46	23.0	-15.1	20.1	**35.2**	-10.9	20.1	**31.0**
250	1.53	24.1	-15.1	20.9	**36.0**	-10.9	20.9	**31.8**
300	1.59	25.0	-15.1	21.5	**36.6**	-10.9	21.5	**32.4**
306	1.59	25.0	-15.1	21.5	**36.6**	-10.9	21.5	**32.4**

Shear Forces acting at each story in both directions

EAST_WEST				NORTH_SOUTH		
Floor	**Story Height**	**Shear(Kip)**	**Over. Mom (ft-kip)**	**Story Height**	**Shear**	**Over. Mom**
2	24	58.5	1404	24	123.9	2973.6
3	37	43.3	1602.1	37	91	3367
4	50	44.4	2220	50	92.9	4645
5	63	45.2	2847.6	63	94.4	5947.2
6	76	46.1	3503.6	76	96.1	7303.6
7	89	46.8	4165.2	89	97.3	8659.7
8	102	47.5	4845	102	98.7	10067.4
9	115	47.9	5508.5	115	99.3	11419.5
10	128	48.6	6220.8	128	100.7	12889.6
11	141	49	6909	141	101.3	14283.3
12	154	49.3	7592.2	154	101.8	15677.2
14	167	49.9	8333.3	167	103	17201
15	180	50.1	9018	180	103.4	18612
16	193	50.4	9727.2	193	103.9	20052.7
17	206	51.6	10629.6	206	106.2	21877.2
18	219	51.7	11322.3	219	106.2	23257.8
19	232	51.7	11994.4	232	106.2	24638.4
20	245	51.8	12691	245	106.4	26068
21	258	54.7	14112.6	258	112.2	28947.6
22	272	68.9	18740.8	272	141.2	38406.4
mezzanine	292	68.9	20118.8	292	141.2	41230.4
roof	306	28.4	8690.4	306	58.2	17809.2
		1104.7	**182196.4**		**2285.5**	**375333.8**

APPENDIX D:

COUPLING BEAM DESIGN

Story Shears for coupling beams

Beam 7,9,10,12	
Story	**V (kips)**
Roof	54.9
Mezz	82.4
20	106.7
19	122.7
18	142.3
17	162.1
16	180.4
15	196.9
14	211.7
13	224.9
12	237.1
11	248.6
10	259.9
9	271.1
8	281.9
7	291.9
6	300.1
5	**305.1**
4	304.9
3	296.9
2	277.5
1	258.7

Beam 8,11	
Story	**V(kips)**
Roof	94.2
Mezz	136.7
20	166.5
19	185.1
18	209.6
17	234.6
16	257.9
15	279
14	298
13	314.9
12	330.3
11	344.6
10	358.1
9	370.9
8	382.9
7	393.2
6	400.6
5	403.6
4	399.7
3	385.6
2	357.1
1	327.7

Design Shears for Coupling Beams

Beam 7,9,10,12			Beam 8,11		
Floors	**0.8*Vmax (kips)**	**Vavg**	**Floors**	**0.8*Vmax (kips)**	**Vavg**
16-Roof	**144**	121	16-Roof	**206**	183
8 to 15	225	**241**	8 to 15	306	**334**
1 to 7	244	**290**	1 to 7	322	**381**

Example Design Calculation for Coupling Beams

BEAMS 7,9,10,12 : FLOORS 1-7

$V_u = 290^k \Rightarrow M_u = \frac{290(6)}{2} = 870'k$

$A_s = \frac{M_u}{cd}$ where $c \approx 4.3$ FOR $9 ksi = f'_c$

$A_s = \frac{870}{4.3(33)} = 6.13 in^2 \Rightarrow$ TRY 4#11 w/ $A_s = 6.24 in^2$

