Executive Summary:

Lateral System Analysis and Confirmation Design is an depth look at the lateral system of Lexington II and the loads of which it must carry.

The structural system of Lexington II is 2-way flat plate slab on a monolithically poured frame. Lexington II has a central core of shear walls which run the entire length of the building. The shear walls and columns rest directly on a MAT foundation. Of these systems, it is the shear walls which support the entire lateral load which affects Lexington II.

Wind and seismic loads are calculated so that it is possible to determine the critical load case as defined in ASCE 7- 02. For Lexington II the critical load case was found to be 1.2D + 1.6W +.5L + .5Lr.

Using rigidity, loads are distributed between the shear walls of Lexington II. Due to the small number of shear walls in Lexington II, it can easily be seen that in the E/W direction each wall carries 50% of the shear load while in the N/S direction there is only one wall which will carry the entire concentric shear. Mathematical calculations can prove this logic to be correct. Torsional loads are distributed between walls based on a wall's rigidity and proximity to the center of rigidity.

Lateral loads will have many effects on building design. Along with a shear force, lateral loads produce a torsional moment on floor diaphragms. This moment is transferred to the shear walls as an additional shear load. Lateral loads can also affect the building as an entirety by creating an overturning moment which must be opposed by the buildings dead load. Another way in which lateral loads can affect a building is by causing drift or horizontal displacement of the building.

All lateral loading affects on a building can be checked using hand calculations or computer software. The ETABS computer program can provide in depth data on applied lateral forces, story shear, pier torsion, and shear wall design. All calculations preformed by ETABS should also be verified manually. Both hand calculations and simple logic and help in this process.

For Lexington II it was determined that current shear wall system is efficient in supporting the building's entire lateral load while staying in deflection, over turning, and torsional requirements.

Introduction:

Lexington II at Market Square North is a 12 story residential tower in Washington D.C. Due to its location in the Historic Penn Quarter of Washington D.C., Lexington II is under a height restriction limiting all local buildings to 130 feet. Lexington II follows this restriction standing at a height of 116 feet with an additional 33 feet of below grade structure. This below grade area is designated for retail and parking. The floor area of Lexington II is approximately 72,000 square feet.

FOUNDATION:

Lexington II sits on a 3'-6" MAT foundation, constructed of 5000psi concrete. The MAT foundation rests on original bearing soil of 800psi. The south end of the MAT foundation is supported by HP 14 x 89 piles every 5 feet. These piles are in place to avoid undermining the foundation of a preexisting neighboring building with a lower foundation. The MAT foundation is reinforced with deformed #8 bars every 9 inches as well as additional top bars where needed.

The below grade structure consists of concrete retaining walls 14" thick which reduce to 12" thick by ground level. The retaining walls have a compressive strength of 5000psi and are reinforced with #4 bars every 12 inches running in the longitudinal direction and #5 bars every 12 inches running vertically.

GRAVITY SYSTEM:

The floor system of Lexington II is cast in place 2-way slab. The 2-way slab is flat plate slab which was monolithically poured with concrete columns. The 2-way slab is 8" thick with #4 reinforcement bars every 12". Additional top reinforcing bars are placed as needed. A typical flat plate slab bay has no drop panels and no edge beams. The slab is 4000psi compressive strength concrete.

The columns are placed on an irregular grid and are constructed with compressive strength concrete of 5000psi (See Figure 1 for column grid plan). Each column has been designed to support its load, as well as the load affecting the columns above it. This means that columns on the upper floors are generally smaller in size and contain less reinforcement than the columns located below them. Column sizes range from 14" x 14" to 42" x 14".

LATERAL LOADS:

Lateral loads in Lexington II are resisted by a core of small shear walls located around the elevator shafts (See Figure 1 for shear wall plan). The shear walls are cast in place concrete with a compressive strength of 4000psi. The walls are 12" thick and are reinforced with 2 #4 bars placed every 12" on center each way, each face.

Since Lexington II's gravity system is monolithically poured, it naturally creates moment framing. However, contact with the structural engineering confirmed that the

shear walls in Lexington II were designed with the intention of carrying the entire lateral load.

