Final Report

April 12th, 2007

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AE 482 - Senior Thesis
Pennsylvania State University

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Tower 333

Bellevue WA

General Statistics:

Height: 260ft
Number of Stories: 20 above grade, 8 below grade
Floor Plate: 23,000sq. ft.
Total Square Footage: 410,000
Occupancy: Office

Structural:

- Uses an existing concrete core system from a previous abandoned project.
- Combines concrete core and steel moment frames into a dual lateral system.
- Deepest excavation in Bellevue history: 93 feet below grade
- Deepest "soil nailing" in U.S. construction history

MEP:

- 16,500 cfm AHU on every floor
- Series fan powered VAV boxes and linear diffusers along windows
- 4-450 TON chillers on parking levels 1&2
- 2-4,000A 3P 4W main feeders
- 800kW 277/480V diesel emg. generator equipped with 480 gal tank

LEED Rating Info:

- Highly transparent glass maximizes view of Lake Washington and the Olympics
- 10ft floor to finished ceiling heights and full height glass windows maximizes daylight penetration
- State of the art operating systems minimizes energy consumption

Architectural Features:

- Open, column free floorplates allows for flexible space planning
- Glass curtain wall design
- Half acre outdoor plaza on mezzanine level

Owner: Hines Development,
Structural Engineers: Magnussun Klemencic, **Architect:** LMN Architects

Paul Parfitt - Structural

http://www.arche.psu.edu/thesis/eportfolio/current/portfolios/pjp164

Acknowledgments:

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Executive Summary:

The purpose of this report is to determine through analytical methods the performance of the lateral force resisting system implemented as a part of the AE Senior Thesis for the design of the 260 foot tall office tower, Tower 333 in Bellevue, Washington under seismic and wind loads. Costs and related issues were investigated as a part of the overall study.

Existing Lateral System:

Originally implemented as a dual-resisting lateral system, a combination of special exterior moment frames and a special 24" thick concrete shear wall centralized core was used. The concrete core is 40 feet by 32 feet with 7 foot openings for elevator access in the 32 foot length side. Moment frames consisting of rolled W shapes with columns ranging from W14x730 at the mezzanine level to W14x132 at the penthouse level are used. The moment frame beams range in size from W36x256 at floor 1 to W18x86 at the penthouse level..

Gravity System:

A 2-1/2" concrete slab on a 3" deep metal composite deck with an f'c of 4,000psi and WWF 6x6 W3.5xW3.5 reinforcing is used as the existing floor system. Supporting the slab are W18x40 composite steel beams which span 42' N-S in a typical bay. Beams frame into composite steel girders on the interior which are typically W18x97 spanning E-W.

Conclusion:

In order to determine Tower 333's lateral resisting system response to seismic and wind loads a model of Tower 333's lateral system was created in ETABS. Lateral elements consisting of the core and perimeter moment frames were modeled and connected with a rigid diaphragm on each floor. The model was loaded with seismic and wind forces calculated using spreadsheets in accordance with ASCE-7 '05 and analyzed under the different load combinations required by ASCE-7. Using this ETABS model, in conjunction with hand calculated spot checks I was able to confirm that the assumptions made prior to the design of Tower 333 were appropriate. These assumptions include drift limitations of L/400 for wind and ASCE-7 '05 section 12.8.6 allowable drift for seismic. Based on the relative base shear distributions, it was determined that the moment frames resist 10% of the lateral load. However, these frames were initially designed for 25% of the seismic force in conjunction with the dual system requirements of ASCE-7 for the Seattle area. An examination of the drift results reveals that the dual system as originally designed is well balanced and subject to only minimal building torsions.

This thesis study included an in depth examination of the lateral system as a core-only design. Eliminating the exterior moment frames from the lateral system saves not only money in the design and fabrication, but also in erection time, which will ultimately decrease the total cost of the building as well as provide an earlier move in date for tenants. An initial lateral analysis using the Equivalent Lateral Force Method, (ELF,) resulted in seismic forces controlling over wind. (For a table of the wind and seismic forces and drifts see Appendix D1.) An initial core size was first determined and from this preliminary design, a set of iterative analyses were conducted in ETABS to develop the most economical and efficient design of the core walls and coupling beams. The main controlling factor in the design of the core was the drift limitations. Once these limitations were under control the rest of the building could be modeled and tested in ETABS to obtain design forces. From these design forces then, a design of the concrete core and coupling beams using a combination of hand calculations following ACI318-05 and computer programs such as PCA Column was determined. Despite an increase in concrete volume of the core, the elimination of the moment frames results in a more economical design and added revenue to the owner due to the early move in date. It is the recommendation of the author of this thesis study that the alternative core-only design of the lateral system is a beneficial design appropriate for implementation.

Introduction:

Tower 333 is an 18 story office building located in Bellevue Washington. The total height of Tower 333 is 260 feet tall with an additional 8 levels of below ground parking that extends 93 feet below grade. Floor 1 will contain retail and professional services, while floors 2-18 are designated for office use. The building is scheduled to be completed in December of 2007. However, due to the tower-crane collapse on the construction site on November 16th this date may be postponed further, (see additional links on Author's CPEP website for more details.) The code used to design Tower 333 was the IBC 2003 with reference to ASCE-7 02' for load values. For this analysis, ASCE -7 05' was used as an update. When using ETABS for this evaluation user defined loading for seismic and wind forces were calculated by spread sheet using ASCE-7 '05 and assigned to the model as a static representation of the dynamic loads.

Hine's Development, the owner of Tower 333, chose to place the building on an existing foundation of another building which was abandoned due to financial reasons early in its erection phase. The previous building, which was to be called the Bellevue Tech Tower, had only sub levels 8-5 completed when it was abandoned. Bellevue Tech Tower was designed to be a cast in place concrete building of similar height to Tower 333 and utilize a concrete core and shear walls as its lateral force resisting system. Hine's decision to utilize this abandoned building saved considerable time and money in the excavation and foundation process.

The architecture of Tower 333 is meant to take advantage of its location with full 10 foot high, highly transparent windows which perfectly frame Lake Washington and the Olympic Mountains lingering in the background. Another advantage to these windows is that they allow maximum light penetration into a column free open plan floor layout. These features, coupled with state of the art operation systems, and drought resistant vegetation located in the ½ acre plaza qualifies the building for LEED certification, which allows Tower 333 to do its part in the push for green buildings in the Seattle area.

Existing Structural System:

Existing Gravity System:

A typical bay of the upper office floors of Tower 333 are supported by 42' long W18x40 composite beams with a camber of 1-1/2" and 30' long W18x97 composite girders with a camber of ¾". Both have a strength of 50ksi. These members in turn support a 2-1/2" concrete slab on a 3" deep composite metal deck with the strength of the concrete being 4,000psi. To control shrinkage and expansion and contraction of the concrete, there is WWF 6x6 W3.5xW3.5 reinforcing in the slab. The floor to floor height is 13'-10" and the overall weight of this system is 58 psf with a framing depth of 24". The finished floor to finished ceiling height is 10' which allows 2-10" of plenum clearance space. This plenum space is utilized for the mechanical equipment which incorporates a variety of 12" and 14" deep ducts to transport air to strip diffusers along the perimeter of the building. (Refer to Figure 1. for a framing plan of the existing system.)

Existing Lateral Framing System:

Tower 333 utilizes a dual-resisting lateral system with a special concrete core and perimeter special moment frames. The concrete core consists of 2 foot thick walls, 40 feet in length along the North-South direction and 32 feet in length with 7 foot openings for elevator access in the East-West direction. See Figure 2 for layout of the core and frames.

Having a bearing capacity of f'c = 9000 psi, the concrete shear walls contain two curtains of #7 rebar at 12 inches on center and #5 hoops and ties at 6 inches on center. The core extends the full height of the building from sub parking level 8 to the roof level, a total of 338 feet. There are a total of four moment frames around the perimeter of Tower 333. One moment frame is on each North and South face, consisting of 3-30 foot bays with columns ranging from W14x730 at the mezzanine level to W14x132 at the penthouse level. The beams on the North and South frames range from W24x176 at floor 1 to W18x86 at the penthouse floor. The other two moment frames are on the East and West face, with one 26 foot bay and one 42 foot bay containing a range of columns from W14x550 at the mezzanine level to W14x132 at the penthouse level and beams ranging in size from W36x256 at floor 1 to W18x86 at the penthouse floor.

