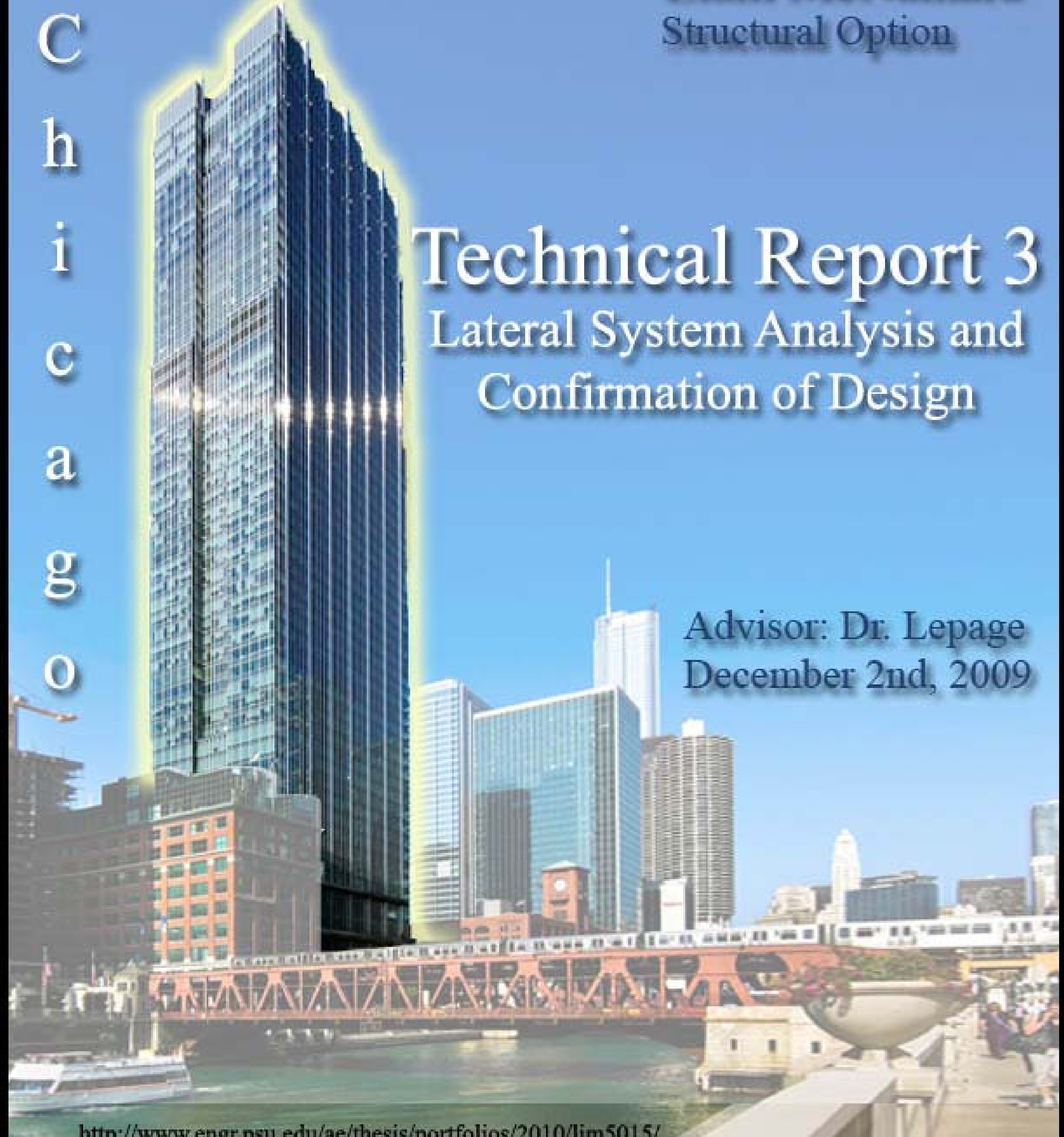
300 North La Salle

Liam McNamara
Structural Option

Chicago

Technical Report 3

Lateral System Analysis and Confirmation of Design



Advisor: Dr. Lepage
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http://www.engr.psu.edu/ae/thesis/portfolios/2010/ljm5015/

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Executive Summary

The third technical report for 300 North La Salle will focus on lateral system analysis and confirmation of the original design by Magnusson Klemencic Associates (MKA). The lateral loads calculated in the first technical report on structural concepts and existing conditions were applied to the lateral force resisting system. The lateral force resisting system for 300 North La Salle is a reinforced concrete shear wall core. The core is stiffened by a series of 3 outrigger trusses running through the building north-south working in unison with two belt trusses running east-west on the north and south exteriors. An ETABS computer model was created and once verified its output was used to determine the controlling ASCE 7-05 load combinations. The ETABS output was also used to spot check the strength and serviceability of the building.

Only the lateral system of the building was modeled in ETABS. This was assumed to include only the shear wall core, the outrigger and belt trusses, and all the columns that support the trusses. The gravity systems were not modeled at this stage, as they would make the model much more complex. However, the building weight is still included as an area mass on the rigid diaphragms that make up each level and provide a seismic response.

After running the various load combinations it was determined that the 1.6W combinations would control the lateral forces in both the north-south and east-west directions. Further examination will be needed to determine the effects of various other loads such as live, dead, and snow on the load combinations.

Hand Calculations verified the relative stiffness and center of rigidity that ETABS produced for several floors. Therefore ETABS output was used to verify that the shear walls designed can adequately carry the applied lateral loads. The output and spot checking also confirmed that the building stays within successfully meets strength and serviceability requirements. Some areas that require future examination are the overturning moment's effects on the foundations, wall 5's high internal shear at select levels, and the shear returns observed between floors 41 and 43.

Introduction

300 North La Salle is a 60-story high rise office building located on the north bank of the Chicago River in Chicago Illinois. It offers 25,000 gsf of rentable, column free floor space per level, with a total square footage of 1.3 million. Construction on the building began in 2006 and was completed in February of 2009 at a cost of $230 million. It is owned and managed by Hines developers and was designed by Pickard Chilton Architects. The primary tenant is Kirkland & Ellis, Chicago's largest law firm, occupying between 24 and 28 floors.

300 North La Salle rises elegantly above the Chicago River with a subtle set back above the 42nd floor. Its "fin-like" steel outriggers and aluminum mullions emphasize verticality. The appearance of structural members on the façade as well as the large open floor plans allude to Mies van der Rohe and the international style he helped make famous in Chicago. His international style incorporated open "universal" spaces that were easily adaptable with clearly arranged structural framework and a "simple is beautiful" motto.

The structural engineers for the design were Magnusson Klemencic Associates. The superstructure is composed of a bearing concrete core and six steel W-shape outrigger trusses spanning north-south, three on both cardinal sides, and two belt trusses spanning east-west on the north and south faces of the building. The trusses are all located between floors 41 and 43. The bearing concrete core wall also acts as a shear wall core to carry lateral forces to the foundation. The "belt" of trusses spanning from the 41st to 43rd floors aides in controlling lateral deflection of the structure and rotation within the shear wall core. The concrete strength of the core varies between 6,000 and 10,000 psi and the wall thicknesses vary between 1'6" and 2'3".

The typical floor system is composite beam with steel decking. It is composed of a 3" cast-in-place concrete slab on a 3" steel deck, and W-shape steel beams. The composite decking is typically 4,000 psi light-weight concrete. The steel members are Fy = 50 Ksi except for select columns on the lower level that are high strength Fy = 65 Ksi steel. The typical bay size is 28.5' x 45'. The system was chosen to efficiently span the 45' length creating a column free floor plan between the core and exterior of the building.

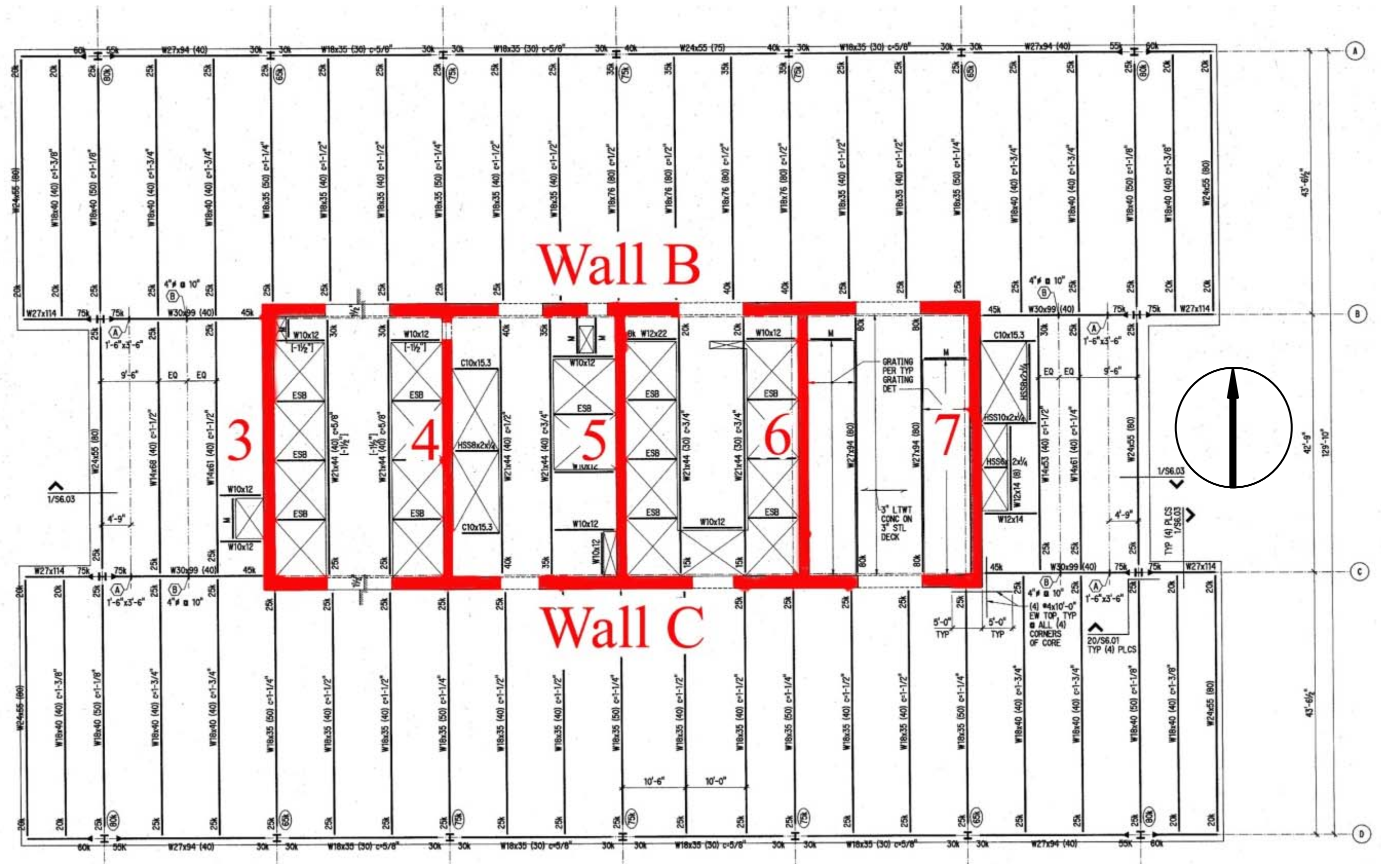

Figure 1 : Core Layout and Wall Designation

This report will be an in depth examination of the existing lateral system composed of the reinforced concrete shear wall core with outrigger and belt trusses. The study will include a computer model of the building performed on ETABS computer software. Once the accuracy of the model is confirmed through hand calculations, it will be used to determine a reasonable distribution of the lateral loads to the lateral resisting members. Governing load patterns will be determined for both the north-south and east-west directions. Spot checks will then be used to confirm the strength of the shear wall core, and the building's ability to meet suggested serviceability requirements.

Existing Structural System

Foundations:

The foundation of the building is a combination of poured concrete piers and driven steel H-Piles with a 12" concrete slab sloping away from the core. The foundation slab is 28'-3" below grade and the foundation walls are 18" thick cast-in-place concrete around 3 levels of sub grade parking. The piers are drilled to approximately 72' below grade from top depths of 27'-41' below grade and have a bearing pressure of 40ksf. The piles are driven to refusal in bedrock at approximately 110' below grade and have a design bearing strength of 270 tons.