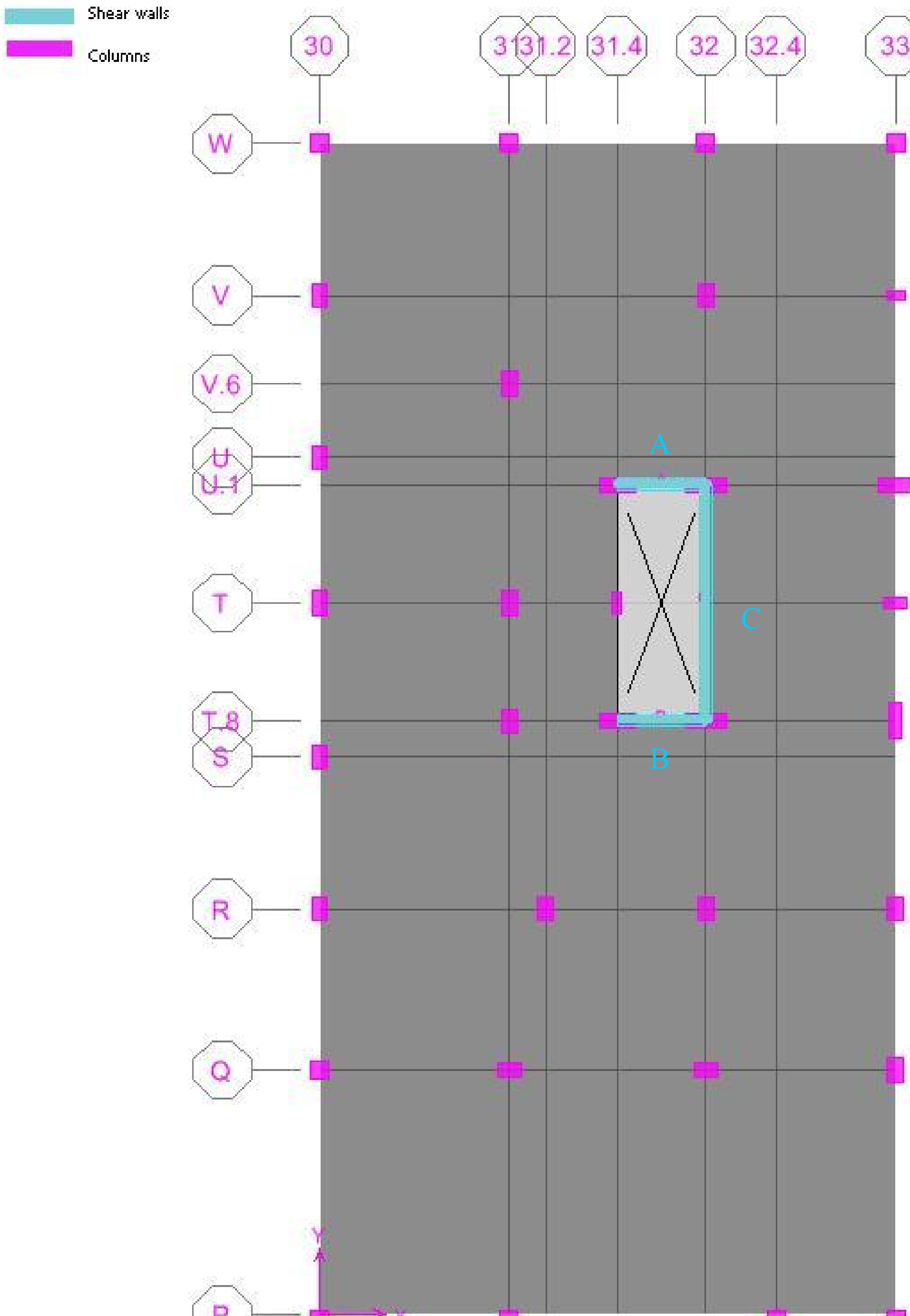

Figure 1. Plan of Lexington II showing column grid and shear walls.
Note: Shear wall digitations, A, B,C as shown above are used throughout this paper.

This report includes an analysis and confirmation of the integrity of the current lateral force resisting system. This analysis is based on the controlling load cases according to ACSE 7-02 and has been preformed by computer model analysis and checked with manual verification. Along with member strength; torsion, drift, and overturning moment have been checked.

Sections of this report are:

- Loads and Load Cases
- Distribution of Loads
- Analysis
- Member Checks
- Conclusion

Loads and Load Cases:

All loads for Lexington II have been calculated in accordance with ASCE 7-02 and are explained in more detail in technical report #1.