Foundation:

Being located on an previously abandoned construction site, Tower 333 takes advantage of an existing foundation that was modified for the building's structural design. Plans indicate that sub levels 8-5 were completed before the project was abandoned. The existing foundation consists of spread concrete footings. Where designated, these footings were either demolished, partially demolished and replaced or thickened to provide higher capacity. Where the footings are reinforced, rebar was drilled and grouted into the bottom of the footings. The foundation supporting the concrete core shear walls is a mat slab foundation with a new additional 24" of topping applied to the existing mat for added structural stability.

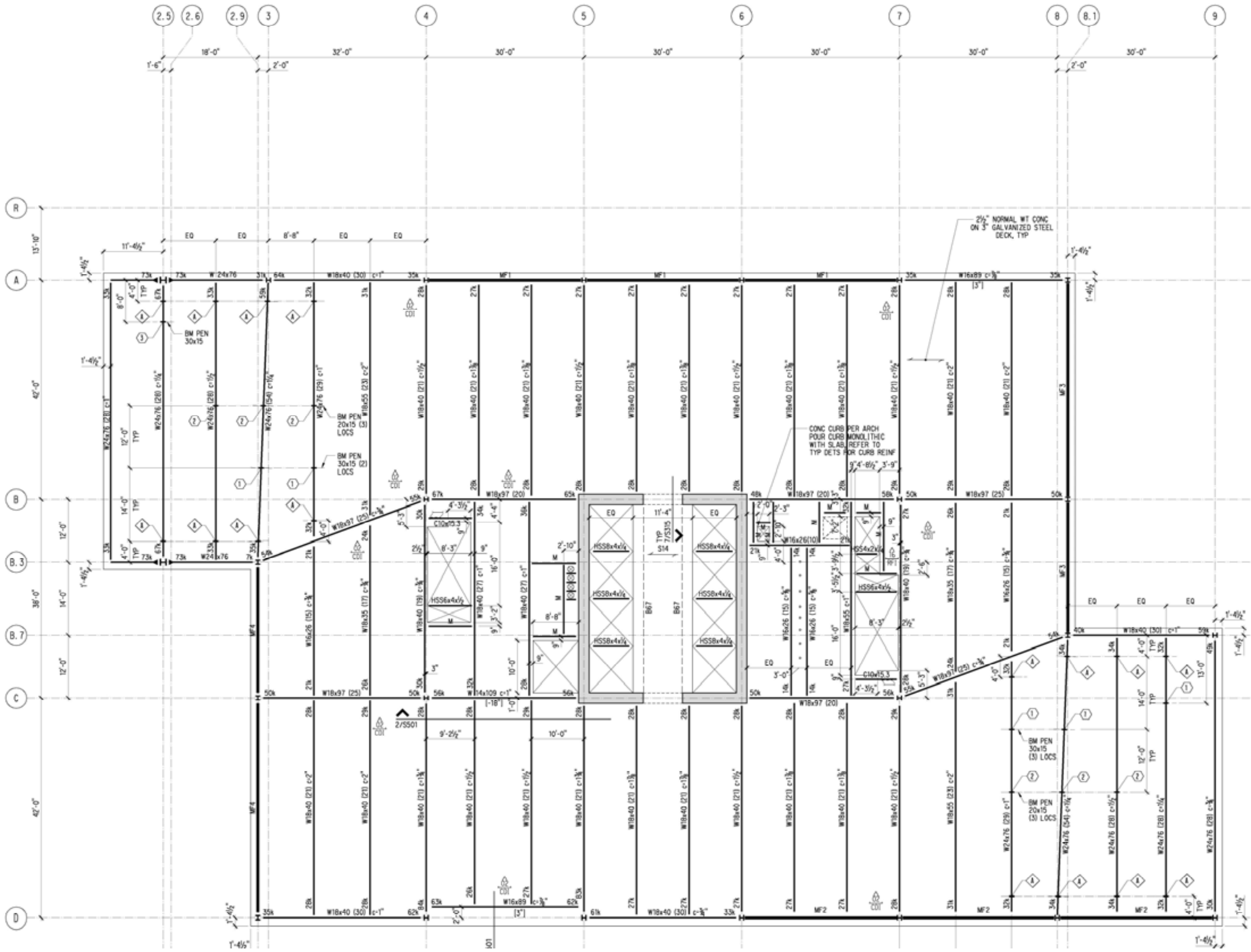

Figure 1:
Existing Structural Steel Floor Framing

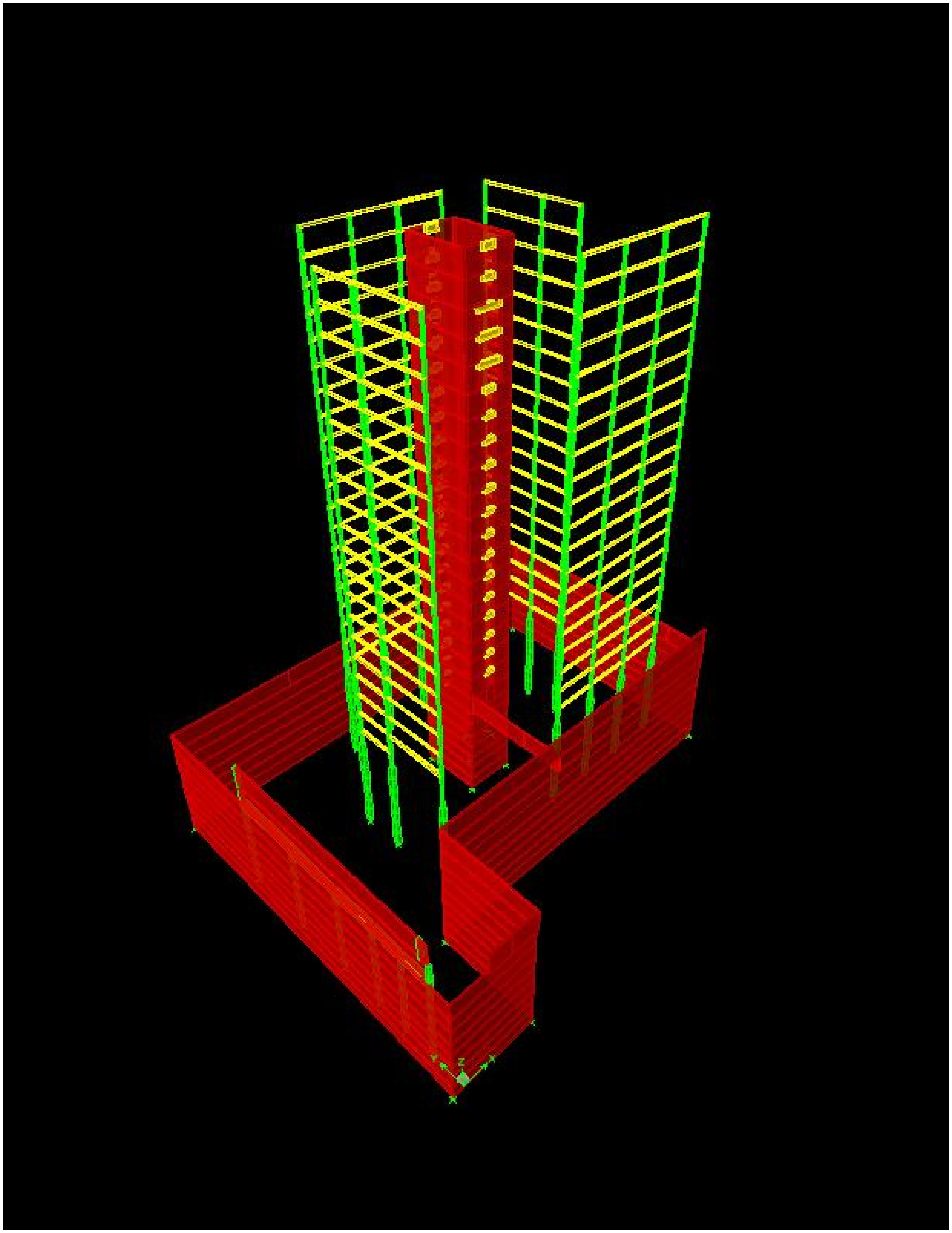

Figure 2.
Existing Dual Core and Moment Frame Lateral System.