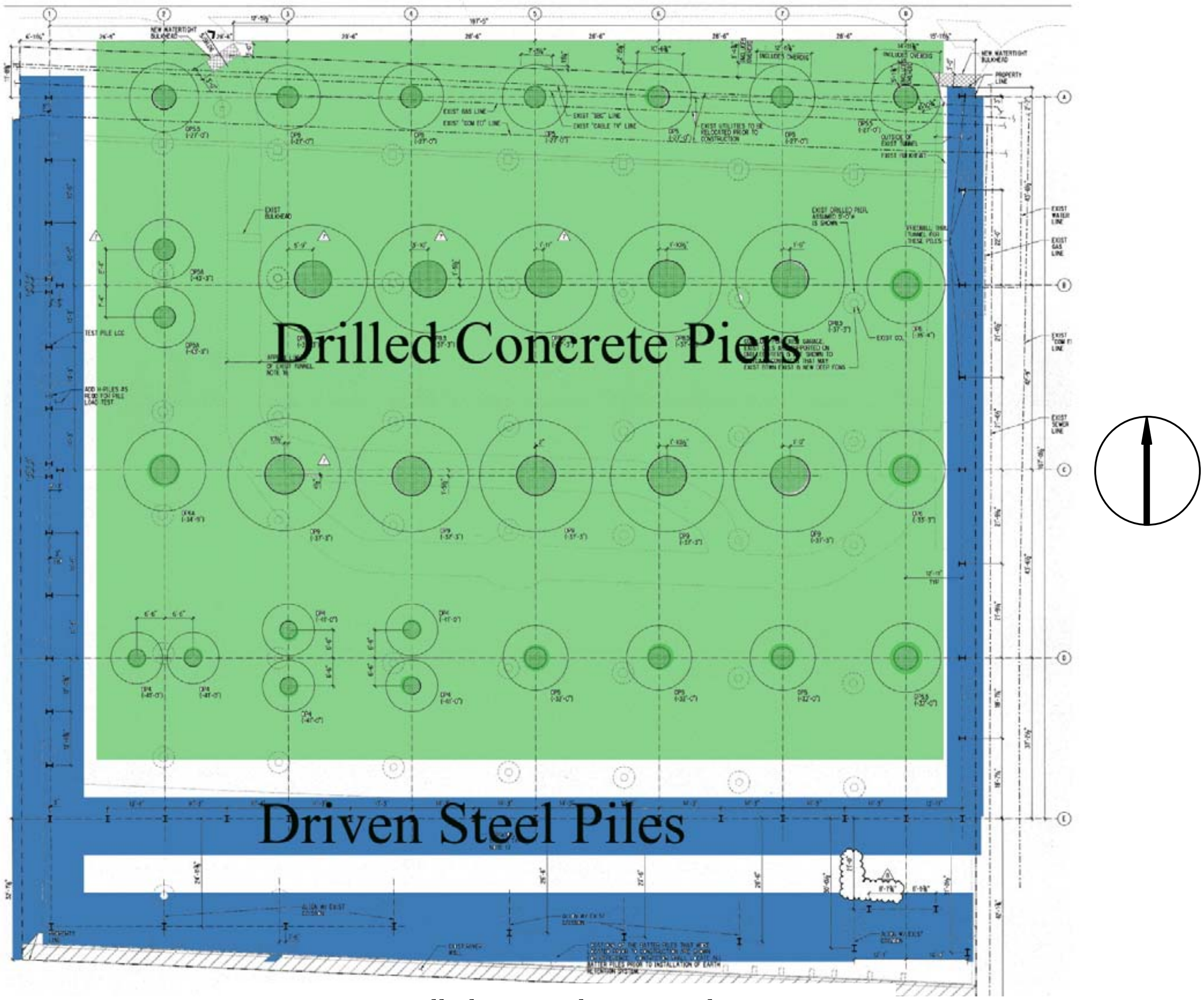

Figure 2 – Drilled Pier and Driven Pile Locations

Gravity System:

The main gravity-load is carried to the ground by exterior steel columns and an interior concrete core wall. The floor system on every floor is poured concrete slab over composite decking. While the slab varies from 3" light-weight concrete, on the office floors, to as thick as 8" normal-weight concrete in the mechanical area, the deck is a consistent 3" Type W minimum 20 gage galvanized steel. The composite decking transfers its loads onto 50ksi steel Wide flange beams typically spanning between 42'-9" and 43'-6½" spaced at 9.5' o.c. Below the elevator pits and Com Ed rooms on Lower Levels 1-4 the slab changes to normal weight 2-way flat concrete slab between 12" and 14" deep. The thickened two way flat slab is used to more readily carry the large live loads in these areas to the core. The roof system is also a light-weight concrete slab on 3" decking, however the beam size is increased to carry the additional weight from the green roof around the core of the building.

Lateral System:

Wind and seismic forces are resisted by a concrete shear wall core, strengthened by a series of outrigger and belt trusses between the 41st and 43rd floors. The shear wall core is cast-in-place normal weight concrete of 6,000; 8,000; and 10,000 psi strength depending on location. The wall reduces in thickness and plan as it rises through the building. The thickness reduces from 2'-3" to 2'-0" and then to 18" on the north and south walls at levels 9 and 43 respectively. The core has four 28'-6" bays running east-west as it rises from Lower Level 4 to Level 42, at Level 43 the core drops its outer two bays and continues through the penthouse with the inner two bays. The shear wall's step back to two bays corresponds to a 10' reduction in east-west width, at the top of the two story "belt" truss system. The floor and roof diaphragms carry the lateral loads to the shear wall core. The shear walls in the core then transfer the base shear, overturning moment, and rotational forces to the foundation.

The belt truss system is comprised of two multi-bay braced frames running east-west on the north and south exteriors, and three braced frames spanning north-south to the concrete shear wall on the interior of the building. The truss members are varying sizes of steel Wide flanges. The purpose of this "belt" truss system is to create a couple moment, from the outrigger steel columns in the event of lateral loading. This couple moment is applied on the shear wall core to fight rotation within the core, and therefore reduce the deflection of the building.

Structural Materials

Structural Steel:

W-Shapes……………………………………………..ASTM A992 or A913, Fy=50 KSI
Angles…………………………………………………ASTM A36, Fy=36 KSI
Square of Rectangular
Structural Tube…………………………..ASTM A500, Grade B, Fy=36 KSI
Steel Pipe d ≤ 12”…………………………………ASTM A53, Type E or S, Grade B, Fy=35 KSI
Material called out on
as (Fy= 65 KSI)…………………………..ASTM 913, Fy=65 KSI
All other steel……………………………………ASTM A572, A588, A441, Fy=50 KSI

Metal Decking:

3” Composite Deck……………………………Verco W3 - 20 gage minimum

Welding Electrodes:

E70 XX………………………………………………..70 KSI minimal tensile strength

Cast-in-Place Concrete:

Misc. Concrete, Curbs,
Sidewalks……………………………………..f’c = 4,000 psi – Normal Weight
Slab on Grade……………...………………………f’c = 4,000 psi – Normal Weight
Foundation Walls…………………………………..f’c = 5,000 psi – Normal Weight
Concrete on Steel Deck…………………………..f’c = 4,000 psi – Normal Weight
f’c = 4,000 psi – Light Weight
Columns, Reinforced Beams,
and Slabs……………………………………..f’c = 5,000 psi – Normal Weight
Shear Walls…………………………………………..f’c = 6,000 psi – Normal Weight
f’c = 8,000 psi – Normal Weight
f’c = 10,000 psi – Normal Weight
Grade Beams, Elevator Pits,
Caissons, Caps………………………………..f’c = 8,000 psi – Normal Weight

Reinforcement:

Reinforcing Bars……………………………………….ASTM A615, Grade 60
Welded Wire Fabric…………………………………..ASTM A185

Masonry:

Hollow Concrete Units………………………………ASTM C90, $f'c_{min}$ = 1,900 psi

Codes and References

Design Codes:

National Model Code:

Chicago Building Code 2005

Design Codes:

American Concrete Institute (ACI), ACI 530-92, Requirements for Masonry Structures

ACI 318-83, Requirements for Structural Concrete

American Institute of Steel Construction (AISC), LRFD-86," Load and Resistance Factor Design Specification for Steel Buildings"

AISC-2000, "Specification for Structural Joints using ASTM A325 or A490 Bolts"

American Welding Society (AWS), AWS D1.1-2000, "Structural Welding Code- Steel"

AWS D1.3-98, "Structural Welding Code- Sheet Steel"

AWS D1.4-98, "Structural Welding Code-Reinforcing Steel"

AWS A2.4-98, "Symbols for Welding and Nondestructive testing"

American Iron and Steel Institute (AISI), "Specifications for the Design of Cold Formed Steel Structural Members," 1996 with supplement No.1 July 30, 1999

Structural Standards:

American National Standards Institute (ANSI), ANSI A58.1-1982

Thesis Codes:

National Model Code:

2006 International Building Code

Design Codes:

Steel Construction Manual 13^{th} edition, AISC

ACI 318-05, Building Code Requirements for Structural Concrete

Structural Standards:

American Society of Civil Engineers (ASCE), ASCE 7-05, Minimum Design Loads for Buildings and other Structures

Design Loads

Floor Live Loads			
Load Description	Load Location	Design Load (psf)	ASCE 7-05 Load
Parking	Lower Levels 2-4	50	40
Storage	LL 3,2,1 Level 4,5 Roof	125*	--
Plaza-General	LL 1, Level 1	100*	--
Lobby	Level 1,9-40, 43-58	100*	100
Office	Levels 9-40, 43-57	50 20 - Partitions	50
Tenant Filing	Levels 9-40	200*	Designed per anticipatory occupancy
Office-Increased Live Load	Levels 43-57	100 20 - Partitions	50
Com Ed	LL 2, Levels 2-58	150*	--
Conference	Levels 6 & 7	100*	--
Data Center	Level 4	200*	--
Central Plant	Lower Level 4	50	--
Mechanical	LL 1-4, Levels 1-58, Roof	125	125
Amenity		100*	--
Green Roof	Roof	40	100
UPS/ Battery	Level 4	350*	--
Terrace	Level 6	100*	100
Elevator Machine	LL 1, Levels 26,42, Roof, Penthouse	150*	300 lb (concentrated load)
Truck Dock	LL 1	250	--
Retail	LL 1, Level 1	100	100
Retail and Built up	Level 1	100	100
Roof	Level 4, Roof, Penthouse, Penthouse Roof (59-61)	40	20
Stairs	All Levels	100	100
Note - * Denotes a non-reducible live load as specified on load diagrams			

Table 1 : Floor Live Loads

Superimposed Dead Loads		
Load Description	Load Location	Design Load (psf)
Parking	Lower Levels 2-4	5 - Mech/ Elec
Storage	LL 3,2,1 Level 4,5 Roof	5 - Mech/ Elec
Plaza-General	LL 1, Level 1	5 - Mech/ Elec 75 - Topping
Lobby	Level 1,9-40, 43-58	15 - Mech/Elec/Ceiling
Office	Levels 9-40, 43-57	15 - Mech/Elec/Ceiling
Tenant Filing	Levels 9-40	15 - Mech/Elec/Ceiling
Office-Increased Live Load	Levels 43-57	15 - Mech/Elec/Ceiling
Com Ed	LL 2, Levels 2-58	5 - Mech/ Elec
Conference	Levels 6 & 7	15 - Mech/Elec/Ceiling 40 - Floor Finish
Data Center	Level 4	15 - Mech/Elec/Ceiling
Central Plant	Lower Level 4	Weight of Equipment
Mechanical	LL 1-4, Levels 1-58, Roof	30 - Mech/Elec
Amenity		20 - Mech/Elec
Green Roof	Roof	40 - Green Roof/ Roofing 10 - Mech/ Elec
UPS/ Battery	Level 4	15 - Mech/Elec
Terrace	Level 6	60 - 5" Topping slab 40 - Pavers
Elevator Machine	LL 1, Levels 26,42, Roof, Penthouse	30 - Mech/Elec
Truck Dock	LL 1	15 - Mech/Elec
Retail	LL 1, Level 1	20 - Mech/Elec
Retail and Built up	Level 1	60 - Built up slab 20 - Mech/Elec
Roof	Level 4, Roof, Penthouse, Penthouse Roof (59-61)	10 - Mech/Elec 15 - Roofing
Curtain Wall	All Levels	15 –vertical surface

Table 2 : Superimposed Dead Loads

ETABS Model

ETABS computer modeling software was used to model and analyze 300 North La Salle's existing lateral system. Some important assumptions were made during the modeling process to simplify the model, while still providing accurate results. The gravity systems for each floor were neglected. Each floor was modeled as a rigid diaphragm carrying all of the lateral loads directly to the lateral systems. The weight of the existing gravity systems were accounted for when determining the building's weight for seismic loads. The self weight previously calculated for each story in Tech 1 and attached in Appendix D were applied as area mass loads on each floor diaphragm.