DEAD LOAD:

Finishes……………………………15psf
<u>Mechanical/Lighting………………. 5psf</u>
Total Superimposed………………..20psf

Self Weight Slab …………………..100psf (Appendix 1)
Exterior Wall………………………30psf

LIVE LOAD:

Residential Floors…………………40psf + 20psf for partitions
Public Levels………………………100psf
Roof……………………………….. 20psf (Appendix 1)

SNOW LOAD:

Snow Load…………………………15.75psf (Appendix 1)

WIND LOAD:

N/S direction

Floor	P (net)	Trib Area (ft^2)	Fx (kips)	Vx (kips)	Mx (kip ft)
ground	21.22	281.75	5.98	139.07	0.00
1	21.22	497.15	10.55	133.09	121.32
2	22.30	430.71	9.60	122.54	194.89
3	22.78	430.78	9.81	112.93	285.34
4	23.50	430.96	10.13	103.12	383.53
5	24.10	430.78	10.38	92.99	484.45
6	24.58	430.71	10.58	82.61	587.02
7	25.06	430.76	10.79	72.03	693.43
8	25.53	430.71	11.00	61.24	803.29
9	25.89	430.83	11.16	50.24	912.89
10	26.25	430.96	11.31	39.08	1025.34
11	26.25	446.02	11.71	27.77	1164.17
12	26.85	414.30	11.12	16.06	1210.73
roof	26.85	183.75	4.93	4.93	573.99

moment total
8440.40

E/W direction

Floor	P (net)	Trib Area (ft^2)	Fx (kips)	Vx (kips)	Mx (kip ft)
ground	11.51	575.00	6.62	170.79	0.00
1	11.51	1014.60	11.67	164.18	134.24
2	12.58	879.00	11.06	152.51	224.45
3	13.06	879.15	11.48	141.44	333.97
4	13.78	879.50	12.12	129.96	459.10
5	14.38	879.15	12.64	117.84	590.06
6	14.86	879.00	13.06	105.20	724.42
7	15.34	879.10	13.49	92.13	866.44
8	15.82	879.00	13.91	78.65	1015.64
9	16.18	879.25	14.23	64.74	1164.06
10	16.54	879.50	14.55	50.52	1318.20
11	16.54	910.25	15.05	35.97	1496.69
12	17.14	845.50	14.49	20.92	1576.94
roof	17.14	375.00	6.43	6.43	747.61

moment total
10651.82

For full wind load calculation, see Appendix 2.
See Figures 2, 3, 4 and 5 for load placement on building.

SEISMIC LOAD:

Floor	height (ft)	Total Load (kips)	wx*hx^k	Cvx	Fx (kips)	Vx (kips)	Mx (kip ft)
roof	108.58	423.23	68449.38	0.14	14.88	0.00	1615.74
12	99.17	457.01	66987.79	0.14	14.56	14.88	1444.20
11	90.375	454.65	60253.93	0.12	13.10	29.44	1183.82
10	81.58	454.63	53916.66	0.11	11.72	42.54	956.22
9	72.79	454.61	47641.36	0.10	10.36	54.26	753.89
8	64	454.61	41432.54	0.09	9.01	64.62	576.47
7	55.21	534.65	41510.32	0.09	9.02	73.63	498.23
6	46.42	548.54	35284.38	0.07	7.67	82.65	356.07
5	37.625	548.56	28094.23	0.06	6.11	90.32	229.80
4	28.83	548.53	21044.17	0.04	4.57	96.43	131.90
3	20.042	548.52	14183.91	0.03	3.08	101.01	61.80
2	11.25	545.65	7540.78	0.02	1.64	104.09	18.44
Ground	0	540.29	0.00	0.00	0.00	105.73	0.00
			486339.46				

Total Building Weight (kips)	6513.4607
Overturning Moment	7826.58356

For full seismic loading calculations, see Appendix 3.
See Figure 6 for seismic load placement on building.