Problem Statement:

The primary reason for Tower 333 to utilize the existing abandoned foundation is that it saves time and money. However, along with utilizing the existing foundation and core comes the problem of retrofitting these systems as appropriate to merge with the framing design of Tower 333.

Originally, another building was set to occupy the current site and was roughly the same height as Tower 333. However, the superstructure of the original building was to be cast in place concrete with concrete shear walls outside of the core to help resist the lateral forces. This allowed the core to be small in comparison to the size of the building. Tower 333 on the other hand, is a steel structure and no shear walls are implemented outside of the concrete core. When comparing the existing core for Tower 333 to traditional shear wall cores of similar buildings, Tower 333's core is undersized, despite the reduction in weight from concrete to steel. It is because of this undersized core that a dual lateral system of exterior moment frames in combination with the concrete core was designed for Tower 333. This dual system is required by IBC 2003 for any building over 160' in height to prevent the need for a peer review panel in the design phase.

In an effort to save construction time, labor costs and certain material costs, it is proposed that the exterior moment frames be eliminated and a core-only lateral system be designed for Tower 333. Using IBC 2003 which refers to ASCE7-05 and ACI 318-05, along with software such as ETABS, this thesis proposes to determine whether or not the proposed core-only system is a viable and economic alternative to the existing lateral system.

Depth Study: Lateral System Redesign

Although core-only lateral systems are becoming more popular in seismic regions such as the west coast, the main challenge with utilizing the existing core as a core-only lateral system in the case of Tower 333 is its small size in comparison to the building footprint. Problems encountered with the new design were; base shear resistance, accidental torsion causing large drifts at the top story and the design of the coupling beams. The 30'x 40' concrete core must resist all the lateral loads generated by the 18 story tower. With a floor plate size of 22,000 square feet and an average total dead load including the weight of the floor system, the core walls, and superimposed dead loads of 118psf per floor, the total base shear that the core walls must resist with the new design is 3,772 kips. (For a detail of the floor loads applied to the building see Appendix A1.) Along with resisting the entire base shear, the core must be able to handle the accidental torsion generated by the building's eccentric shape on both axes. This torsion effect was the controlling factor in the design of the core walls due to building drift in the short direction, (North-South.) The most critical component in the whole structure however, was the design of the coupling beams. All coupling beams would have to be designed to resist the massive shear forces generated by the building during a seismic event. These problems, in addition to having to take into account the peer review now required since the building is to be redesigned as a core-only lateral system made this study quite complex.

Peer Review:

As required by IBC 2003, a building over 160 feet in height in a seismic region that does not have a dual lateral system is required to undergo a peer review by an outside engineering firm. This requirement is due to the fact that there are no prescriptive design criteria or procedures for a seismic design of these types of structures in most local building codes, including Bellevue, Washington where Tower 333 is located. Although, due to the performance based design process having increasing popularity, many industry leaders are working towards a more prescriptive form of peer review process which will allow a more efficient and timely peer review to be carried out. Communities such as Los Angeles, and San Francisco already have such provisions in their tall building codes and government documents such as FEMA-349 discuss action plans for developing an efficient performance based design criteria involving peer reviews.

The objective of a structural peer review panel is to provide an objective and technical review of the structure under seismic conditions. The independent peer review is meant to provide the Building Code Director with the knowledge that the building under review is generally conforming to the intent of the seismic design conditions set forth by the local code.

According to Ronald Hamburger, Principal of structural and seismic engineering at SGH's West Coast Region, the scope of the peer review panel should include:

- A review of the design criteria proposed by the Engineer Of Record (EOR) for adequacy.
- A review of the geotechnical and seismic hazards investigation report and evaluate the adequacy of characterization of site response spectrum and selection and scaling of ground motion records.
- A review of the overall structural layout for continuity, redundancy and regularity.
- A review of the modeling and analysis assumptions and actual computer models.
- A review of the analysis results.
- A review of the drawings and detailing to ensure the design was carried out adequately.
- Providing the Building Official with a letter indicating acceptance of the design.
- Interaction with the design team to resolve any differences of opinion or concerns.

In order to avoid disruption in the design and development stage of the building process, a peer review panel should be brought onto the project as soon as possible. This way the review panel will have the ability to provide timely feedback on the fundamental design decisions that would otherwise prove to be detrimental to the schedule if not addressed early on in the design phase.

Code provisions are meant to provide a minimum level of safety for the design of buildings. Often, this is interpreted as a life-safety criterion. The building code allows a building to be designed to sustain maximum structural damage without failure. This allows all occupants of the building to escape relatively unharmed while the building itself would be considered a complete loss. As the development of structural seismic design progresses, more and more buildings are being designed under the peer review criteria. This is leading more engineers into the performance-based seismic design of buildings, where buildings are not only designed to perform under the minimum life safety guidelines, but are also designed to perform to specific criteria set forth by the owner depending on the occupancy and use of the building. Tower 333 under the design proposed by this thesis is considered to fall into this category of performance-based design.

The Federal Emergency Management Agency (or FEMA) produced a document in April of 2000 called FEMA-386. In this document is what FEMA describes as an action plan for performance based seismic design. FEMA describes the basic objective of a performance based seismic design is to allow engineers to design buildings to perform not only reliably but also predictably during a seismic event.

The basic concept to performance based seismic design is the owner chooses the level of performance they want their building to obtain given the specifics of the building's occupancy, and use. There are three basic categories that a performance based seismic design can fall under. The first is Basic Objective, the second is Essential/Hazardous, and finally there is Safety Critical. Each one of these categories can be refined to perform under very specific criteria. (See Table 1. below for these three standards.) From this table the typical predictability of a building can be determined based on the seismic event and the level of performance for which the building had been designed for.

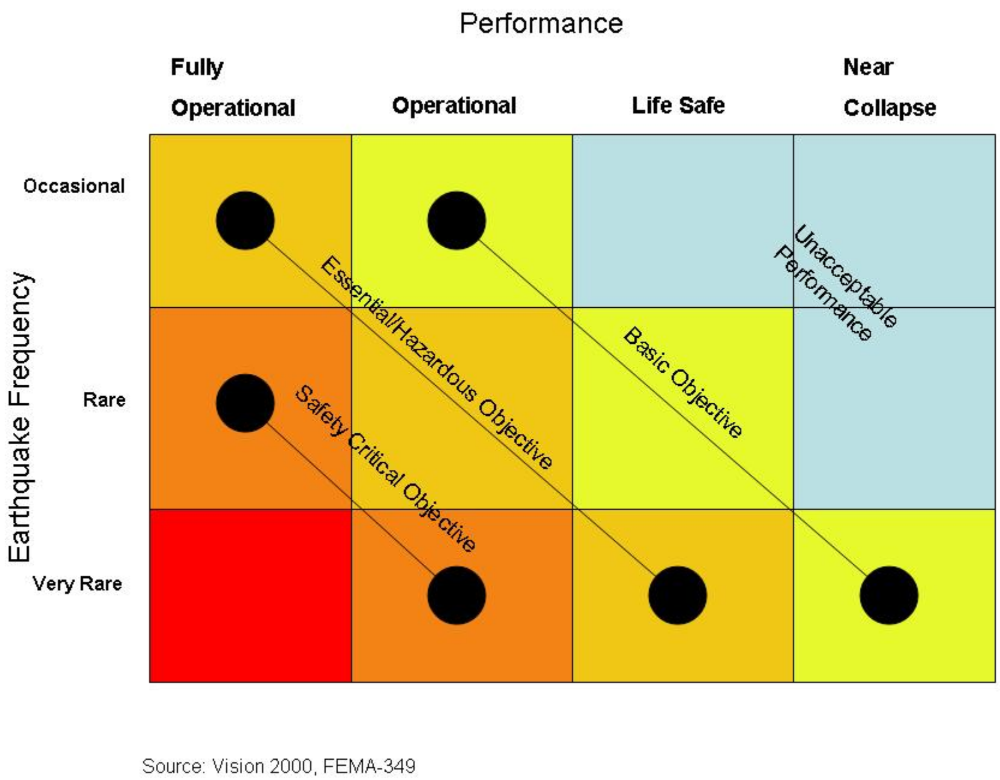

Table 1. FEMA-349 Performance Criteria

For instance, if the owner of a 30 story condominium wanted to design their building to a better performance than merely life safety standards in a rare seismic event, they would choose an Essential/Hazardous Objective for a level of performance. By doing so, the owner protects not only the occupants of the building but also his economic investment in it as well. Had the condo been designed for a life safety and a rare seismic event occurred, everyone in the building would have been able to get out however the building would have sustained so much structural damage that it would be considered a total economic loss and demolished. The cost of designing the building to sustain minor structural damage and repairing that damage after a seismic event might prove much more beneficial to the owner than having to tear the building down and rebuild it, losing valuable income from rent or general disruption of business in the meantime.