The shear walls were meshed with a maximum dimension of 48" x 48" in order to allow the core walls to act as a rigid unit. The columns supporting the three outrigger and two belt trusses were included in the model. They work in unison with the trusses when wind load is applied to limit rotation of the building's core as seen in Figure 3. The columns above the trusses were not modeled and can be neglected because their capacity to resist the shear is negligible compared to the shear wall core.

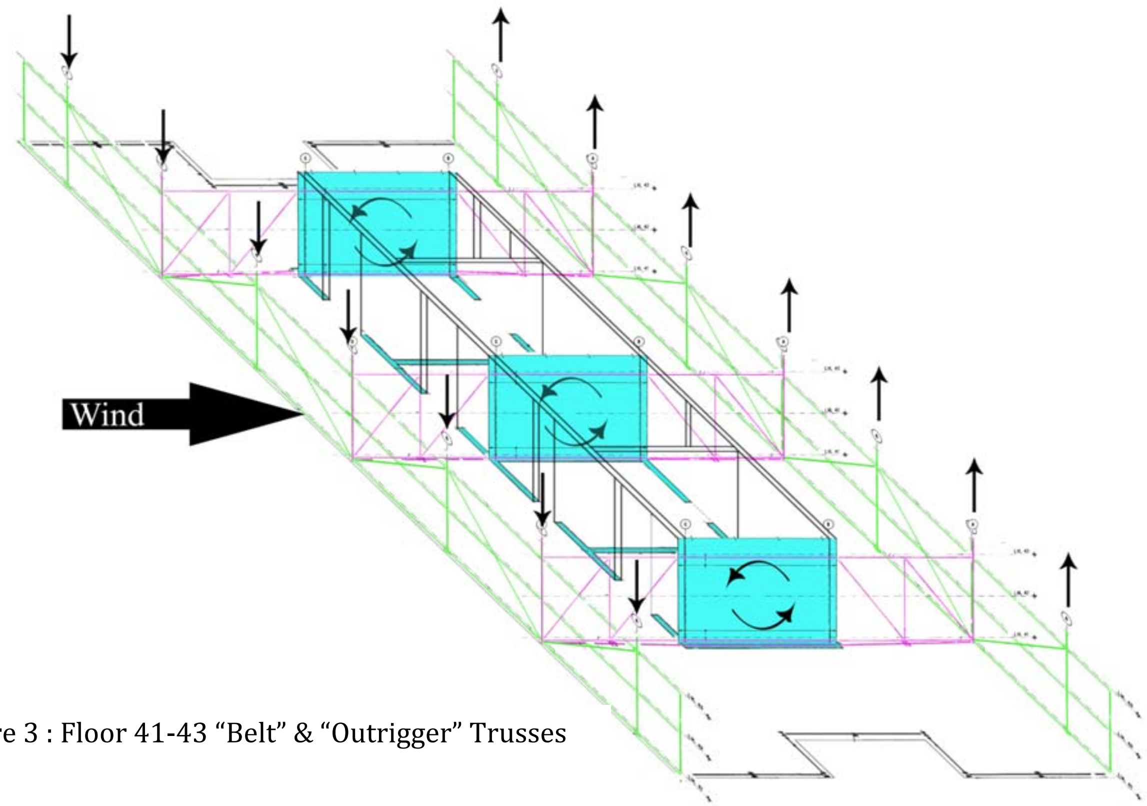

Figure 3 : Floor 41-43 "Belt" & "Outrigger" Trusses

Wind loads were applied at the center of pressure of each level, and seismic loads were calculated by ETABS and applied at each levels center of mass. The seismic loads calculated by ETABS were verified to be within one percent of the previous hand calculated loads.

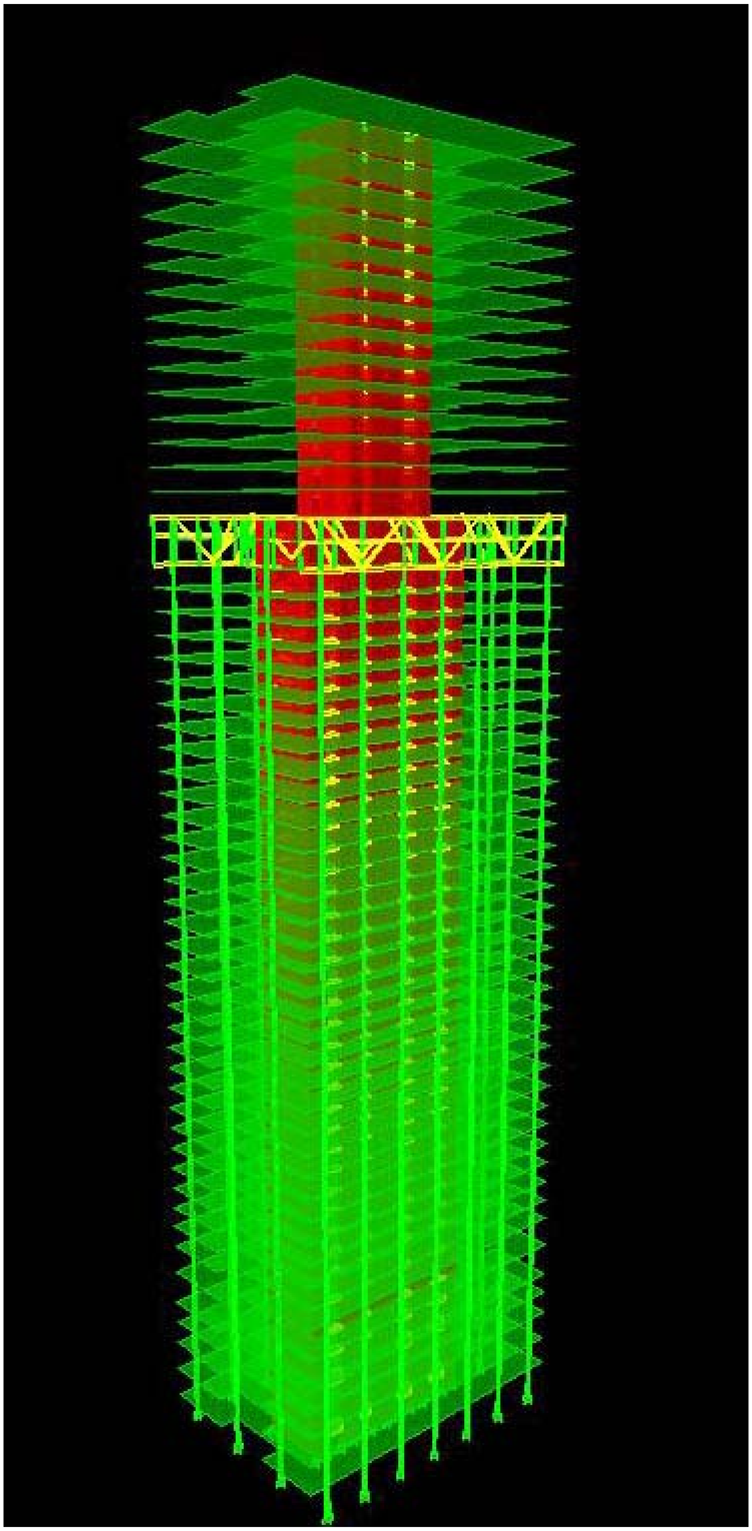

Figure 5: 3D ETABS Model

Load Combinations

Various load cases are specified by ASCE 7-05 Section 2.3 for factored loads using strength design.

1.4 (D+F)

1.2 (D+F+T)+1.6(L+H) +0.5(Lr or S or R)

1.2D + 1.6(Lr or S or R) + (L or 0.8W)

1.2D + 1.6W + L + 0.5(Lr or S or R)

1.2D + 1.0E + L + 0.2S

0.9D + 1.6W + 1.6H

0.9D + 1.0E + 1.6H

As stated previously the gravity loads were neglected during the modeling of the structure in ETABS. Therefore the comparison was made between 1.6W and 1.0E for both the north-south and east-west directions. However, as per ASCE 7-05 -Figure 6-9: Design Wind Load Cases, Cases 1,2, and 3 were analyzed within the 1.6W strength comparison. It was determined from base shear and overturning moment values that Case 1 wind controls both the north-south and east-west directions. This confirms the hand calculations performed. The conclusion that wind controls is reasonable considering that 300 North La Salle is a tall flexible building exposed to large wind loads that is also located in a relatively low seismic activity area. This confirms the hand calculations performed. Due to the complexity of the building further investigation will need to be completed to examine the gravity loads affect on the various load cases that include wind.

Load	Base Shear X (k)	Base Shear Y (k)	Overturning Moment (k-ft)	Overturning Moment (k-ft)	Torsional Moment (k-ft)
WINDEW	**-6757.19**	**0**	**0**	**-2759426**	**690078**
WINDNS	**0**	**-10268.1**	**4194284**	**0**	**-1149172**
WINDC2EW	-5067.89	0	0	-2069570	509117
WINDC2NS	0	-7701.1	3145667	0	-880801
WINDC3	-5067.89	-7701.1	3145667	-2069570	-344323
WINDC2EW2	-5067.89	0	0	-2069570	525999
WINDC2NS2	0	-7701.1	3145667	0	-842962
SEISMICEW	-2002.39	0	0	-1125819	204494
SEISMICNS	0	-2002.39	1125819	0	-224101

Load Path and Distribution

As the lateral loads contact the building's façade they are carried through the floor diaphragms into the shear wall core. The distribution of these forces into the various shear walls that the core consists of is determined by the concept of relative stiffness. The stiffer a lateral load resisting element is, compared to the other elements it is working simultaneously with, the more load it will carry.

The relative stiffness of each level's shear walls in 300 North La Salle can be simplified to the ratio between each walls moment of inertia and the total moment of inertia for all the walls. This derivation can be seen in Appendix F.

In order to confirm the ETABS model, the relative stiffness of each wall was calculated by hand for the levels up to Level 40 before the step back in the core. A 1000 kip load was then applied to the model on Level 40 and the relative stiffness of each wall in ETABS was calculated based on the portion of the 1000 kip load the wall carried. At this stage the outriggers were neglected to ensure accuracy of the check. The hand calculations verified the ETABS model within 5% error as seen below.

Relative Stiffness - No Modifiers			Relative Stiffness - ETABS w/o truss		
Wall #	Lvl 1-7	Lvl 9-40	Wall #	Lvl 1-7	Lvl 9-40
Wall 3	16%	17%	Wall 3	17%	16%
Wall 4	23%	23%	Wall 4	22%	23%
Wall 5	22%	22%	Wall 5	20%	21%
Wall 6	24%	23%	Wall 6	23%	24%
Wall 7	16%	16%	Wall 7	18%	16%

The relative stiffness values were then used to determine the center of rigidity on a typical floor between levels 9 and 40. The ETABS model provided a center of rigidity with a maximum percent error under 1% as can be verified in Appendix F. The near symmetry of the shear wall core about both axes, as well as its close proximity to the center of mass results in a very small eccentricity between the center of rigidity and the center of mass. This technical assignment will use the values provided by the ETABS model for both the center of mass and center of rigidity. This is assumed to be acceptable as select floor values were confirmed by hand calculations, and the ETABS values will be more accurate for levels which experience irregularities in shear wall locations.