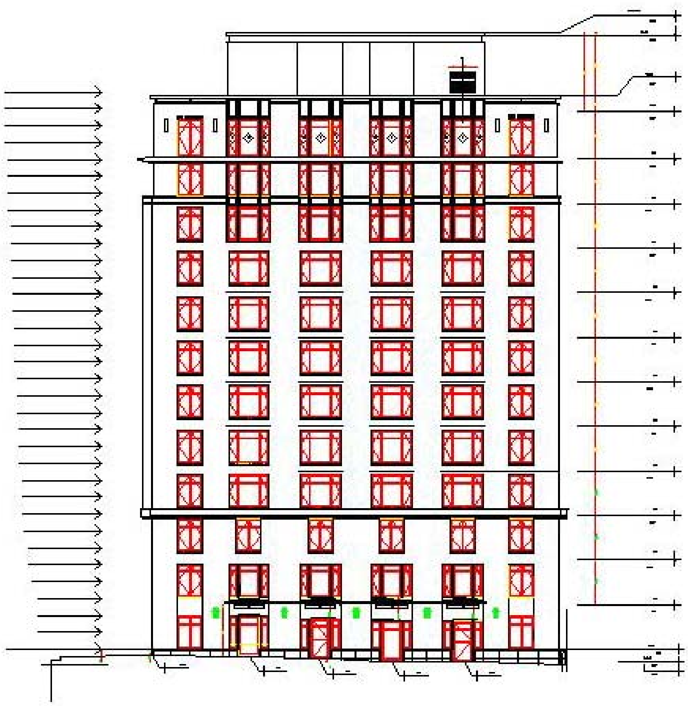

Figure 2. Wind hitting the building in the North South Direction

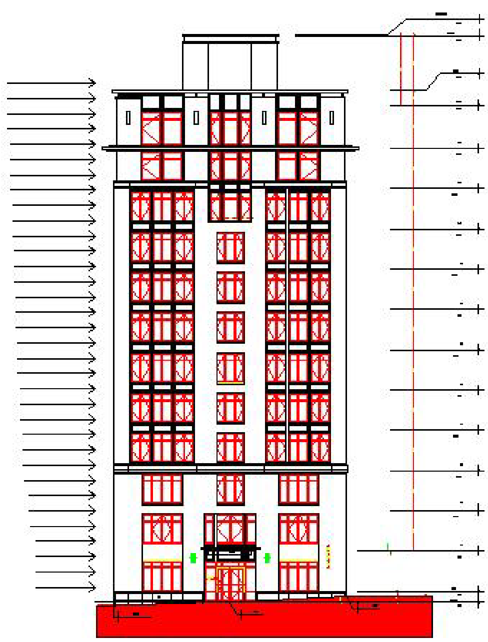

Figure 3. Wind hitting the building in the East West Direction

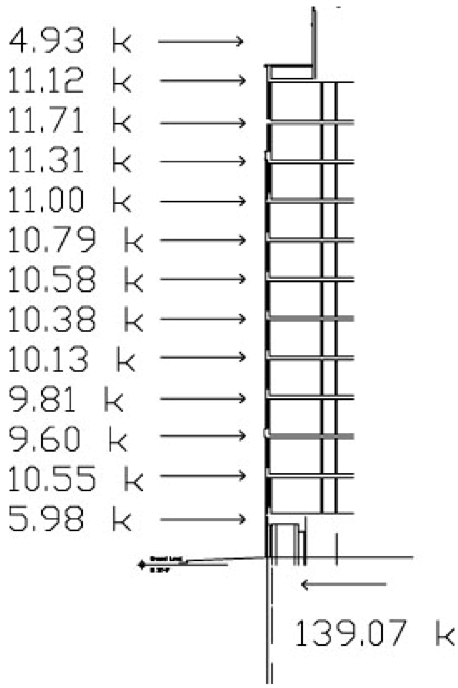

Figure 4.
Wind (North-South Direction) Story Forces

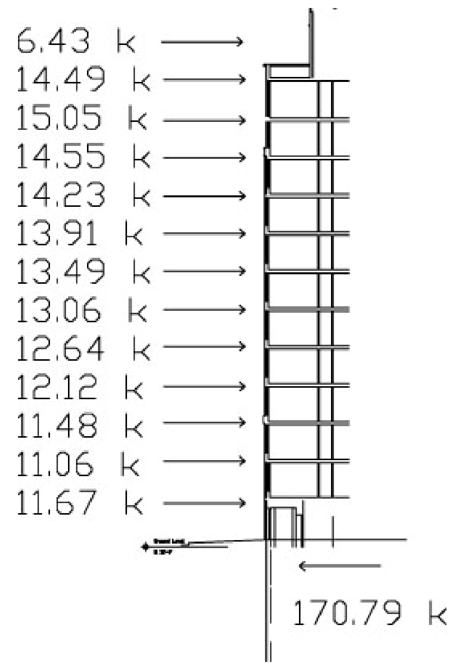

Figure 5.
Wind (East-West Direction) Story Forces

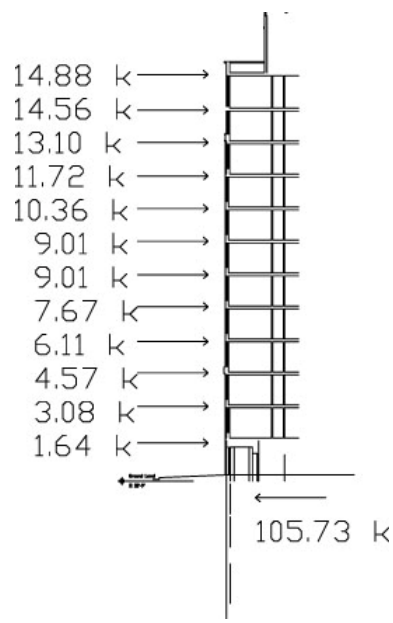

Figure 6.
Seismic Story Forces

LOAD COMBINATIONS:

Taken from ASCE 7-02.

1.4D
1.2D + 1.6L + .5Lr
1.2D + 1.6Lr + (L or .8W)
1.2D + 1.6W + .5L + .5Lr
1.2D + E + .2S
.9D + 1.6W + 1.6H
.9D + 1E + 1.6H

The controlling load case will be 1.2D + 1.6W +.5L + .5Lr. This was determined by running all load cases (psf) in an excel spread sheet. See Appendix 4 for excel spread sheet and results.

Distribution of Loads:

Loads were distributed to each shear wall based on rigidity. All shear walls run the entire length of the building, and therefore the rigidity ratio between shear walls will not vary by floor. To simplify the shear wall rigidity calculation, rigidity was taken to be R/Et. This is accepted for buildings such as Lexington II where all shear walls are constructed of concrete with the same modulus of elasticity and are the same thickness. The wall rigidity was divided by the sum of the rigidities for all shear walls acting in the same direction; this gave the proportion of the story shear that each wall carries.

The results of this are easily checked for Lexington II by common logic. There is only one shear wall acting in the North-South direction, this wall therefore carries the entire North-South lateral load and has a proportion of 1. Running in the East-West direction are two identical walls. Due to the equivalence of their lengths, both of these walls have the same rigidity and therefore are each proportioned to carry one half of the lateral load acting in the East-West direction.