Due to the proposed core-only design for the lateral force resisting system in this thesis, Tower 333 is considered a performance based design. The lack of redundancy in the lateral system according to ASCE 7-05 requires the building to be designed with a response modification factor of R=5. This factor accounts for the amount of redundancy in a building's lateral system. The lower the R value, the higher the base shears become, thus resulting in a more conservative design of the building. As discussed in the section below, the proposed redesign of Tower 333's lateral system makes the building behave with a R value of approximately R= 2.0, which is much less than the prescribed life safety requirement of R=5. This over design of Tower 333, along with its required peer review, categorizes it as a performance based design.

Design of Lateral System:

Design of the core walls was performed with an initial calculation based on the building's period and moment of inertia. This resulted in a calculated period of 2.5 seconds and a trial size of the core walls at 30" thick. Elimination of the webs in the concrete core were also implemented and replaced with concrete coupling beams. There were three reasons this was done. The first reason was an attempt to control additional torsion effects by creating a more symmetric core. The second was to eliminate any unnecessary concrete that wasn't imperative to the design. Ultimately, this second reason will allow flexibility in terms of the cost of materials if additional concrete must be added to thicken the core for stiffness. Finally, and most importantly, the elimination of the webs in the core was done to protect the concrete piers from failing in flexure due to the moments caused by the lateral force as discussed below.

By eliminating the webs of the concrete core shear walls and replacing them with concrete coupling beams, a plastic hinge is developed. Allowing the coupling beams to yield in flexure and develop this plastic hinge at the connections to the piers, causes the building to "rack" sideways as it drifts, (see Figure 3.) The intent of the coupling beams is to then transfer the forces via shear from one pier to the other. It is because of this "racking" that allow the piers to be over designed and thus meet peer review criteria. The plastic hinge development in the coupling beams is so important that it makes the coupling beams the critical design component in the success of Tower 333's core-only design. It is absolutely imperative that the coupling beams do not yield in shear during this process so the forces can be spread out and transferred to each concrete pier.

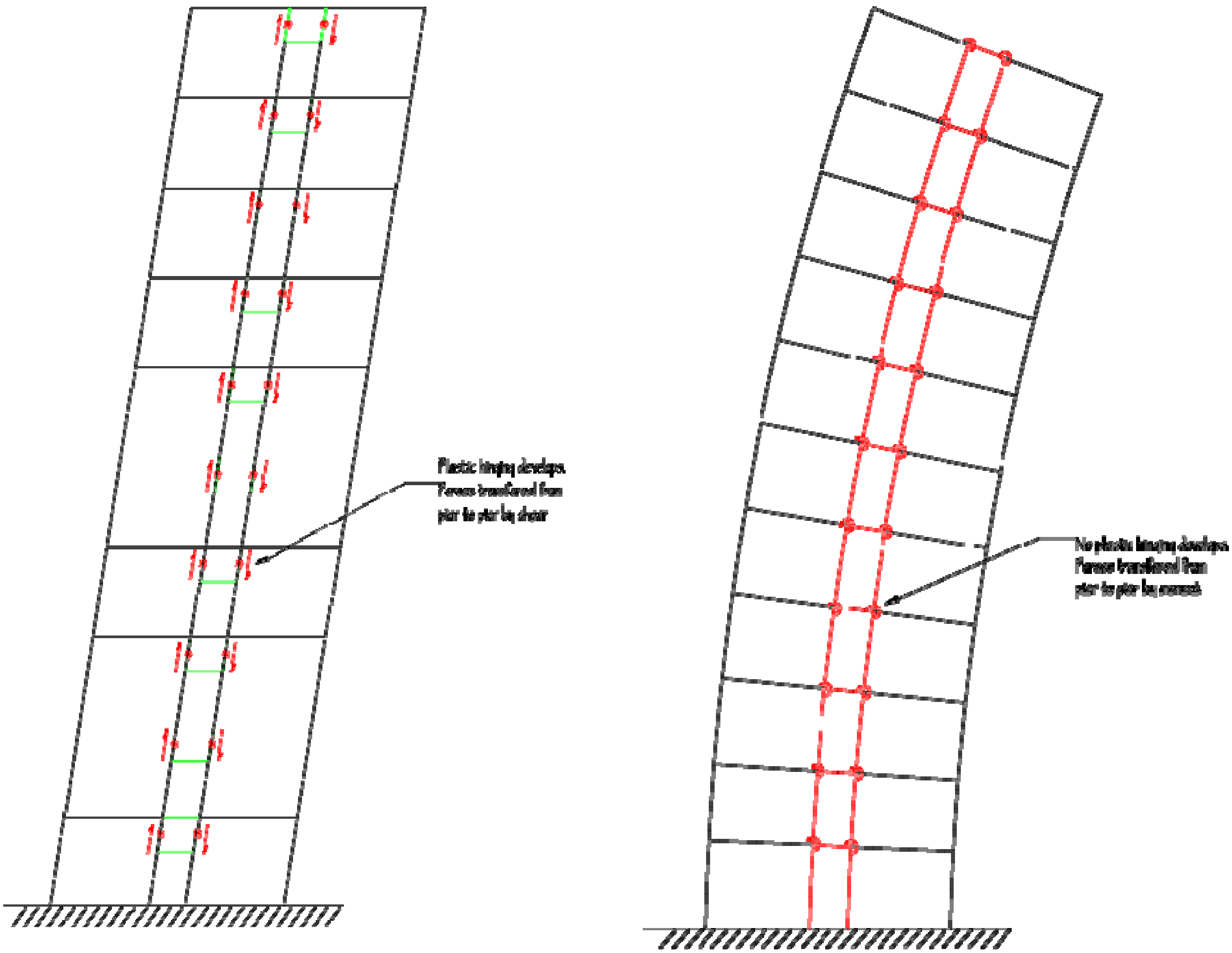

Figure 3. Plastic Hinge Development in Coupling Beams

Building Model:

The building was initially modeled in ETABS using four symmetrical “L” shaped piers 30 inches wide for all the floors, from sub level 8 up to the roof. The coupling beams connecting the four piers were 30” x 45” and were chosen to be the same size for all four connections. (Refer to Figure 4 for the initial design of the core and coupling beams.)