Relative Stiffness - North/South Direction					
Story	**Wall 3**	**Wall 4**	**Wall 5**	**Wall 6**	**Wall 7**
L57	N/A	22%	52%	26%	N/A
L56	N/A	24%	52%	24%	N/A
L55	N/A	26%	49%	25%	N/A
L54	N/A	27%	48%	25%	N/A
L53	N/A	27%	47%	26%	N/A
L52	N/A	28%	47%	26%	N/A
L51	N/A	28%	47%	26%	N/A
L50	N/A	28%	47%	26%	N/A
L49	N/A	28%	46%	25%	N/A
L48	N/A	28%	46%	25%	N/A
L47	N/A	28%	47%	25%	N/A
L46	N/A	28%	46%	25%	N/A
L45	N/A	28%	46%	26%	N/A
L44	N/A	28%	46%	26%	N/A
L43	N/A	27%	46%	28%	N/A
L42	-57%	-175%	-166%	-175%	-52%
L41	-57%	-175%	-166%	-175%	-52%
L40	68%	-14%	-11%	-15%	70%
L39	46%	3%	4%	1%	47%
L38	33%	11%	12%	10%	33%
L37	26%	17%	17%	16%	25%
L36	21%	20%	20%	19%	20%
L35	19%	21%	22%	21%	18%
L34	17%	22%	23%	22%	16%
L33	17%	23%	23%	22%	16%
L32	16%	23%	23%	22%	15%
L31	16%	23%	24%	22%	15%
L30	16%	23%	24%	22%	15%
L29	16%	23%	24%	22%	15%
L28	16%	23%	24%	22%	15%
L27	16%	23%	24%	22%	15%
L26	16%	23%	24%	23%	15%
L25	16%	23%	24%	23%	15%
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L22	16%	23%	24%	23%	15%
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L20	16%	23%	24%	22%	15%
L19	16%	23%	24%	22%	15%
L18	16%	23%	24%	22%	15%
L17	16%	23%	24%	22%	15%
L16	16%	23%	24%	22%	15%
L15	16%	23%	24%	22%	15%
L14	16%	23%	24%	22%	15%
L13	16%	23%	25%	22%	15%
L12	15%	23%	25%	22%	14%
L11	15%	23%	26%	22%	14%
L10	15%	22%	27%	22%	14%
L9	15%	22%	28%	22%	13%
L7	15%	22%	29%	22%	13%
L6	15%	22%	28%	22%	14%
L5	16%	21%	26%	22%	15%
L4	17%	21%	25%	22%	15%
L2	18%	21%	24%	21%	16%
L1	19%	21%	23%	20%	18%
LL1	20%	20%	22%	19%	19%

Relative Stiffness- East/West Direction		
Story	**Wall B**	**Wall C**
L57	57%	43%
L56	53%	47%
L55	52%	48%
L54	51%	49%
L53	50%	50%
L52	50%	50%
L51	50%	50%
L50	50%	50%
L49	49%	51%
L48	49%	51%
L47	49%	51%
L46	49%	51%
L45	49%	51%
L44	50%	50%
L43	51%	49%
L42	-41%	-48%
L41	45%	53%
L40	51%	48%
L39	49%	52%
L38	48%	51%
L37	49%	51%
L36	49%	51%
L35	49%	51%
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L15	47%	53%
L14	47%	53%
L13	46%	54%
L12	46%	54%
L11	45%	55%
L10	45%	54%
L9	52%	48%
L7	61%	39%
L6	52%	48%
L5	47%	53%
L4	64%	36%
L2	105%	-5%
L1	63%	36%
LL1	48%	52%

Observations on Distribution of Lateral Loading:

The chart of relative stiffness has a noticeable irregularity around levels 41 and 42. The sudden negative values occur at the location of the outrigger and belt trusses. This is due to a shear reversal within the core. The three outrigger trusses spanning north-south engage the two exterior belt trusses running east-west, the belt trusses then engage the columns axially. The addition of rigidity from these members will restrict the movement of the rigid diaphragms on levels 40 and 42. The rigid diaphragms then theoretically create an internal hinge in the shear walls about which the internal shear will reverse directions. The theory is illustrated diagrammatically in Figure 7. This shear reversal can also be observed in the image captured from ETABS in Figure 6, illustrating the shear reversal in wall 3.

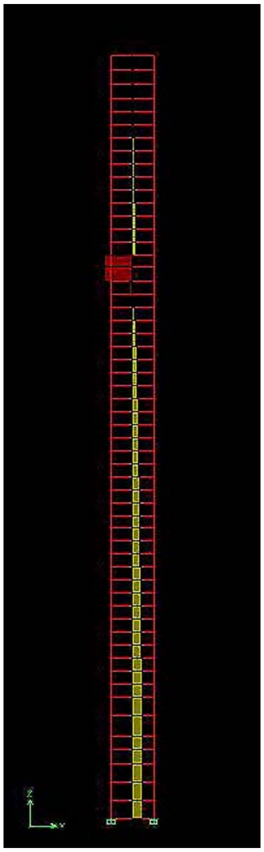


Figure 6: Shear Reversal at Level 41-43

Figure 7: Shear Reversal Diagram

The shear reversal at the outrigger and belt truss levels is fairly high and will be investigated in further detail during future phases. More examination will be done in variations of modeling the diaphragms rigidity.

Torsion

Torsion is produced when the center of rigidity is not located at the center of mass. Seismic forces are applied at the center of mass, when these forces are applied at an eccentricity from the center of rigidity they create moments. Torsional shear then also must be accounted for and will be discussed further in the shear section of this report.

When evaluating a structure based upon the assumption that its diaphragms are rigid, or not flexible, two different types of torsion must be evaluated by ASCE 7-05 – Section 12.8.4. These two types of torsion are inherent torsion and accidental torsion. The inherent torsion is present from the eccentricity between the center of mass and the center of rigidity. The accidental torsion is produced by an assumed displacement either way of the center of mass from its original location by 5% of the dimension of the structure perpendicular to the applied forces.

Overall Building Torsion						
	North-South			East-West		
Story	M_t	M_{ta}	$M_{t,tot}$	M_t	M_{ta}	$M_{t, tot}$
L57	36.12	476.99	513.11	-62.46	397.70	335.24
L56	38.39	535.29	573.68	-68.09	446.32	378.23
L55	35.89	525.58	561.47	-65.25	438.22	372.97
L54	33.55	515.32	548.88	-62.72	429.67	366.95
L53	29.12	466.23	495.35	-55.90	388.74	332.83
L52	27.42	457.76	485.18	-54.19	381.67	327.48
L51	25.89	450.42	476.31	-52.72	375.55	322.83
L50	24.38	441.92	466.30	-51.21	368.47	317.26
L49	23.06	435.13	458.19	-50.00	362.80	312.80
L48	21.78	426.60	448.38	-48.70	355.69	307.00
L47	20.76	420.29	441.05	-47.79	350.43	302.65
L46	19.81	411.72	431.53	-46.76	343.29	296.52
L45	19.12	404.59	423.72	-46.05	337.34	291.29
L44	18.59	395.99	414.58	-45.27	330.17	284.90
L43	18.36	388.71	407.07	-44.67	324.10	279.43
L42	54.37	1100.54	1154.91	-127.43	917.61	790.18
L40	57.10	1117.18	1174.28	-129.37	931.49	802.12
L39	28.58	551.08	579.66	-61.10	459.48	398.38
L38	28.52	542.31	570.84	-56.96	452.17	395.21
L37	25.96	486.01	511.97	-47.84	405.23	357.39
L36	25.69	473.53	499.22	-43.10	394.82	351.72
L35	25.45	461.91	487.36	-38.25	385.14	346.88
L34	25.13	449.41	474.54	-33.16	374.71	341.55
L33	24.87	438.31	463.18	-28.02	365.45	337.43
L32	24.51	425.76	450.28	-22.64	355.00	332.35
L31	24.13	412.97	437.10	-17.12	344.33	327.21
L30	23.74	400.44	424.18	-11.48	333.88	322.40
L29	23.42	389.25	412.67	-5.73	324.55	318.82
L28	23.01	376.67	399.68	0.22	314.06	314.29
L27	22.65	365.17	387.82	6.58	304.47	311.06
L26	22.22	352.55	374.78	12.40	293.95	306.35
L25	21.85	340.77	362.62	18.75	284.13	302.88
L24	21.10	323.30	344.40	25.16	269.57	294.73
L23	20.84	313.34	334.18	32.44	261.26	293.71
L22	20.40	300.78	321.19	39.83	250.79	290.62
L21	19.96	288.13	308.09	47.53	240.24	287.77
L20	19.52	275.58	295.10	55.58	229.77	285.36
L19	19.13	263.77	282.90	64.18	219.92	284.10
L18	18.69	251.17	269.87	73.00	209.43	282.42
L17	18.88	246.79	265.67	85.05	205.77	290.82
L16	18.41	233.77	252.18	95.01	194.91	289.93
L15	18.00	221.36	239.35	105.67	184.56	290.24
L14	17.88	212.62	230.50	118.83	177.28	296.11
L13	17.39	199.52	216.91	130.13	166.36	296.49
L12	16.84	186.17	203.02	141.23	155.23	296.46
L11	16.32	173.71	190.03	152.56	144.83	297.39
L10	15.31	157.04	172.35	158.65	130.94	289.59
L9	14.56	144.13	158.68	166.20	120.17	286.37
L7	14.29	137.42	151.71	176.27	114.58	290.85
L6	18.60	178.10	196.70	256.49	148.49	404.98
L5	16.08	154.90	170.98	261.75	129.15	390.90
L4	4.57	44.40	48.97	82.30	37.02	119.32
L2	20.00	200.38	220.38	361.74	167.08	528.82
L1	4.69	53.79	58.48	29.75	44.85	74.60
LL1	4.72	63.68	68.39	0.62	53.09	53.72
Torsion Moment Total (ft-k)			21309.93			17999.89

Shear

The shear in each wall is a combination of two types of shear, direct shear and torsional shear. Direct shear is the shear induced into the building and distributed to the shear walls. The direct shear in each wall is distributed by its relative stiffness and is simply the product of the story shear and the relative stiffness. Torsional shear is present from torsion within the building, and is defined as the product of story shear, eccentricity, relative stiffness, and distance of the wall from the center of rigidity divided by the torsional moment of inertia. The equation used to determine torsional shear:

$$T = \frac{V_{tot} e d_i R_i}{J}$$

When torsional shear was calculated for a typical level it can be to have a negligible effect on the total shear in each wall, less than a 2% increase or decrease in shear at the farthest walls. The ETABS output for shear will therefore be used when spot checking the strength of the shear walls.

Effect of Torsional Shear on Total Shear N-S Direction							
Level 38	**Wall 3**	**Wall 4**	**Wall 5**	**Wall 6**	**Wall 7**		
V	3886.146	3886.146	3886.146	3886.146	3886.146		
di	57.91667	28.33333	-0.16667	-30.75	-58.75		
e	0.524667	0.524667	0.524667	0.524667	0.524667		
Ri	0.33	0.11	0.12	0.10	0.33		Total J
J	1115.272	91.92142	0.003259	99.16841	1135.154		2441.519
Torsional Shear	16.08125	2.709328	-0.01633	-2.69321	-16.1358		
Direct Shear	1292.09	444.98	455.96	407.57	1278.08		
Total Shear	1308.171	447.6893	455.9437	404.8768	1261.944		
% Difference b/w D.S. and Total Shear	1%	1%	0%	-1%	-1%		

Shear Strength Check

Shear Walls

According to ACI 318-08 Section 21.9.4.1 the shear strength of a reinforced concrete shear wall is defined as:

$$V_n = A_{cv}\left[\left(\alpha_c \lambda \sqrt{f'_c}\right) + (\rho_t f_y)\right]$$

The hand calculations can for the walls supporting Floor 18 and carrying the load in the north-south direction can be found below. All of the walls except for wall 5 were well within the allowable capacity. While wall 5 was still within the allowable capacity it was very close, and may need further examination in the future.