Proportional Distribution of Concentric Shear:

Wall A = 50% of E/W load
Wall B = 50% of E/W load
Wall C = 100% of N/S load

As well as concentric shear distribution explained above, distribution of eccentric shear must also be accounted for. To distribute the eccentric shear, the center of rigidity for the entire building must be found. This is determined by the previously found rigidities their geometric placement in regard to one another.

N/S:

X= 1*33’/1
= 33’ from bottom left comer

E/W:

Y= ((.5* 50.7) + (.5*70.9)) / (.5+.5)
= 60.8’ from bottom left corner

Eccentric shear (or torsion) will be distributed between shear walls based on the proportion of rigidity*distance from center of rigidity. Full calculations verifying the shear and torsional proportions each wall carries can be seen in Appendix 10.

Proportional Distribution of Eccentric Shear:

Wall A = 4.4%
Wall B = 4.4%
Wall C = 2.5%

Logically, the lateral loads will be carried to the base of the building following the shear walls. Load follows stiffness, and the shear walls are the stiffest element in Lexington II. Lateral loads will act on the exterior wall of the building which will transfer them into the floor diaphragms and finally from the floor diaphragms into the shear walls.

Analysis:

Analysis of the lateral force resisting system was done two ways, using an ETABS computer generated model and by hand calculations to verify the ETABS result. As mentioned before, the structural engineer confirmed that the shear wall system is the primary lateral load resisting system and was designed to carry Lexington II’s entire wind and seismic loadings. For this analysis, the moment frame and columns were ignored as lateral bearing elements to comply with the original design of Lexington II.