$M_{pr} = \frac{(6.24)(4.3)(33)}{0.9}(1.25) = 1229'k$

$V_{pr} = \frac{2(1229)}{6} = 409^k$

$\phi V_s = 0.75(60) A_v/12 (33) > 409 \Rightarrow A_v/ft = 3.30 in^2/ft$

TRY 4#5 @ 4.5" $\Rightarrow$ .82 in^2/ft each

$\phi V_s = 0.75(60)(4)(.31)(60)(33/4.5) = 409 \geq 409$ ∴ OK

BEAMS 7,9,10,12 : FLOORS 8-15

$V_u = 241^k \Rightarrow M_u = \frac{241}{2}(6) = 723'k$

Assume $f_s > f_y$: TRY 4#10, $A_s = 5.08 in^2$

$A_s = \frac{723}{4.3(33)} = 5.09 in^2 \Rightarrow a = \frac{A_s f_y (1.25)}{0.85 f'_c b} = \frac{5.08 in^2 (60)(1.25)}{0.85(9)(24)} = 2.08"$

$M_{pr} = A_s f_y (d - a/2) = 5.08(1.25)(60)(33 - 2.08/2) = 1015'k$ → COMPARE TO $\frac{A_s c d (1.25)}{\phi} = 1001'k$ ← USE

$\phi V_n \geq V_u = \frac{2M_{pr}}{6} = \frac{2(1001)}{6} = 333 \Rightarrow V_{pr} + V_g$ ⊳ 0 → do not count gravity

$\phi V_s = 0.75(60) A_v/12 (33) = 333 \Rightarrow A_v/ft = 2.69 in^2/ft \Rightarrow$ 4#5 @ 5.5^k

$\phi V_s = 0.75(4)(.31)(60)(33/5.5) = 335^k > 333^k$ ∴ OK

APPENDIX E:

SHEAR WALL DESIGN

NOMINAL SHEAR CAPACITY OF SHEAR WALLS

1835

$V_n = A_{cv}(\alpha_c\sqrt{f'_c} + \rho_t f_y) > V_u$

Y-DIRECTION: CHECK FOR 1961^k AND 2167^k FOR $33' = L_w$, $2' =$ THK

$V_n = (33')(2')(144)\left(2.0(\sqrt{9000}) + 0.0025(60000)\right) = 3229000\ lb\ (0.6) = 1937^k$ OK

NEED MORE REINFORCING

$3611600 = 9504\,in^2\left(2.0\sqrt{9000} + \rho_t(60000)\right)$

$380 = 2\sqrt{9000} + \rho_t(60000) \Rightarrow \rho = 0.003$

$\rho_t = \frac{A_s}{bd} \Rightarrow .003 = \frac{A_s}{12(24)} \Rightarrow A_s = 0.864\ in^2/ft$

USE 2 CURTAINS OF #6 BARS @ 12" w/ $A_s = 0.88\ in^2/ft$

$\boxed{\text{USE \#6 @ 12" EW EF FOR ALL WALLS}}$

CHECK X-DIRECTION CHECK 1155^k FOR $26.5' = L_w$

CHECK 400^k FOR $12' = L_w$

$V_n = (26.5)(2)(144)\left(2.0\sqrt{9000} + .0025(60000)\right) = 0.6(2592^k) = 1555^k > 1155^k$ ∴ OK

USE $\rho = 0.0025 = \frac{A_s}{bd} \Rightarrow A_s = 0.72\,in^2/ft \Rightarrow$ $\boxed{\text{\#6 @ 12" EW EF}}$

$V_n = (12)(2)(144)\left(2.0\sqrt{9000} + .0025(60000)\right) = 0.6(1174^k) = 704^k > 400^k$ ∴ OK

USE $\rho = .0025 = \frac{A_s}{bd} \Rightarrow A_s = 0.72\,in^2/ft \Rightarrow$ $\boxed{\text{\#6 @ 12" EW EF}}$