Shear Wall Strength Check (Supporting Floor 18)										
	V_u (k)	Vertical Reinf.	Spacing (in)	Length (in)	Thickness (in)	A_{cv} (in^2)	α_c	ρ_t	f'c	ΦV_n(k)
Wall 3	1189.88	(1) #6	12	513	22	11286	2	0.001667	8,000	2360.626
Wall 4	1746.62	(1) #5	12	513	18	9234	2	0.001435	8,000	1835.234
Wall 5	1832.14	(1) #5	12	513	18	9234	2	0.001435	8,000	1835.234
Wall 6	1706.65	(1) #5	12	513	18	9234	2	0.001435	8,000	1835.234
Wall 7	1116.95	(1) #6	12	513	22	11286	2	0.001667	8,000	2360.626

Drift and Displacement

Seismic drift, the displacement of a floor under seismic load, is controlled as a strength requirement by ASCE 7-05 – Table 12.12.1. The table limits story drift, the difference in displacement between a chosen floor and the displacement of the floor below it, to $0.20h_{sx}$, where h_{sx} is the height of the story below a chosen level x. This limit is set based upon 300 North La Salle's occupancy category of II and that it does not fit into any of the other specified building types.

The story drifts were calculated from ETABS results for diaphragm displacements at each level. The drifts in both the north-south and east-west directions were acceptable under the code limitation. These drifts can be found in Appendix D.

In addition to strength checks on the building's lateral system, there are also some basic serviceability guidelines. The serviceability concern for lateral loads is the drift of the building under wind loads.

The drift limitations on wind load for 300 North La Salle was assumed to be H/400 where H is any height above ground level. This value was taken from the *Structural Engineering Guidebook* (1968) by Gaylord and Gaylord.

The wind drifts for each story were taken from ETABS and compared to this limitation as seen in Appendix C. When modeling in ETABS some additional assumptions were made in order to get the find drift under service loads. The shear walls were cracked to 0.7Ig based on ACI 318-08 – Section 8.8 (Effective stiffness to determine lateral deflections). Also as permitted only 70% of the wind loads were applied to each story. The drift at each story height was then under the set limit. At the roof level the drift was 18.12" when wind was applied in the north-south direction and 6.5" when wind was applied in the east-west direction, these are both below the H/400 value of 23.16", and were the closest the drift came to the limit.

To hand calculate the drift based on story drifts and shear wall displacements is beyond the scope of this report due to the outrigger trusses restricting the shear wall from acting like a simple cantilever, and therefore no values can be directly compared to the ETABS model.

Overturning

Overturning moments are moments created by the lateral forces acting at each story level some height above the foundation. These moments are transformed into axial loads transmitted through the lateral members and into the foundation. Depending on the location of the lateral members these axial loads could be in tension or compression. The tension stresses act to pull the foundation up fighting gravity, while the compression forces are additive to gravity adding stresses pushing down on the foundation. The magnitude of these forces can have large effects on the design values for the foundations.

A rough estimate was performed to compare the effects of the self-weight of the building versus the effects of the overturning moment on the foundation. By assuming that all of the overturning moment was carried by the 10 primary drilled piers below the shear wall core, and that these piers also carried all of the weight of the building, a very rough comparison could be made. The idea to apply all of both loadings to these 10 piers was based upon the reasoning that the shear walls are going to carry a large percent of the lateral force while simultaneously carrying a more even percentage of the gravity load, compared to the exterior columns, to the foundation. If overturning is an issue at the full load for both weight and overturning moment, it will remain an issue in the actual design.

It was found that when applying the full overturning moment and weight each pier will experience nearly the same force from overturning as it will from the self weight. This leads to the conclusion that the overturning moment will be a factor in the design of the deep foundations. Further investigation will need to be performed to check the piers under the combined loads defined in ASCE 7-05.

North - South

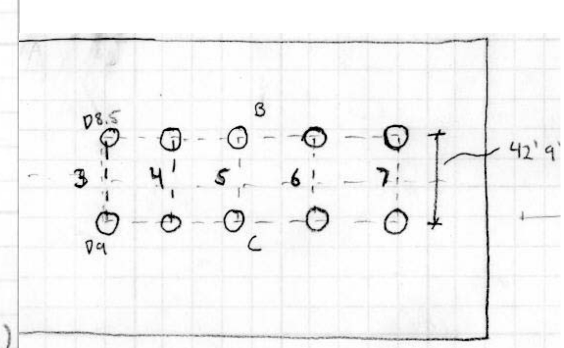
Overturning Moment : Resisted by couple moment created by piers

$$OM = \frac{4{,}194{,}284'^{k}}{5 \text{ Walls}} = \frac{838{,}857'^{k}}{42'\text{-}9''} = 19{,}622^{k}/\text{drilled pier}$$

Compare to Self-weight of building:

If all weight is distributed to shear walls (under conservative)

$$\text{Total weight} = \frac{202{,}538^{k}}{10 \text{ piers}} = 20{,}254^{k}/\text{drilled pier}$$



Conclusion

The lateral loads from Technical Report 1 were reevaluated and applied to an ETABS model of 300 North La Salle. In addition to the wind case 1 evaluated previously the ETABS model was used to evaluate wind cases 2 & 3. The evaluation of these wind cases and seismic loads allowed the determination of the controlling load combination in each direction. It was found that the combinations with 1.6 W, loaded as wind case 1, would control in both the North-South and East-West directions. This is a reasonable determination as 300 North La Salle is a tall building exposed to large wind loads, located in a relatively low seismic zone.

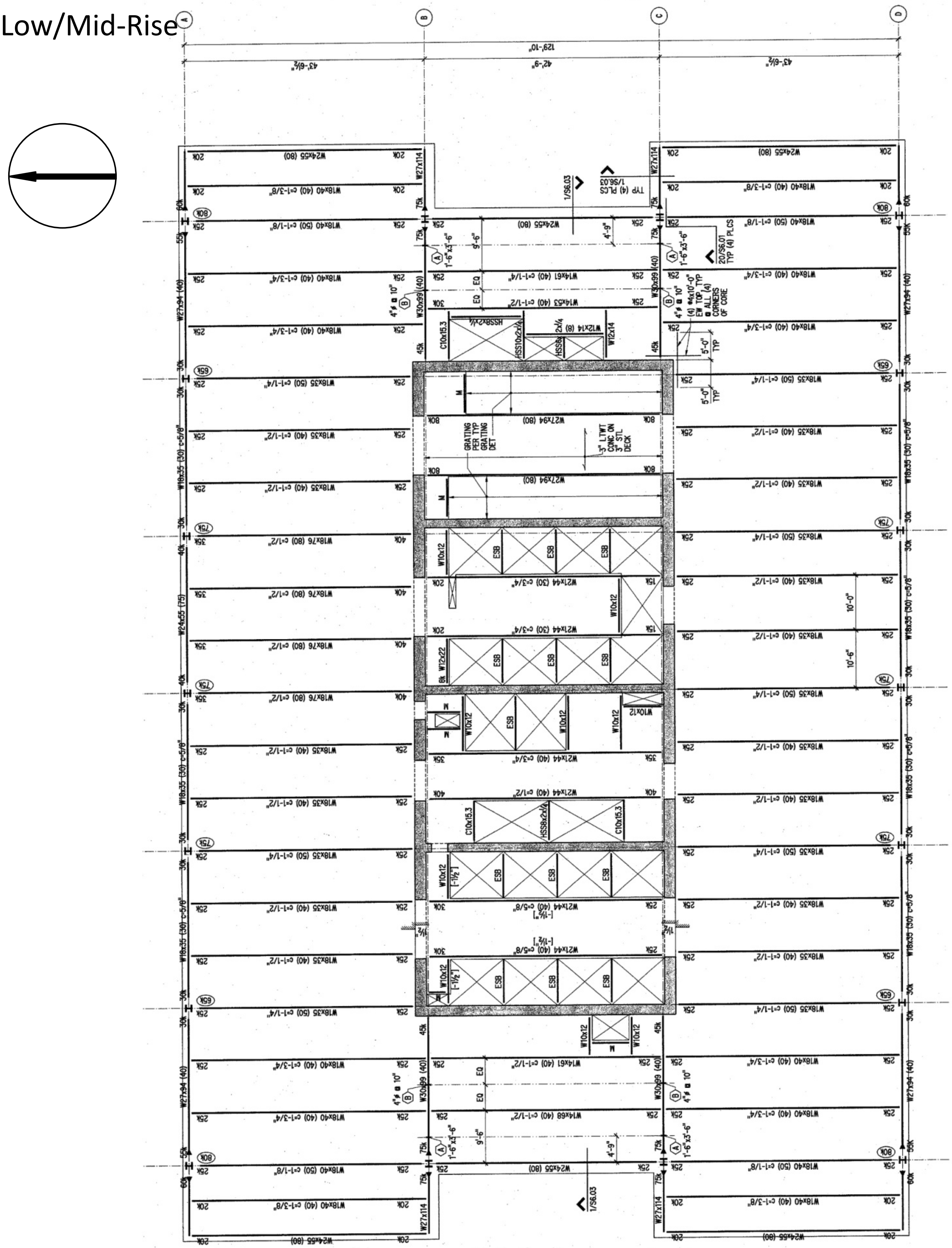
Once the accuracy of the ETABS model was confirmed through hand calculations of the relative stiffness and center of rigidity, values provided by ETABS were used in subsequent calculations. The primary reason of using the ETABS values was to provide quicker more accurate calculations in areas where there were shear wall irregularities. It was also beneficial in providing an easier observation of the effects of the outrigger and belt trusses, and the shear reversal they induced into the shear wall core.

This report confirms that the shear walls are the primary lateral resisting system. While there is torsion within the building, it was found that the shear affects of this torsion at each level were negligible in comparison to the direct shear. Strength checks based on the shear taken from ETABS confirmed that the walls could adequately carry the shear forces. However, further investigation of the shear distribution to Wall 5 spanning north-south is recommended due to its shear values close proximity to the code limit for shear. The strength check for seismic story shift also confirmed that the structure can adequately meet the demands. The serviceability recommended limit for building drift of H/400 was also met in both directions at each level. The overturning moments present from the lateral loading appear to have a significant effect on the foundation design, and will be looked at in greater detail during later stages of the project.

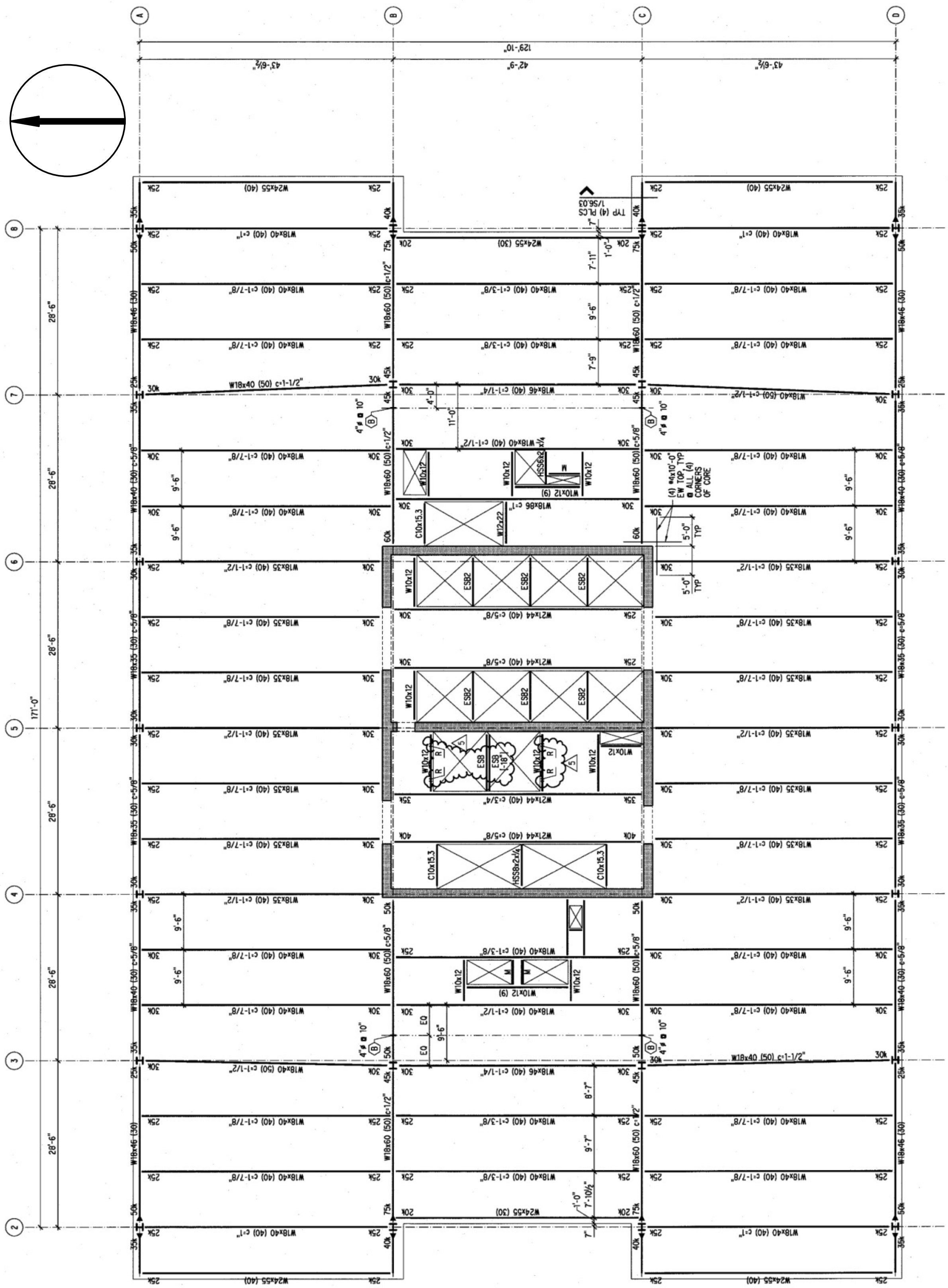
The shear walls are designed adequately to resist the determined load combinations. A more complex model will be produced in the upcoming sections of thesis and can be used to investigate the aforementioned areas of concern.