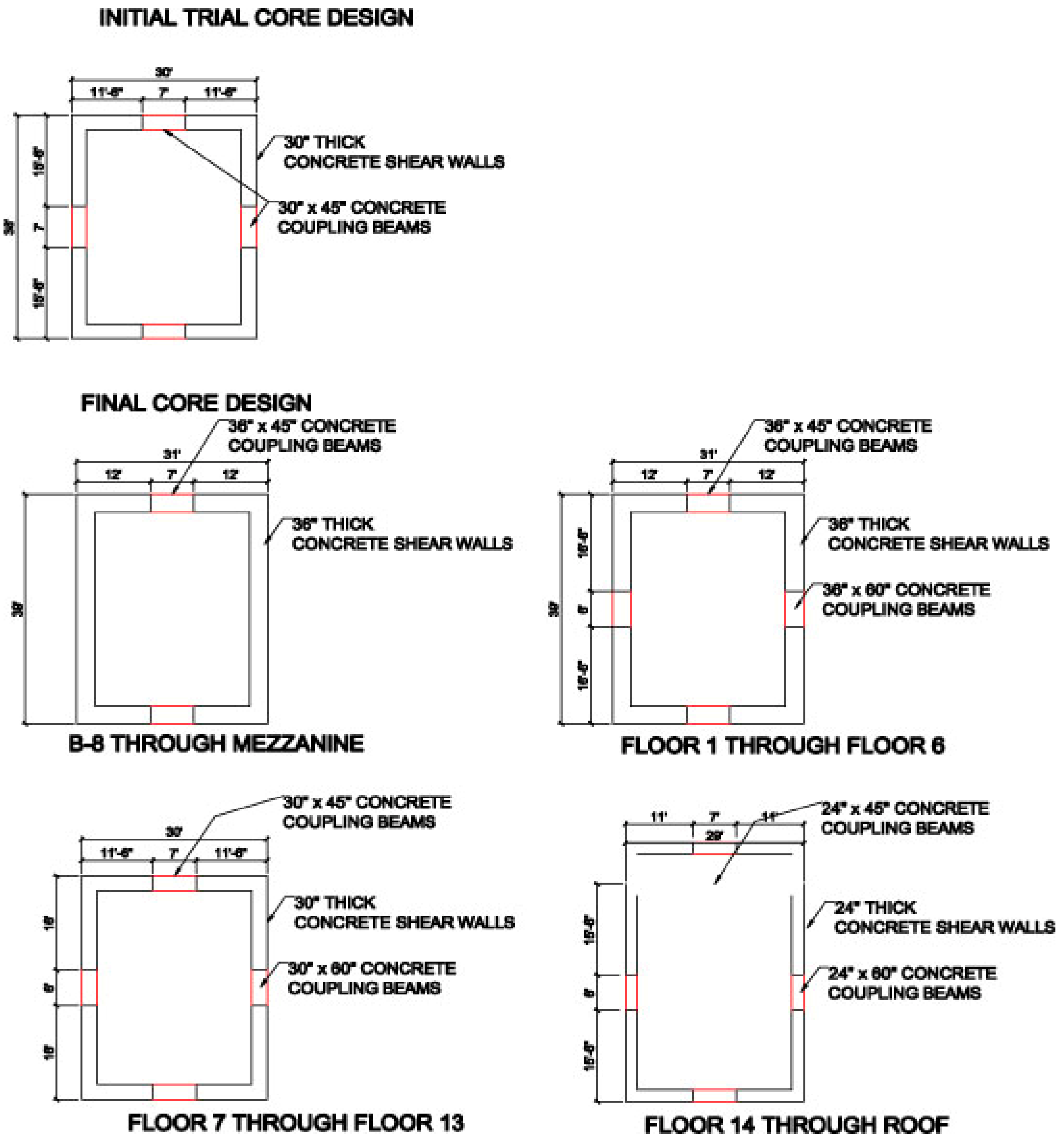

Figure 4. Initial and Final Core Designs

The first criterion considered, which affects the size of the piers, was drift. With a chosen industry standard acceptable maximum seismic drift value of approximately 1% of the total building height, there was an accepted drift limit of 32 inches at the roof. The average story drifts were limited to less than 2% of the floor to floor height, resulting in an allowable limit of 3.3 inches. Forces calculated from the Equivalent Lateral Force Method (ELF) were imposed as static load cases, (see Appendix D1 for final ELF forces used,) and were used to scale the dynamic spectral response load cases to the desired forces. Compared to a full scaling of 386 for the building to behave with an R value of 5, scaling the dynamic forces resulted in a scaled factor of 199 in the X-direction Spectral Force case and a value of 143 in the Y-direction Spectral Force case. This scaling causes the building to behave under an R value of R= 2.0 in the X-direction (East-West) and to behave under a R value of R=2.7 in the Y-direction (North-South.) To represent the seismic base level, translational springs in the X,Y direction and a rotational spring in the Z direction were placed at all levels below Floor 1 where the below grade foundation walls exist. To ensure that no motion would take place at these levels the springs were given a K value of $1e^{20}$.

The model was then analyzed and it was determined that the proposed new design was inadequate under the drift limitations. The critical drift direction was the Y-direction (North-South,) caused by the dynamic loading in the (N-S) direction, and was in excess of 50 inches at the top floor, indicating that a much stiffer design was needed. Refer to Figures 5 and 6 for the behavior of the ETABS model.

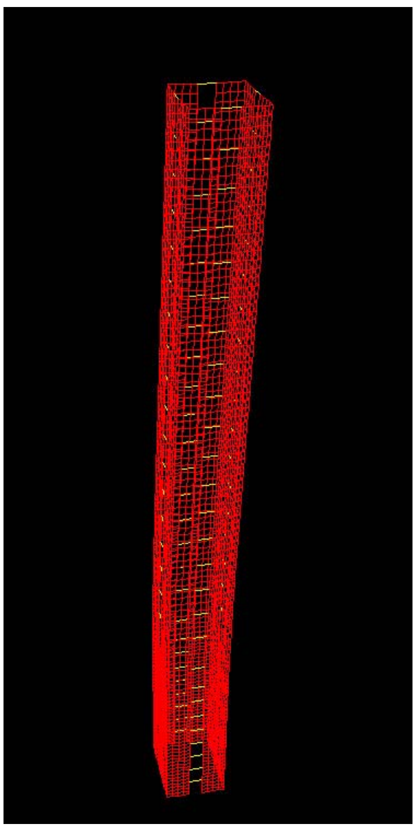

Figure 5. Deformed Shape From (E-W) Dynamic Spectral Load

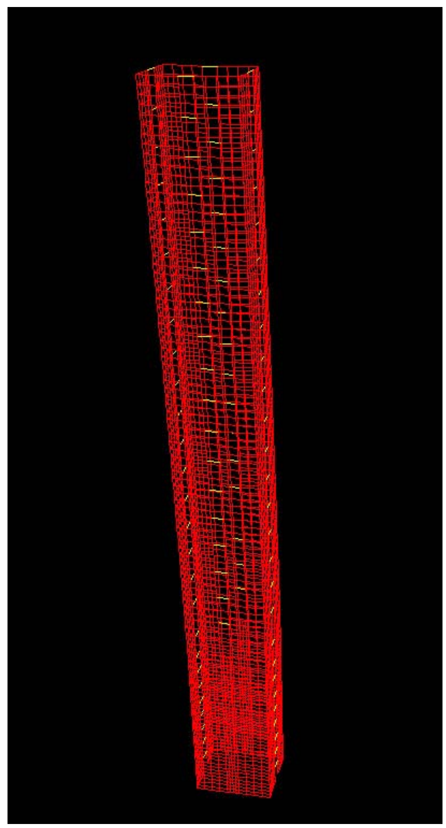

Figure 6. Deformed Shape From (N-S) Dynamic Spectral Load

To add stiffness to the core, the walls were thickened to 36 inches and a series of design alterations were then pursued in attempts to achieve even greater stiffness from the core. These alterations included, adding flanges to the core at the basement levels, adding a flanges to the cores at all levels, closing the web at the basement levels and changing the concrete strength to 12KSI at the basement levels as well as close the web. From these parametric studies, (see Figure 5,) it was determined that the period of the building was sensitive to the flanges that were incorporated and, as predicted, the max displacements were sensitive to the webs in the core. The smallest deflection at the top floor in the critical North-South direction obtained from these simulations was 45 inches, which was 41% over the set limit 32 inches.

Another series of design alterations were then tested. In attempts to lighten the overall dead load at the upper floors, the core walls were reduced in thickness as the height of the building increased, assuming that the upper core wall stiffness's were not as critical as the bottom floors. The core design was then to use 36" core walls from the Basement levels to Floor 6 with closed webs in the basement, 30" core walls from Floor 7 through Floor 13 and 24" core walls from Floor 14 through the Roof. With the idea that the deflection in the North South direction was critical to the amount of concrete in the webs, the opening in the webs of the core were reduced from 7 feet to 6 feet and the depth of the coupling beams in this direction was increased by 6 inch intervals starting at 54 inches to its maximum allowable depth of 72 inches leaving 7'-10" head room at the core openings, (see figure 7.) The preferred design from these sets of studies resulted in a design of 60 inch deep coupling beams.