★ DESIGN ALL SHEAR WALLS W/ #6 @ 12" EW EF

PCA Column Output for Pier One – Channel shape (Flexural Steel)

```
General Information:
   ====================
      File Name: T:\Mueller\PCA\Pier One_WORKS.col
      Project:  Mueller_SW1
      Column:   Pier 1                        Engineer: ERM
      Code:     ACI 318-02                    Units: English

      Run Option: Investigation               Slenderness: Not considered
      Run Axis:   Biaxial                     Column Type: Structural

   Material Properties:
   ====================
      f'c   = 9 ksi                           fy   = 60 ksi
      Ec    = 5407.5 ksi                      Es   = 29000 ksi
      Ultimate strain = 0.003 in/in
      Beta1 = 0.65

   Section:
   ========
      Exterior Points
        No.     X (in)     Y (in)    No.     X (in)     Y (in)    No.     X
(in)     Y (in)
      ----- ---------- ----------  ----- ---------- ----------  ----- ---------
- ----------
          1        0.0        0.0      2      144.0        0.0      3
144.0       24.0
          4       24.0       24.0      5       24.0      372.0      6
144.0      372.0
          7      144.0      396.0      8        0.0      396.0

      Gross section area, Ag =  15264 in^2
      Ix =  3.23748e+008 in^4                 Iy =  2.59602e+007 in^4
      Xo =  39.1698 in                        Yo =  198 in

   Reinforcement:
   ==============
      Rebar Database: ASTM A615
      Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)   Size Diam (in)
Area (in^2)
      ---- --------- -----------   ---- --------- -----------   ---- ---------
-----------
      #  3      0.38        0.11   #  4      0.50        0.20   #  5      0.63
0.31
      #  6      0.75        0.44   #  7      0.88        0.60   #  8      1.00
0.79
      #  9      1.13        1.00   # 10      1.27        1.27   # 11      1.41
1.56
      # 14      1.69        2.25   # 18      2.26        4.00

      Confinement: Tied; #3 ties with #10 bars,  #4 with larger bars.
      phi(a) = 0.8,  phi(b) = 0.9,  phi(c) = 0.65

      Pattern: Irregular
```

Total steel area, As = 117.00 in^2 at 0.77%

Area in^2	X (in)	Y (in)	Area in^2	X (in)	Y (in)	Area in^2	X (in)	Y (in)
6.00	12.0	12.0	6.00	33.0	12.0	6.00	54.0	12.0
6.00	75.0	12.0	6.00	96.0	12.0	6.00	117.0	12.0
6.00	138.0	12.0	6.00	12.0	384.0	6.00	33.0	384.0
6.00	54.0	384.0	6.00	75.0	384.0	6.00	96.0	384.0
6.00	117.0	384.0	6.00	138.0	384.0	3.00	12.0	43.0
3.00	12.0	74.0	3.00	12.0	105.0	3.00	12.0	136.0
3.00	12.0	167.0	3.00	12.0	198.0	3.00	12.0	229.0
3.00	12.0	260.0	3.00	12.0	291.0	3.00	12.0	322.0
3.00	12.0	353.0						

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04/11/07
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Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)

No.	Pu kip	Mux k-ft	Muy k-ft	fMnx k-ft	fMny k-ft	fMn/Mu
1	5885.0	55418.0	19934.0	171775.0	61787.9	3.100
2	5885.0	-55418.0	19934.0	-171775.0	61787.9	3.100
3	5885.0	55418.0	-19934.0	116294.9	-41831.5	2.099
4	5885.0	-55418.0	-19934.0	-116294.8	-41831.5	2.099
5	5885.0	184726.0	5980.0	193263.8	6256.4	1.046
6	5885.0	-184726.0	5980.0	-193264.1	6256.4	1.046
7	5885.0	184726.0	-5980.0	191496.7	-6199.2	1.037
8	5885.0	-184726.0	-5980.0	-191496.9	-6199.2	1.037