Appendix A – Typical Floor Plans

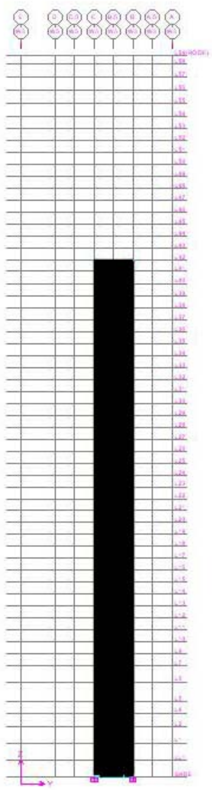
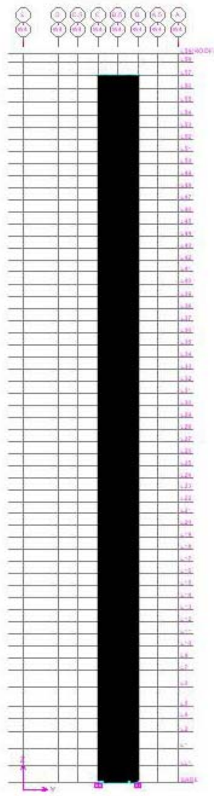
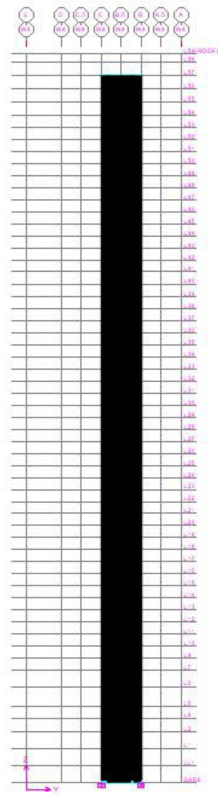
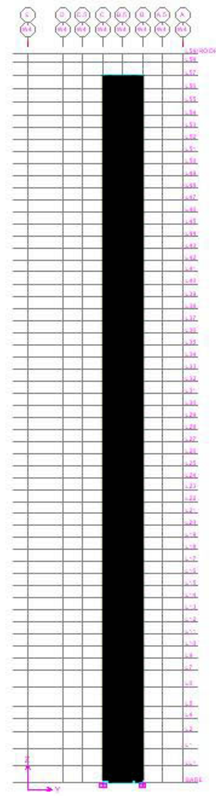
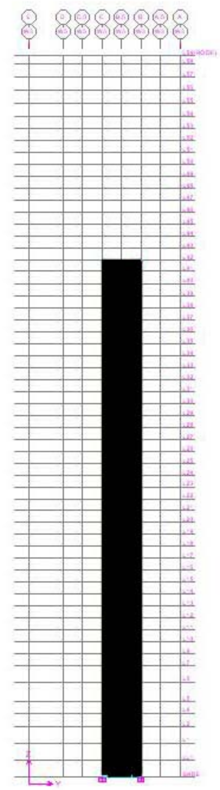
Typical Low/Mid-Rise



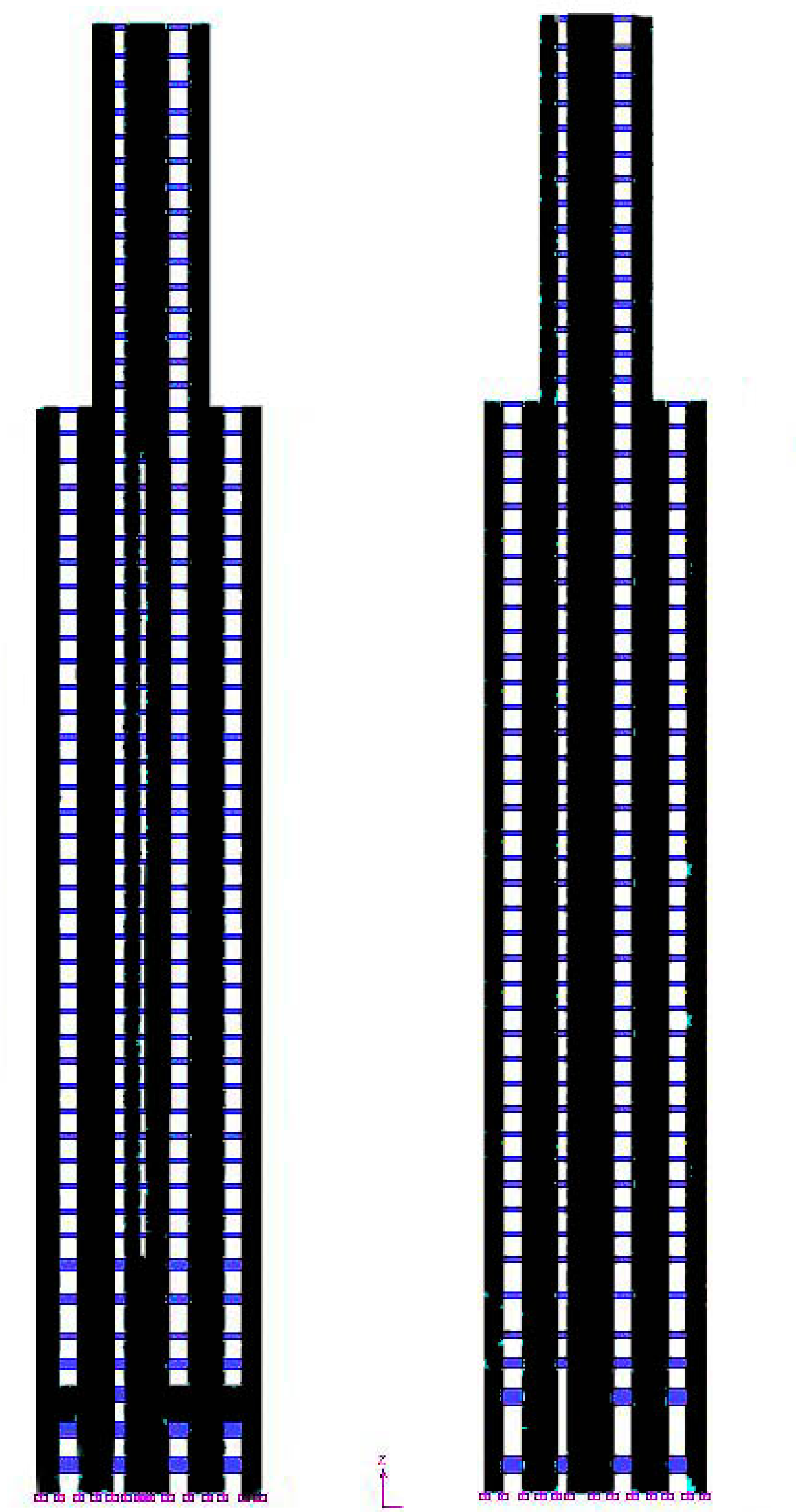
Typical High Rise



Appendix B – Shear Wall Elevations

Wall 3 **Wall 4** **Wall5** **Wall 6** **Wall 7**



Wall B Wall C

Appendix C – Wind Forces & Drift

East / West Wind Forces			
Story Level	Story Height (ft)	Story Force (Kips)	Moment (k-ft)
Roof	786.00	148.1	116373
58	772.00	87.7	67735
57	757.50	89.3	67629
56	743.00	88.5	65746
55	728.50	83.9	61119
54	715.50	79.3	56754
53	702.50	79.1	55558
52	689.50	78.6	54167
51	676.50	78.6	53146
50	663.50	78.6	52125
49	650.50	78.2	50862
48	637.50	77.8	49570
47	624.50	77.8	48559
46	611.50	77.8	47548
45	598.50	77.2	46223
44	585.50	76.9	45027
43	572.50	73.9	42334
42	560.50	76.9	43105
41	546.50	82.5	45077
40	532.33	79.3	42209
39	519.33	75.9	39409
38	506.33	75.9	38417
37	493.33	75.0	37021
36	480.33	75.0	36045
35	467.33	75.0	35070
34	454.33	74.9	34014
33	441.33	74.0	32654
32	428.33	74.0	31692
31	415.33	74.0	30730
30	402.33	73.6	29605
29	389.33	72.7	28314
28	376.33	72.7	27369
27	363.33	72.7	26423
26	350.33	72.1	25268
25	337.33	71.5	24106
24	324.33	71.5	23177
23	311.33	71.5	22248
22	298.33	70.5	21043
21	285.33	70.0	19970
20	272.33	70.0	19060
19	259.33	70.0	18150
18	246.33	68.7	16915
17	233.33	68.3	15937
16	220.33	68.3	15049
15	207.33	68.3	14161
14	194.33	67.7	13158
13	181.33	67.3	12210
12	168.33	66.8	11249
11	155.33	66.1	10268
10	142.33	65.6	9344
9	129.33	77.4	10013
7	111.33	95.8	10671
6	90.33	82.4	7448
5	77.33	66.8	5163
4	62.33	76.1	4746
2	44.67	82.3	3677
1	26.00	80.9	2104
LL-1	7.5	70.9	532
Total Base Shear (kips)=	4442.2		
Total Overturning Moment (k-ft)=	1873297		

North / South Wind Forces			
Story Level	Story Height (ft)	Shear Force (Kips)	Moment (k-ft)
Roof	786.00	222.7	175023
58	772.00	133.4	102992
57	757.50	135.7	102830
56	743.00	134.5	99962
55	728.50	127.6	92927
54	715.50	120.6	86291
53	702.50	120.2	84472
52	689.50	119.4	82354
51	676.50	119.4	80801
50	663.50	119.4	79249
49	650.50	118.9	77327
48	637.50	118.2	75360
47	624.50	118.2	73824
46	611.50	118.2	72287
45	598.50	117.4	70270
44	585.50	116.9	68450
43	572.50	112.4	64356
42	560.50	116.9	65528
41	546.50	125.4	68522
40	532.33	120.5	64161
39	519.33	115.4	59906
38	506.33	115.3	58398
37	493.33	114.1	56272
36	480.33	114.1	54789
35	467.33	114.1	53306
34	454.33	113.8	51701
33	441.33	112.5	49630
32	428.33	112.5	48168
31	415.33	112.5	46706
30	402.33	111.8	44995
29	389.33	110.5	43031
28	376.33	110.5	41594
27	363.33	110.5	40157
26	350.33	109.6	38399
25	337.33	108.6	36632
24	324.33	108.6	35220
23	311.33	108.6	33809
22	298.33	107.2	31975
21	285.33	106.3	30342
20	272.33	106.3	28960
19	259.33	106.3	27577
18	246.33	104.3	25699
17	233.33	103.8	24212
16	220.33	103.8	22863
15	207.33	103.8	21514
14	194.33	102.9	19989
13	181.33	102.3	18548
12	168.33	101.5	17088
11	155.33	100.4	15596
10	142.33	99.7	14192
9	129.33	117.6	15207
7	111.33	145.5	16205
6	90.33	125.2	11309
5	77.33	101.4	7839
4	62.33	115.6	7204
2	44.67	125.0	5581
1	26.00	122.8	3193
LL-1	7.5	107.6	807
Total Base Shear (kips)=	6748.2		
Total Overturning Moment (k-ft)=	2845601		