					Max Edge Point Disp. (in.)		Max Disp. @ COG (in.)		
	Trial	Description	T_x (sec.)	T_y (sec.)	X	Y	X	Y	Max Story Drift
30" Thick Walls All Floors	1	All Coupling Beams 30"x45"	3.46	2.9	22	56	0	18	2.40%
36" Thick Walls All Floors All Coupling Beams 36 x 45"	2a.	All Coupling Beams 36"x45"	3.13	2.58	22.3	54.4	0	15.9	1.85%
	2b.	Add Flange in Basement Levels	2.98	2.3	17.6	44.8	0	14.3	1.70%
	2c.	Add Flange to All Levels	2.17	1.8	17.3	41	0	11.25	1.50%
	2d.	Close Web In Basement	3.07	2.34	17.36	44.9	0	14.5	1.70%
	2e.	Close Web & Use 12ksi Concrete In Basement Levels	3.26	2.4	18	46	0	15	1.70%
36" Walls B-8 through FL. 6 30" Walls FL. 7-13 24" Walls FL. 14-Roof	3a.	45" Deep CB's 7' long (E-W) , 54" Deep CB's 6' long (N-S)	2.97	2.15	13.4	36.8	0	13.73	1.46%
	3b.	45" Deep CB's 7' long (E-W), 60" Deep CB's 6' long (N-S)	2.97	2.13	11.75	33.8	0	13.6	1.33%
	3c.	45" Deep CB's 7' long (E-W), 66" Deep CB's 6' long (N-S)	2.97	2.1	11.5	33	0	13.5	1.30%
	3d.	45" Deep CB's 7' long (E-W), 72" Deep CB's 6' long (N-S)	2.98	2.1	12.89	35.8	0	13.6	1.40%

Figure 7. Core Design Analysis Results From Critical (N-S) Directional Dynamic Loading

With the initial design sizes now determined, the torsion multiplier and eccentricity ratios could be calculated using accidental torsion load cases with eccentricity in the N-S and E-W directions. This resulted in a torsion multiplier, Ax = 1.7 and an eccentricity ratio of 0.085. These values were determined using a ratio of the maximum and average displacement at the top floor, (refer to Table 2. below.)

		δ_1	δ_2	$\delta_{avg.}$	δ_{max}	A_x
Floor-9	AutoEZX1	0.815	-0.815	0.978	0.815	0.694444
	AutoEZX2	-0.815	0.815	0.978	0.815	0.694444
	AutoEZY1	1.06	3.870	2.958	3.87	1.711692
	AutoEZY2	3.87	1.06	2.958	3.87	1.711692
Roof	AutoEZX1	1.43	-1.43	1.43	1.43	1
	AutoEZX2	-1.43	1.43	1.716	1.43	0.694444
	AutoEZY1	3.5	8.53	7.218	8.53	1.396575
	AutoEZY2	8.5	3.58	7.248	8.5	1.375313

ABS	ABS		A_x MAX
0.815	0.815		1.7116919
0.815	0.815		
1.06	3.870		Ecc. Ratio
3.87	1.06		0.0855846
1.43	1.43		
1.43	1.43		
3.5	8.53		
8.5	3.58		

Table 2. Torsion Multiplier & Eccentricity Ratio

After the eccentricity ratio is applied to the dynamic spectrum load cases, an output of the building model was generated. From this model, the maximum shears and moments in the coupling beams and piers were found. A full table of these values is available upon request. For the design intent purposes, a summary of the design values are listed below. (See Figures 8 & 9)

	Beam	Max Mom. (ft-kips)	Beam	MAX Shear
	B1	3467.78	B1	990.79
	B2	3467.79	B2	990.8
	B3	3712.24	B3	1237.41
	B4	3712.24	B4	1237.41
	Group	80% Max Shear (kips)	Average Shear (kips)	$V_{u\ (kips)}$
Beam 1	1	792.6	713.9	792.6
	2	695.7	535.3	
	3	493.0	313.1	
Beam 2	1	792.6	713.9	792.6
	2	695.7	535.3	
	3	342.7	313.1	
Beam 3	1	989.9	789.4	989.9
	2	783.8	511.4	
	3	496.1	262.3	
Beam 4	1	989.9	767.3	989.9
	2	783.8	524.2	
	3	449.2	271.7	

Figure 8. Coupling Beam Load Summary

		Max Moment (ft-k)	
Max Shear (kips)		About Y-Axis	About X-Axis
P_1	NA	41878.05	51476.856
P_2	NA	41878.14	51477.279
P_3	1454.8	NA	NA
P_4	1454.8	NA	NA
P_5	1984.36	NA	NA
P_6	1984.37	NA	NA

Max Axial Load (kips)
7319.07

Figure 9. Concrete Pier Load Summary

Coupling Beam Design:

When designing the coupling beams, code allows the shear design to be reduced by 20% if the beams are considered grouped together to allow load sharing. For this case, the beams were divided into 3 different groups, Floor 1 through Floor 6, Floor 7 through Floor 13 and Floor 14 through the Roof. With this reduction, the design shear must be the larger of either the 20% reduction of the max shear from the group or the average of the shears. For all three groups, the largest shears were the 20% reduced maximum and the largest value in the North-South coupling beams was 990 kips and in the East-West coupling beams 792 kips.

The designs of the coupling beams in the East-West direction utilize standard horizontal reinforcing and shear ties. This is due to the assumption that the lower l_n/h ratio of these beams would not provide an efficient angle for diagonal reinforcing. The North-South coupling beams however have a much larger l_n/h. Utilizing diagonal reinforcing proved beneficial in that case. In accordance with Chapter 21 of ACI-318-05, designing of the East-West coupling beams for the worst case moments and shear yields a steel layout of 14-#11 bars in two rows of 7, top and bottom, with 7-#6 ties @ 4" on center and 4-#5 bars for skin reinforcing. Design of the North-South coupling beams with diagonal reinforcing for the worst case loading yields a steel layout of 8-#11 bars diagonally placed at an angle of 34 degrees with 3-#5 ties at 6" on center, and skin reinforcing of 4-#6 bars on both sides with #5 stirrups at 6" on center. (See Appendix B1 for a detailed layout of both beam designs)

	V_u (kips)	h (in.)	d (in.)	V_u/bwdsqrt(f'c)	Diag Bars	A_d (in^2.)	α (degrees)	ϕV_n (kips)	$\phi V_n/V_u$
Zone 3 (FL. 14-18)	421	60	48	3.9	6-#11	9.36	33.7	530	1.26
Zone 2 (FL. 7-13)	667	60	48	6.5	8-#11	12.48	33.7	706	1.06
Zone 1 (FL. 1-6)	841	60	48	5.1	10-#11	15.6	33.7	883	1.05

Figure 10. Diagonal Beam Design

Pier Design:

Designing of the piers for flexure was done using PCA Column. In this case, the piers are treated as one unit due to both legs combine to resist moment in each direction. It was determined that the controlling load cases for the design would be, *0.9Dead ± 1.0Ex ± 0 .3Ey and 0.9Dead ± 0.3Ex ± 1.0Ey*. From the building output table developed from the ETABS analysis, the maximum moments in the North-South and East-West directions as well as the maximum axial force were determined. A summary of those loads are listed below. Due to the symmetrical design of the core it was only necessary to design one pier and utilize that for all four piers. As a comparison check, two models were created in PCA Column, one for the design of the worst case loads at Floor 1 and another for the design of the loads half way up the structure at Floor 9. (Refer to Appendix B2 for a detail of the steel design in the piers.)

For the piers at Floor 1, a 1.5% ratio of steel to concrete was required. A minimum requirement of 9.5 square inches per foot of reinforcing in the East-West leg of the piers and 4.25 square inches per foot of reinforcing in the North-South leg of the piers thus resulted from the minimum steel ratio. A bar layout was then designed to be 5-#14 bars @ 12" on center in the East-West Leg and 3-#11 bars @ 12" on center in the North-South legs.

Comparatively, a minimum steel ratio of 0.25% was required for the piers at Floor 9. Minimum steel area in the pier legs was determined to be 1 square inch per foot in the East-West leg and 0.9 square inches per foot in the North-South leg. The bar layout for Floor 9 is then 2-#7 bars @ 12" on center in both the East-West and North-South legs.