*** Program completed as requested! ***

PCA Column Output for Pier Two – I beam shape (Flexural Steel)

```
General Information:
   ====================
      File Name: T:\Mueller\PCA\Pier Two less reinforce.col
      Project:  Mueller_SW2
      Column:   Pier 2                        Engineer: ERM
      Code:     ACI 318-02                    Units: English

      Run Option: Investigation               Slenderness: Not considered
      Run Axis:   Biaxial                     Column Type: Structural

   Material Properties:
   ====================
      f'c   = 9 ksi                           fy   = 60 ksi
      Ec    = 5407.5 ksi                      Es   = 29000 ksi
      Ultimate strain = 0.003 in/in
      Beta1 = 0.65

   Section:
   ========
      Exterior Points
        No.     X (in)     Y (in)    No.     X (in)     Y (in)    No.     X
(in)     Y (in)
      ----- ---------- ----------  ----- ---------- ----------  ----- ---------
- ----------
          1        0.0        0.0      2      318.0        0.0      3
318.0       24.0
          4      156.0       24.0      5      156.0      372.0      6
318.0      372.0
          7      318.0      396.0      8        0.0      396.0      9
0.0      372.0
         10      132.0      372.0     11      132.0       24.0     12
0.0       24.0

      Gross section area, Ag =  23616 in^2
      Ix =  6.13094e+008 in^4                 Iy =  1.30245e+008 in^4
      Xo =  153.695 in                        Yo =  198 in

   Reinforcement:
   ==============
      Rebar Database: ASTM A615
      Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)   Size Diam (in)
Area (in^2)
      ---- --------- -----------   ---- --------- -----------   ---- ---------
-----------
      #  3      0.38        0.11   #  4      0.50        0.20   #  5      0.63
0.31
      #  6      0.75        0.44   #  7      0.88        0.60   #  8      1.00
0.79
      #  9      1.13        1.00   # 10      1.27        1.27   # 11      1.41
1.56
      # 14      1.69        2.25   # 18      2.26        4.00

      Confinement: Tied; #3 ties with #10 bars,  #4 with larger bars.
      phi(a) = 0.8,  phi(b) = 0.9,  phi(c) = 0.65

      Pattern: Irregular
```

Total steel area, As = 158.00 in^2 at 0.67%

Area in^2	X (in)	Y (in)	Area in^2	X (in)	Y (in)	Area in^2	X (in)	Y (in)
2.00	144.0	43.0	2.00	144.0	74.0	2.00	144.0	105.0
2.00	144.0	136.0	2.00	144.0	167.0	2.00	144.0	198.0
2.00	144.0	229.0	2.00	144.0	260.0	2.00	144.0	291.0
2.00	144.0	322.0	2.00	144.0	353.0	4.00	7.0	12.0
4.00	26.0	12.0	4.00	45.0	12.0	4.00	64.0	12.0
4.00	83.0	12.0	4.00	102.0	12.0	4.00	121.0	12.0
4.00	140.0	12.0	4.00	159.0	12.0	4.00	178.0	12.0
4.00	197.0	12.0	4.00	216.0	12.0	4.00	235.0	12.0
4.00	254.0	12.0	4.00	273.0	12.0	4.00	292.0	12.0
4.00	311.0	12.0	4.00	7.0	384.0	4.00	26.0	384.0
4.00	45.0	384.0	4.00	64.0	384.0	4.00	83.0	384.0
4.00	102.0	384.0	4.00	121.0	384.0	4.00	140.0	384.0
4.00	159.0	384.0	4.00	178.0	384.0	4.00	197.0	384.0
4.00	216.0	384.0	4.00	235.0	384.0	4.00	254.0	384.0

Area in^2	X (in)	Y (in)	Area in^2	X (in)	Y (in)	Area in^2	X (in)	Y (in)
4.00	273.0	384.0	4.00	292.0	384.0	4.00	311.0	384.0

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)

No.	Pu kip	Mux k-ft	Muy k-ft	fMnx k-ft	fMny k-ft	fMn/Mu
1	8320.0	72213.0	21485.0	267963.4	79725.2	3.711
2	8320.0	-72213.0	21485.0	-267962.8	79725.0	3.711
3	8320.0	72213.0	-21485.0	267468.6	-79578.0	3.704
4	8320.0	-72213.0	-21485.0	-267468.2	-79577.9	3.704
5	8320.0	240711.0	6446.0	271485.0	7270.1	1.128
6	8320.0	-240711.0	6446.0	-271484.0	7270.1	1.128
7	8320.0	240711.0	-6446.0	270983.8	-7256.7	1.126
8	8320.0	-240711.0	-6446.0	-270982.8	-7256.7	1.126

*** Program completed as requested! ***