Wind Drift vs. Recommended Drift for Serviceability							
		Wind E-W			Wind N-S		
Story	Height (ft)	UX (in)	H/400 (in)	% Allowable	UY (in)	H/400 (in)	% Allowable
L57	772	6.5005	23.16	28%	18.1207	23.16	78%
L56	758	6.3764	22.725	28%	17.69	22.725	78%
L55	743	6.2505	22.29	28%	17.2595	22.29	77%
L54	729	6.1222	21.855	28%	16.8289	21.855	77%
L53	716	6.0051	21.465	28%	16.4429	21.465	77%
L52	703	5.8857	21.075	28%	16.0572	21.075	76%
L51	690	5.7642	20.685	28%	15.6721	20.685	76%
L50	677	5.6407	20.295	28%	15.2878	20.295	75%
L49	664	5.5153	19.905	28%	14.9047	19.905	75%
L48	651	5.3886	19.515	28%	14.5232	19.515	74%
L47	638	5.261	19.125	28%	14.1437	19.125	74%
L46	625	5.1332	18.735	27%	13.7668	18.735	73%
L45	612	5.0065	18.345	27%	13.393	18.345	73%
L44	599	4.8823	17.955	27%	13.0228	17.955	73%
L43	586	4.7633	17.565	27%	12.6566	17.565	72%
L42	573	4.6535	17.175	27%	12.2721	17.175	71%
L40	547	4.4739	16.395	27%	11.6444	16.395	71%
L39	532	4.3621	15.97	27%	11.2444	15.97	70%
L38	519	4.2522	15.58	27%	10.8846	15.58	70%
L37	506	4.137	15.19	27%	10.5205	15.19	69%
L36	493	4.0177	14.8	27%	10.1522	14.8	69%
L35	480	3.8952	14.41	27%	9.7804	14.41	68%
L34	467	3.7701	14.02	27%	9.4059	14.02	67%
L33	454	3.6428	13.63	27%	9.0291	13.63	66%
L32	441	3.5138	13.24	27%	8.6508	13.24	65%
L31	428	3.3832	12.85	26%	8.2716	12.85	64%
L30	415	3.2513	12.46	26%	7.8922	12.46	63%
L29	402	3.1186	12.07	26%	7.5132	12.07	62%
L28	389	2.9853	11.68	26%	7.1355	11.68	61%
L27	376	2.8518	11.29	25%	6.7597	11.29	60%
L26	363	2.7193	10.9	25%	6.3865	10.9	59%
L25	350	2.5879	10.51	25%	6.017	10.51	57%
L24	337	2.4567	10.12	24%	5.6513	10.12	56%
L23	324	2.3256	9.73	24%	5.29	9.73	54%
L22	311	2.1946	9.34	23%	4.934	9.34	53%
L21	298	2.0637	8.95	23%	4.5838	8.95	51%
L20	285	1.9329	8.56	23%	4.2404	8.56	50%
L19	272	1.8023	8.17	22%	3.9046	8.17	48%
L18	259	1.6721	7.78	21%	3.5771	7.78	46%
L17	246	1.5425	7.39	21%	3.2587	7.39	44%
L16	233	1.4135	7	20%	2.9505	7	42%
L15	220	1.2856	6.61	19%	2.6532	6.61	40%
L14	207	1.1589	6.22	19%	2.3677	6.22	38%
L13	194	1.0341	5.83	18%	2.095	5.83	36%
L12	181	0.9115	5.44	17%	1.836	5.44	34%
L11	168	0.792	5.05	16%	1.5916	5.05	32%
L10	155	0.6768	4.66	15%	1.3629	4.66	29%
L9	142	0.5676	4.27	13%	1.1507	4.27	27%
L7	129	0.4676	3.88	12%	0.9561	3.88	25%
L6	111	0.3444	3.34	10%	0.7153	3.34	21%
L5	90	0.2185	2.71	8%	0.4743	2.71	18%
L4	77	0.1571	2.32	7%	0.3481	2.32	15%
L2	62	0.1036	1.87	6%	0.2255	1.87	12%
L1	45	0.0669	1.34	5%	0.1135	1.34	8%
LL1	26	0.0241	0.78	3%	0.0355	0.78	5%

Appendix D – Seismic Forces & Drift

Seismic Calculations						
Level	Height (ft)	Wx (k)	wihi^k	Fx (k)	Vx (k)	Moment (k-ft)
Parapet	796	0	0	0	0	0
Roof	786	2877	6,506,974,248	46	46	36457
58	772	2888	6,439,008,796	46	92	35434
57	758	3089	7,226,124,237	52	144	39018
56	743	3090	7,094,991,612	51	194	37577
55	729	3090	6,956,529,461	50	244	36124
54	716	2966	6,293,843,914	45	289	32100
53	703	2966	6,179,490,356	44	333	30944
52	690	2970	6,080,352,743	43	376	29884
51	677	2970	5,965,712,300	43	419	28768
50	664	2975	5,873,942,067	42	461	27781
49	651	2975	5,758,853,526	41	502	26703
48	638	2983	5,673,690,738	40	542	25783
47	625	2983	5,557,991,947	40	582	24742
46	612	2989	5,461,757,249	39	621	23807
45	599	2989	5,345,644,666	38	659	22806
44	586	2994	5,247,381,515	37	696	21900
43	573	4785	13,106,075,479	93	790	53484
42	561	2811	4,428,290,748	32	821	17693
41	547	5247	15,044,029,996	107	928	58605
40	532	4097	8,937,166,696	64	992	33913
39	519	4118	8,806,512,511	63	1055	32601
38	506	3933	7,832,657,098	56	1111	28270
37	493	3933	7,631,555,303	54	1165	26837
36	480	3936	7,443,131,697	53	1218	25485
35	467	3936	7,241,686,772	52	1270	24124
34	454	3942	7,060,809,918	50	1320	22867
33	441	3942	6,858,776,472	49	1369	21577
32	428	3941	6,653,055,677	47	1416	20313
31	415	3941	6,451,134,143	46	1462	19099
30	402	3947	6,269,017,110	45	1507	17979
29	389	3947	6,066,455,663	43	1550	16836
28	376	3953	5,879,740,582	42	1592	15773
27	363	3953	5,676,631,740	40	1633	14702
26	350	3957	5,485,771,012	39	1672	13699
25	337	3930	5,211,116,641	37	1709	12531
24	324	3945	5,047,147,434	36	1745	11669
23	311	3945	4,844,846,560	35	1779	10752
22	298	3944	4,641,199,444	33	1813	9870
21	285	3944	4,438,957,233	32	1844	9028
20	272	3949	4,247,669,834	30	1874	8246
19	259	3949	4,044,904,689	29	1903	7477
18	246	4011	3,963,105,952	28	1932	6959
17	233	4011	3,753,956,923	27	1958	6244
16	220	4016	3,553,820,942	25	1984	5582
15	207	4054	3,407,797,815	24	2008	5036
14	194	4056	3,197,485,928	23	2031	4429
13	181	4056	2,983,588,927	21	2052	3857
12	168	4066	2,782,631,259	20	2072	3339
11	155	4028	2,519,905,837	18	2090	2790
10	142	4031	2,312,382,249	16	2106	2346
9	129	4121	2,196,129,116	16	2122	2025
7	111	4980	2,761,599,649	20	2142	2192
6	90	5145	2,390,907,244	17	2159	1540
5	77	3118	751,589,052	5	2164	414
4	62	7049	3,097,517,890	22	2186	1376
2	45	4571	933,174,983	7	2193	297
1	26	6083	962,186,503	7	2200	178
ll-1	8	6861	352,998,782	3	2202	19
Total Base Shear (kips) =					2202	
Total Overturning Moment (ft-K) =						1061879

Seismic Story Drift vs. Code Limits							
		Seismic in E-W			Seismic N-S		
Story	Story H (ft)	UX (in)	Story Drift X (in)	Max Drift (in)	UY (in)	Story Drift Y (in)	Max Drift (in)
L57	14 1/2	8.44	0.1906	3 1/2	15.27	0.3762	3 1/2
L56	14 1/2	8.25	0.1937	3 1/2	14.89	0.376	3 1/2
L55	14 1/2	8.06	0.1976	3 1/2	14.52	0.376	3 1/2
L54	13	7.86	0.1807	3 1/8	14.14	0.3369	3 1/8
L53	13	7.68	0.1843	3 1/8	13.81	0.3363	3 1/8
L52	13	7.50	0.1875	3 1/8	13.47	0.3355	3 1/8
L51	13	7.31	0.1904	3 1/8	13.13	0.3342	3 1/8
L50	13	7.12	0.1926	3 1/8	12.80	0.3325	3 1/8
L49	13	6.93	0.1938	3 1/8	12.47	0.3302	3 1/8
L48	13	6.73	0.1938	3 1/8	12.14	0.3272	3 1/8
L47	13	6.54	0.1924	3 1/8	11.81	0.3235	3 1/8
L46	13	6.35	0.1893	3 1/8	11.49	0.3195	3 1/8
L45	13	6.16	0.1834	3 1/8	11.17	0.3146	3 1/8
L44	13	5.97	0.1734	3 1/8	10.85	0.3092	3 1/8
L43	13	5.80	0.1558	3 1/8	10.54	0.3233	3 1/8
L42	12	5.65	0.2442	2 7/8	10.22	0.5208	2 7/8
L41	14	5.40	0.1538	3 1/3	9.70	0.3342	3 1/3
L40	14 1/6	5.25	0.152	3 2/5	9.36	0.3017	3 2/5
L39	13	5.10	0.1594	3 1/8	9.06	0.3064	3 1/8
L38	13	4.94	0.1645	3 1/8	8.76	0.3109	3 1/8
L37	13	4.77	0.168	3 1/8	8.45	0.3146	3 1/8
L36	13	4.60	0.1704	3 1/8	8.13	0.3175	3 1/8
L35	13	4.43	0.1723	3 1/8	7.81	0.3198	3 1/8
L34	13	4.26	0.1734	3 1/8	7.49	0.3214	3 1/8
L33	13	4.09	0.1741	3 1/8	7.17	0.3224	3 1/8
L32	13	3.91	0.1743	3 1/8	6.85	0.3226	3 1/8
L31	13	3.74	0.174	3 1/8	6.53	0.3222	3 1/8
L30	13	3.57	0.1731	3 1/8	6.20	0.3209	3 1/8
L29	13	3.39	0.1716	3 1/8	5.88	0.3191	3 1/8
L28	13	3.22	0.1688	3 1/8	5.56	0.3164	3 1/8
L27	13	3.05	0.1658	3 1/8	5.25	0.313	3 1/8
L26	13	2.89	0.1638	3 1/8	4.94	0.3092	3 1/8
L25	13	2.72	0.162	3 1/8	4.63	0.3049	3 1/8
L24	13	2.56	0.1601	3 1/8	4.32	0.3	3 1/8
L23	13	2.40	0.1583	3 1/8	4.02	0.2942	3 1/8
L22	13	2.24	0.1562	3 1/8	3.73	0.2879	3 1/8
L21	13	2.09	0.1541	3 1/8	3.44	0.2808	3 1/8
L20	13	1.93	0.1517	3 1/8	3.16	0.273	3 1/8
L19	13	1.78	0.1492	3 1/8	2.89	0.2644	3 1/8
L18	13	1.63	0.1463	3 1/8	2.62	0.2553	3 1/8
L17	13	1.48	0.1433	3 1/8	2.37	0.2453	3 1/8
L16	13	1.34	0.1399	3 1/8	2.12	0.2346	3 1/8
L15	13	1.20	0.1361	3 1/8	1.89	0.2233	3 1/8
L14	13	1.06	0.1318	3 1/8	1.66	0.2111	3 1/8
L13	13	0.93	0.1268	3 1/8	1.45	0.1982	3 1/8
L12	13	0.81	0.1209	3 1/8	1.25	0.1847	3 1/8
L11	13	0.69	0.1135	3 1/8	1.07	0.1703	3 1/8
L10	13	0.57	0.1032	3 1/8	0.90	0.1554	3 1/8
L9	13	0.47	0.1265	3 1/8	0.74	0.1912	3 1/8
L7	18	0.34	0.1277	4 1/3	0.55	0.1897	4 1/3
L6	21	0.21	0.0617	5	0.36	0.0985	5
L5	13	0.15	0.0531	3 1/8	0.26	0.0948	3 1/8
L4	15	0.10	0.0374	3 3/5	0.17	0.0856	3 3/5
L2	17 2/3	0.06	0.0405	4 1/4	0.08	0.0584	4 1/4
L1	18 2/3	0.02	0.0216	4 1/2	0.02	0.0244	4 1/2
LL1	26		0	6 1/4		0	6 1/4