When designing the piers for shear, instead of treating both legs as one unit as was done for the flexural design, the piers have to be analyzed with each leg as a separate entity. The core walls were modeled in ETABS to have a zero shear capacity in the out-of-plane axis so that the in-plane shear would receive the full shear load. Again, for comparative results, the shear reinforcing was designed for both Floor 1 as well as Floor 9. As such for the design of the piers in flexure, the design of the piers for shear need only be done for one of the piers due to the core's symmetrical design. Using Chapter 21 of ACI 318-05, the legs in the East-West direction on Floor 1 require a minimum steel ratio of 0.0026. This is greater than the minimum requirement for seismic design which is 0.0025 therefore; the piers require #7 bars at 12" on center. The legs in the North-South direction require a minimum steel ratio of 0.0028, which also resulted in a bar layout of #7 bars at 12" on center.

In comparison, Floor 9's minimum steel ratio for both legs of the pier was well below the minimum requirement of 0.0025 and as a result, the bars layout in both legs was also designed to be #7 bars at 12" on center.

Breadth Study #1: Cost Analysis & Schedule Reduction

Cost Analysis:

Ultimately, the goal of the alternative, core-only lateral system was to provide a system that not only performed well under seismic conditions, but that was also economical for the owner. If the cost of the proposed lateral system for this thesis study resulted in a higher cost for the owner than the existing system it would not be a beneficial option for the owner to pursue. For this breadth study, a cost analysis of the overall lateral system was performed. Included in this analysis were costs of material, shop labor, erection time, and rental income of the tenants.

Removal of concrete from the core on the upper floors resulted in a beneficial savings of material cost. Eliminating a 2 ft thick x 6ft x 13'-10" section from each set of piers at each floor added up to 234 cubic yards of concrete saved. This resulted in a cost savings of $152,000. However, the proposed design of the core only system does require an additional volume of concrete to be added to each floor for the wall thickness previously discussed. The breakdown of the added concrete is as follows:

Sublevel 8 through Mezzanine: 36.4 CY/floor
Floor 1 through Floor 6: 50.4 CY/floor
Floor 7 through Floor 13: 25 CY/floor

The added cost of the concrete and steel added to the core was $523,000, which is a significant increase in cost.

Along with the added concrete, consideration must be taken into account regarding the fire rating of removed portions of the core that expose the elevator shaft. With 6,408 ft^2 of fire rated drywall needed, the additional cost of providing fire rated walls is $23,700.

However, removal of the moment frames saved money in two aspects of the budget; the material and labor cost of the moment connections, and the time saved in erection of the steel frame by significantly reducing the field labor. After contacting a steel fabricator for representative data for the Seattle area, it was determined that a significant amount of money could be saved in the shop labor process of creating the moment frames. The shop costs of creating a moment connection end was $910/end. With approximately 400 ends of members requiring moment connections, the savings of these connections totaled $364,000. Adding to that, the cost of $380/ doubler-plate location and with the 280 doubler-plate/stiffeners locations located in the moment frames an additional savings of $106,400 is realized. The largest savings however, came in the tonnage of steel saved by switching the heavy moment frames down to smaller gravity beams and columns. With a total weight of 682,000 pounds saved a cost savings of $785,000 was realized. The total money savings in the shop production of the moment frames then became $1,255,000. This figured doesn't include savings in erection labor which the steel shop consulted equated to 4,000 hours of field labor.

A savings of 4,000 hours of field labor achieved in conjunction with steel erection being on the critical path of the building erection sequence results in a significant impact on the building schedule. Using R.S. Mean's suggested erection crews of one E-6 crew of 16 workers and one E-9 crew of 16 workers for the erection sequence a total of 256 man hours per day is achieved by the combined two crews. With the savings of 4,000 field hours saved from the moment frames spread out between the two crews results in 16 days worth of man hours cut from the erection schedule.

The erection sequence provided by Hines has 210 total erection days scheduled for the steel. This figure was from the most up to date schedule post tower crane collapse. Saving 16 days on this figure results in an average 7.6% reduction time over the 210 day schedule. In order to ensure an even distribution of time savings over the total erection sequence and not just one portion, the 7.6% reduction was distributed over the entire 210 days. A table for the total amount of days saved on each erection sequence is provided below.

Steel Erection Sequence	Erection Days	Erection Days With 7.6% Reduction	Days Saved Per Sequence
1 & 2	16	15	1
3 & 4	18	17	1
5, 6, 7	21	20	1
8 & 9	19	18	1
10 & 11	19	18	1
12, 13, 14	19	18	1
15 & 16	19	18	1
17 & 18	19	18	1
19 & 20	19	18	1
21 & 22	19	18	1
23, 24, 25	21	20	1
26	1	1	0
	Total Days Saved		
	11		

Table 3. Erection Sequence Savings

According to the table, a total of 11 days off the critical path in erection time was saved through this 7.6% reduction in schedule. With an E-6 crew costing $8,277/day and an E-9 crew costing $8,468/day, (both including O&P,) and multiplying these costs out over 11 days, the total cost savings in man hours results in $184,206. Scaling this number to inflation rates and location factors yields a total savings of $221,581 in erection time. Adding these 11 days of savings into the building schedule saves even more money.

To model the updated building schedule with the 11 days worth of savings, Microsoft Project was used. A simplified schedule was then created using the existing start and end dates of the building schedule modified to include the time savings of each sequence from the proposed alternative lateral system design, (Refer to Appendix C1 for Building Erection Schedule) From this schedule, 7 days of building schedule was able to be saved. Ultimately, this allowed the building to be finished one week ahead of the current set finish date. With this early finish date, the tenants are therefore permitted to move into the building a week early, thus adding an extra week's worth of rent into the owner's revenue.

Although the exact figures of each of the tenants' rent are confidential information, it was determined that the average rent for Tower 333 is close to $25/ft^2 a year. This figure is the net income after the cost of utilities and other services are recovered. Tower 333 utilizes 22,000 ft^2 per floor. With 18 floors of rentable space this figure totals 396,000 ft^2 of rentable square footage and at $25/ft^2 per year the total cost of one week's worth of rent is $190,400. In addition to this figure, there are 951 parking stalls in Tower 333's below grade parking garage. At a rental rate of $47/week per stall, a total of $44,700 is also added to the early revenue. Therefore, the total amount of revenue obtained from the early finish date is $235,100 which can be added to the total net savings of the proposed design.

A breakdown of the savings and costs of materials, fabrication, erection and tenant rent is summarized below.

Summary of Building Cost for Core-Only Lateral System:

Concrete saved: -- (+) $152,000
Concrete added: -- (-) $523,000
Fire Rated Walls: -- (-) $23,700
Steel shop production: -------------------------------------- (+) $470,400
Steel Material: -- (+) $785,000
Labor/Erection: -- (+) $221,900
Rent Revenue: -- (+) $190,400
Parking Revenue: -- (+) $44,700

Total dollars saved with proposed core-only design :------(+) $1,320,000

From these results, it was determined that a beneficial overall savings of $1,320,000 is achieved with the design of the core-only lateral system as proposed by this thesis. With a total building cost of $156.4 million this equates to a distributed building cost of $263/ft^2 over the total 594,000ft^2. A savings of $1,320,000 results in a diminished cost of $1.30/ft^2 to bring the final distributed building cost out to $261.7/ft^2. The total savings of $1,320,000 does not include the money saved on financing and general administrative costs, which according to Hine's Development could potentially have a larger savings than the rental revenue totals.

Breadth Study #2: Building Envelope Performance & Quality Control

The main purpose of a building envelope is to prevent wind and rain from penetrating the inside of a building. Water penetration can cause numerous problems to a building including, but not limited to, deterioration of polymer sealants, rust and mold growth. Despite the many advancements over the years, water leakage into a building is still quite common. The main reasons for this problem are due to poor design and poor workmanship during installation.

This study proposes to look at the process in which building envelopes are tested and installed and provide a list of recommendations to which the owner of Tower 333 could put to use to ensure that the building envelope is a quality product. While these recommendations will add additional first cost to the building’s budget, the benefit of these added costs will far out weigh the additional costs of inspection, repair, and potential litigation to the building if water penetration is discovered after construction is completed. These recommendations include mockup testing, field testing and third party inspection and tests.

There are two main testing methods for building envelope performance, one is the mockup testing for water penetration and the other is a field test for water penetration. Each of the two tests have their own pros and cons. Both tests should follow the testing procedures described by ASTM E331 “Standard Test Method for Water Penetration of Exterior Windows, Curtain Walls, and Doors by Uniform Static Air Pressure Difference” as well as the procedures described by AAMA 501.1-05 “Standard Test Method For Water Penetration of Windows, Curtain Walls and Doors Using Dynamic Pressure.”