Appendix E – Wind and Seismic Calculations

Wind

Factors and Coefficients		
	North/ South	East/ West
V	90	90
Kd	0.85	0.85
I	1.00	1.00
Exposure	B	B
Kzt	1.00	1.00
Kh	1.78	1.78
α	7.00	7.00
Zg	1200.00	1200.00
z	796.00	796.00
B	199.50	133.25
L	133.25	199.50
h	786.00	786.00
g_Q	3.40	3.40
g_V	3.40	3.40
g_R	3.92	3.92
η_1	0.34	0.34
zbar	471.60	471.60
Izbar	0.19	0.19
Lzbar	776.55	776.55
Q	0.76	0.77
$V_{bar}z$	115.49	115.49
N_1	2.26	2.26
R_n	0.08	0.08
R_h	0.09	0.09
R_B	0.30	0.41
R_L	0.15	0.11
R	0.37	0.42
Gf	0.86	0.88
Windward Cp	0.80	0.80
Leeward Cp	-0.50	-0.50
Parapet Windward GC_{pn}	1.50	1.50
Parapet Leeward GC_{pn}	-1.00	-1.00

Kz and qz Calculations		
Height (ft)	Kz	qz
15	0.57	10.05
20	0.62	10.93
25	0.66	11.63
30	0.70	12.34
40	0.76	13.40
50	0.81	14.28
60	0.85	14.98
70	0.89	15.69
80	0.93	16.39
90	0.96	16.92
100	0.99	17.45
120	1.04	18.33
140	1.09	19.21
160	1.13	19.92
180	1.17	20.62
200	1.20	21.15
250	1.28	22.56
300	1.35	23.79
350	1.41	24.85
400	1.47	25.91
450	1.52	26.79
500	1.56	27.50
550	1.61	28.35
600	1.65	29.06
650	1.69	29.73
700	1.72	30.37
750	1.76	30.98
786	1.78	31.39
796	1.79	31.51
Kh = 1.78		

Seismic

Coefficients and References		
Factors	Values	Reference
Longitude/ Latitude	41^{o} 59' N / 87^{o} 54' W	
height (ft)	786.000	
S_S	0.162	USGS website
S_1	0.059	USGS website
Site Class	D	ASCE7-05 11.4.2 Site Class
$S_{MS}=F_aS_S$	0.259	ASCE7-05 Eqn 11.4-1
$S_{M1}=F_vS_1$	0.142	ASCE7-05 Eqn 11.4-2
F_a	1.600	ASCE7-05 Table 11.4-1
F_v	2.400	ASCE7-05 Table 11.4-2
$S_{DS}=(2/3)S_{MS}$	0.173	ASCE7-05 Eqn 11.4-3
$S_{D1}=(2/3)S_{M1}$	0.094	ASCE7-05 Eqn 11.4-4
Ta	2.969	ASCE7-05 Eqn 12.8.7
Ts=SD1/SDS	0.546	ASCE7-05 11.6
.8Ts	0.437	
SDC	B	
V=CsW	2202	
Cs= S_{DS}/(R/I)*	0.043	ASCE7-05 12.8
Cs= SDS/(T*R/I)*	0.0064	
R	4.000	ASCE7-05 12.2-1 B.6.
I	1.000	
T_L	12.000	ASCE7-05 Fig 22-15
T=Ta	T<TL	ASCE7-05 12.8.2
W (k)	220212	

*Note: Since lowest C_s is less tha 0.01, 0.01 was used for calculating V

Appendix F – Relative Stiffness & Center of Rigidity

$$K = \frac{P}{\Delta_P}$$

$$\frac{h}{l} > 3 \quad \therefore \quad \Delta_P = \frac{Ph^3}{3EI}$$

$$K_{wall} = \frac{P}{\frac{Ph^3}{3EI}} \qquad E, P, h = \text{same}$$

$$K_{wall} = \cancel{3}I \qquad \text{Relative Stiffness} = \frac{K_i}{\Sigma K_i} = \frac{I_{wall}}{I_{total}}$$

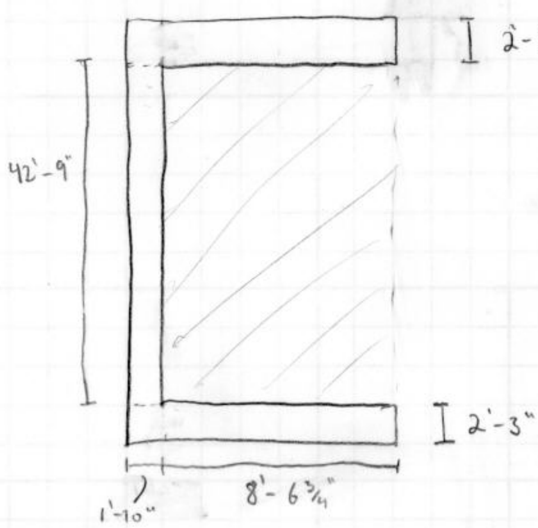
$$_{iy}d = \frac{K_{iy} P_y}{\Sigma K_y} =$$

Wall 3 w/ returns

By symmetry: $\frac{bh^3}{12}$ (whole) $- \frac{bh^3}{12}$ (non wall)

Since reduction of wall on B & C is constant, neglect & use 2' as width for I calcs.

LL1 - L7



whole: $\frac{bh^3}{12} = \frac{(22" + 8'(12)+6.75)\left[(46)(12)+15"\right]^3}{12}$

$= 1.895 \times 10^9 \text{ in}^4$

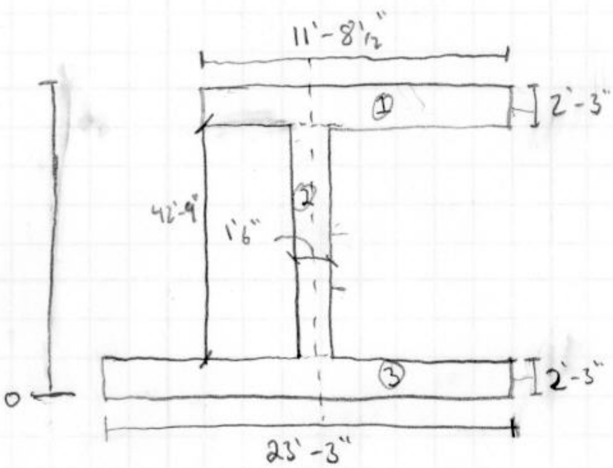
Non wall: $\frac{bh^3}{12} = \frac{(8'(12)+6.75)\left[42(12)+9\right]^3}{12}$

$= 1.156 \times 10^9$

I = whole - (non wall) =

$I = 7.39 \times 10^8 \text{ in}^4$

Wall 5 w/ returns

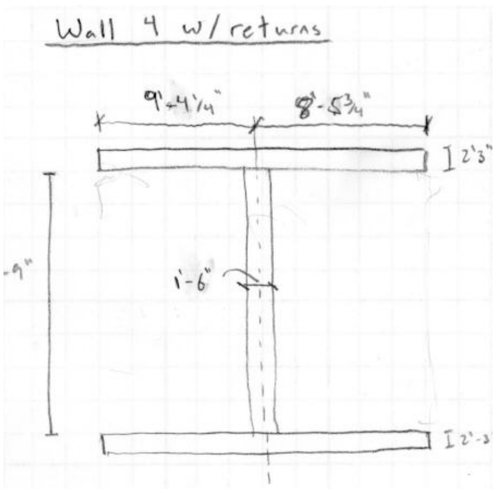


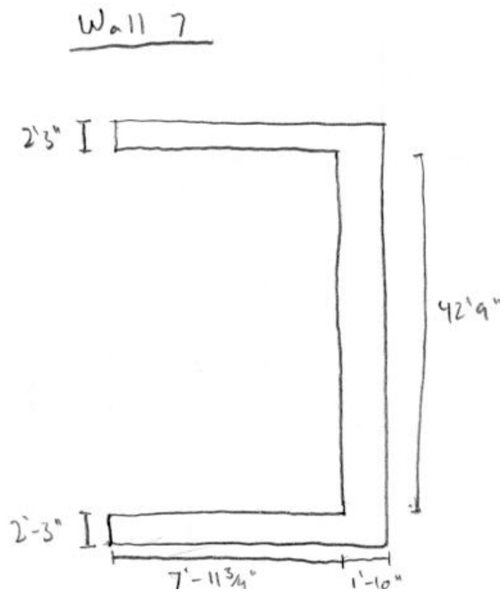
$$N.A. = \frac{A_1 y_1 + A_2 y_2 + A_3 y_3}{\Sigma A}$$

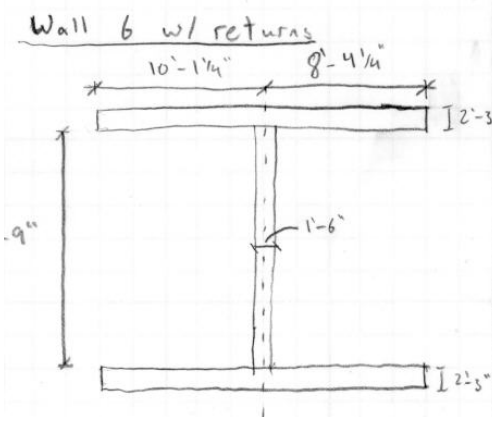
$$N.A. = \frac{(11'\text{-}8\tfrac{1}{2}")(2'\text{-}3")\left[2'\text{-}3" + 42'\text{-}9" + 2'\text{-}3"/2\right] + (42'\text{-}9")(1'\text{-}6")(2'\text{-}3" + 42'\text{-}9"/2) + (23'\text{-}3")(2'\text{-}3")(2'\text{-}3"/2)}{(11'\text{-}8\tfrac{1}{2}")(2'\text{-}3") + (42'\text{-}9")(1'\text{-}6") + (23'\text{-}3")(2'\text{-}3")}$$

$$I = I_1 + A_1 d_1^2 + I_2 + A_2 d_2^2 + I_3 + A_3 d_3^2$$

$$d = N.A. - y$$







Rest of Calculations Completed on Excel

Relative Stiffness - No Modifiers			Relative Stiffness - ETABS w/o truss		
Wall #	Lvl 1-7	Lvl 9-40	Wall #	Lvl 1-7	Lvl 9-40
Wall 3	16%	17%	Wall 3	17%	16%
Wall 4	23%	23%	Wall 4	22%	23%
Wall 5	22%	22%	Wall 5	20%	21%
Wall 6	24%	23%	Wall 6	23%	24%
Wall 7	16%	16%	Wall 7	18%	16%

Center of Rigidity

Center of Rigidity Levels 9-40			
	Ri	di (in)	Ri*di
Wall 3	0.165351	648	107.1477
Wall 4	0.229606	1003	230.2953
Wall 5	0.215983	1345	290.497
Wall 6	0.229606	1712	393.0863
Wall 7	0.159453	2048	326.5593
Center of Rigidity ΣRi*di			1347.586
Center of Rigidity ETABS			1354.669
% Difference			0.52%