A mockup test involves the building of a full scale mockup of the building envelope and testing the water penetration under laboratory conditions for the two different tests mentioned above, ASTM E331 and AAMA 501.

ASTM E331 requires a test chamber be set up to provide a static air pressure difference on both sides of the mockup. Unless otherwise specified this air pressure difference should be 137 Pa (2.86lbf/ft^2) Water is then sprayed evenly over the exterior surface at a minimum rate of 5.0 gal/ft^2 hr. If there are any flaws in the construction of the envelope, the difference in air pressure between the inside and outside of the mockup will draw the water through to the inside. Water penetration as defined by ASTM 331 is the penetration of water beyond a plane parallel to the glazing (the vertical plane) intersection the innermost projection of the test specimen, not including interior trim and hardware, under the specified conditions of air pressure difference across the specimen. Although this test does represent capillary action and migration of water under differential pressures, it does not accurately represent dynamic wind driven rain. Therefore, a second test is required for such a case.

The American Architectural Manufactures Association or AAMA requires a testing of water penetration under dynamic wind load conditions. This is done through a wind generating device, usually an aircraft propeller that is capable of providing a wind stream equivalent to the required wind velocity pressure that the building would see, (see figure 11.) Sometimes when such a device is unattainable a pressurized hose test is supplemented, however this is not as accurate.

The dynamic test starts when water is evenly applied to the outdoor face of the specimen at a rate of 5 gal/ft2 hr. This test should be conducted for no less than 15 minutes. Any water leakage through the envelope is to be documented and reported. AAMA 501 defines water leakage as any uncontrolled water that appears on any normally exposed interior surfaces, that is not contained or drained back to the exterior, or that can cause damage to adjacent materials or finishes. An allowable limit of up to ½ oz. of water (about the amount of a teaspoon) in a 15 minute interval on top of an interior stop shall not be considered water leakage.

Figure 11. AAMA Dynamic Water Test

Although mockup testing provides a controlled condition in which to test the specimen, it is not accurate for real life scenarios. Usually during the installation process of the mockup, great care is taken to ensure quality construction which often does not represent actual construction practices. Therefore, a field test should also be provided.

The field test requires a water penetration test of the actual building envelope as it is erected on the building. The process for both AAMA 501 and ASTM E331 tests in the field are similar to that of the laboratory mockup test. For the ASTM test, an air tight chamber is built on the inside of the building where the test is to take place. Differential pressure is then applied to the inside of the chamber and a curtain of water is evenly distributed on the outside area to be tested

. Similar to the AAMA laboratory test, for the dynamic field test, a mechanical device capable of producing wind pressures equal to that of the designed wind pressures is installed and an even distribution of water is applied to a specified section of the envelope. Water penetration is then identified, documented and repairs implemented. The advantage to the field test is that real life conditions and construction are evaluated; therefore the test produces a more accurate representation of the actual conditions that the building encounters. One drawback to the field tests is that if damage to the building envelope occurs, it is damage to the actual system in place and repairs must be made, as apposed to a mockup where if the specimen is damaged during the test is considered expendable. Although both laboratory and field tests should be specified by the Architect or Building Envelope Designer, it is beneficial if a third party is also brought in to ensure quality control.

By hiring a third party to assist in the design, inspection and testing of the building envelope a better quality assurance program can be implemented. This third party should be brought onto the project as early as possible to ensure a timely manner in which problem areas can be addressed and fixed. Although the third party should be on site during the building envelope erection sequence to monitor quality construction, they are not a substitute for the normal construction administrative services. Usually third party inspection teams will provide on site random inspections and testing of the building envelope as it is being erected. This way, if certain areas of the envelope are discovered to have similar problems, they can be addressed early on in the erection phase and avoid costly repairs later on. To ensure an accurate representation of the building envelope's quality of construction, these inspections are usually done on three random locations throughout the building's façade. This way, the erection crews will have no prior knowledge of the location to be tested and will not provide quality erection for that zone and not the rest. Out of the three zones tested, if one zone should fail the inspection, the crew will test another random set of three zones. Again if one of these three fails another three will be tested and so on until all three randomly chosen zones pass inspection. It is in this manner that an accurate percentage of the performance of the building's façade can be created. From here a full evaluation of the performance and quality of construction of the building envelope can be determined.

For the case of Tower 333, mockup testing, field testing and third party services should be provided in the specifications for the building envelope. While such additions to the construction and inspection of the building's façade could add anywhere from $50,000-$100,000 to the building's budget, these costs are much more beneficial to the owner than the possible millions of dollars in damage that could occur 5 or 10 years later in the building's lifespan. Therefore it is recommended that the owner apply to Tower 333's building specifications the list of laboratory and field tests for water penetration as well as hire a third party consultant to ensure a quality building envelope is constructed

Tower Crane Collapse:

Although construction accidents do happen, very few are catastrophic such was the case with collapse of Tower 333's tower crane on November 16th 2006, which resulted in the death of one individual. When such deadly accidents do occur however, it is imperative that the cause of the accident is identified and that we learn from the mistakes. Based on information to date from the press and investigators, the possible primary causes of the collapse of Tower 333's tower crane are;

- Possible ice accumulation in the crane's supports, causing severe stress and hairline cracks to form in the structure,
- Failure to unlock the crane boom to allow weathervane action during high winds
- Possible flaw in the unique design of the crane's base.
- Material defects
- Erection errors

Due to the fact that the investigation of this collapse is still under way, the actual cause of the collapse has not yet been determined. Therefore, additional discussion of this aspect of the construction is not a formal part of this thesis project. For additional information on the collapse of the tower crane visit the Author's Thesis CPEP website.

Figure 12. Origional Tower Crane 4 Days Prior To Collapse

Figure 13. Replacement Tower Crane, Post Collapse

Conclusion:

The purpose of this thesis study was to design and analyze an alternative lateral force resisting system for Tower 333 located in Bellevue Washington. The existing lateral system is a dual shear wall core at the center of the building and special moment frame system along the perimeter. Due to the use of an existing abandoned building site for the concrete core, the moment frames needed to be utilized to control the drift effects caused by torsion in the undersized core. These special moment frames are also mandated by ASCE 7-05 to be designed to resist at least 25% of the buildings total base shear. A dual system, such as the existing one, also provides the owner with the ability to bypass the peer review process for tall buildings over 160 feet in height as mandated by IBC 2003. Through an ETABS analysis of the existing design using the ELF method to apply static forces to the structure, it was determined that the current design conforms to all code specifications and assumptions made by the Engineer of Record.

A proposed lateral force resisting system design alternative for this thesis study was to eliminate the exterior special moment frames and design a core-only lateral system. To be a plausible and beneficial alternative to the existing lateral system, the proposed design would have to be able to stand up to all the same criteria as the existing system as well as be cost effective. Since the proposed alternative system eliminates the moment frames, the lack of redundancy in the lateral system also requires that the building be scrutinized by a peer review panel which also mandates more stringent design criteria due to its performance based design.

The final proposed design of the lateral system is a core-only structure with 36" thick "C" shaped walls at all 8 of the below grade levels. From Floor 1 through Floor 6, the walls remain 36" thick but eliminate the center webs of the core and transfer to four symmetrical "L" shaped walls with concrete coupling beams. The "L" shaped core walls from Floor 7 through Floor 13 shrink to 30" thick with 30" thick coupling beams and from Floor 14 through Floor 18 the core shrinks again down to 24" thick along with the coupling beams. Through an ETABS analysis undergoing dynamic spectral load cases and taking into consideration criteria such as strength, stiffness, torsion and story drift the final design was determined to be a feasible alternative to the original design.

In addition to the feasibility of a workable design, the proposed changes also resulted in numerous savings and added revenue due to the proposed alternative design. Savings in material cost, shop production and labor, field erection time and labor as well as added revenue from an early finish date were all realized. A total cost savings from switching to a core-only lateral system was determined to be $540,000.

Therefore, in considering the feasibility of design, the cost savings as well as time saved in an early finish date, it is the recommendation of this educational thesis study that the proposed core-only lateral force resisting system is the recommended option in lieu of the existing dual, core and moment frame system.