A three dimensional model was created using ETABS computer software. Although the columns are being ignored as lateral resisting elements, they were included in the model to carry the Lexington II’s self-weight and other gravity loads which can affect the seismic load on a building. To ensure that the moment frame would not interfere with shear wall analysis, all columns were modeled as pin-pin supports to the floor diaphragms above and below them. The base level was modeled with fixed supports into the ground.

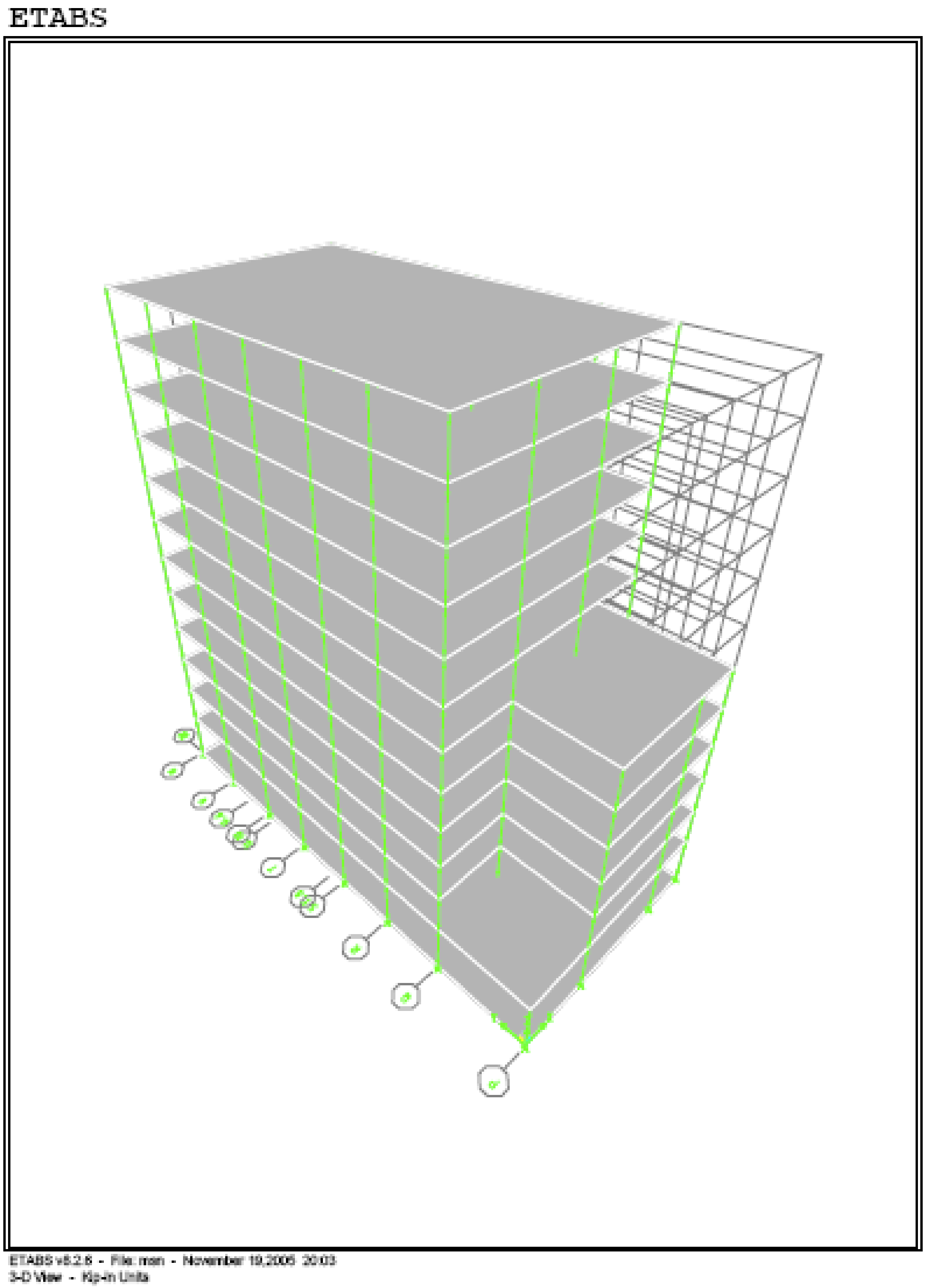

Figure 7.
3-D ETABS Model of Lexington II

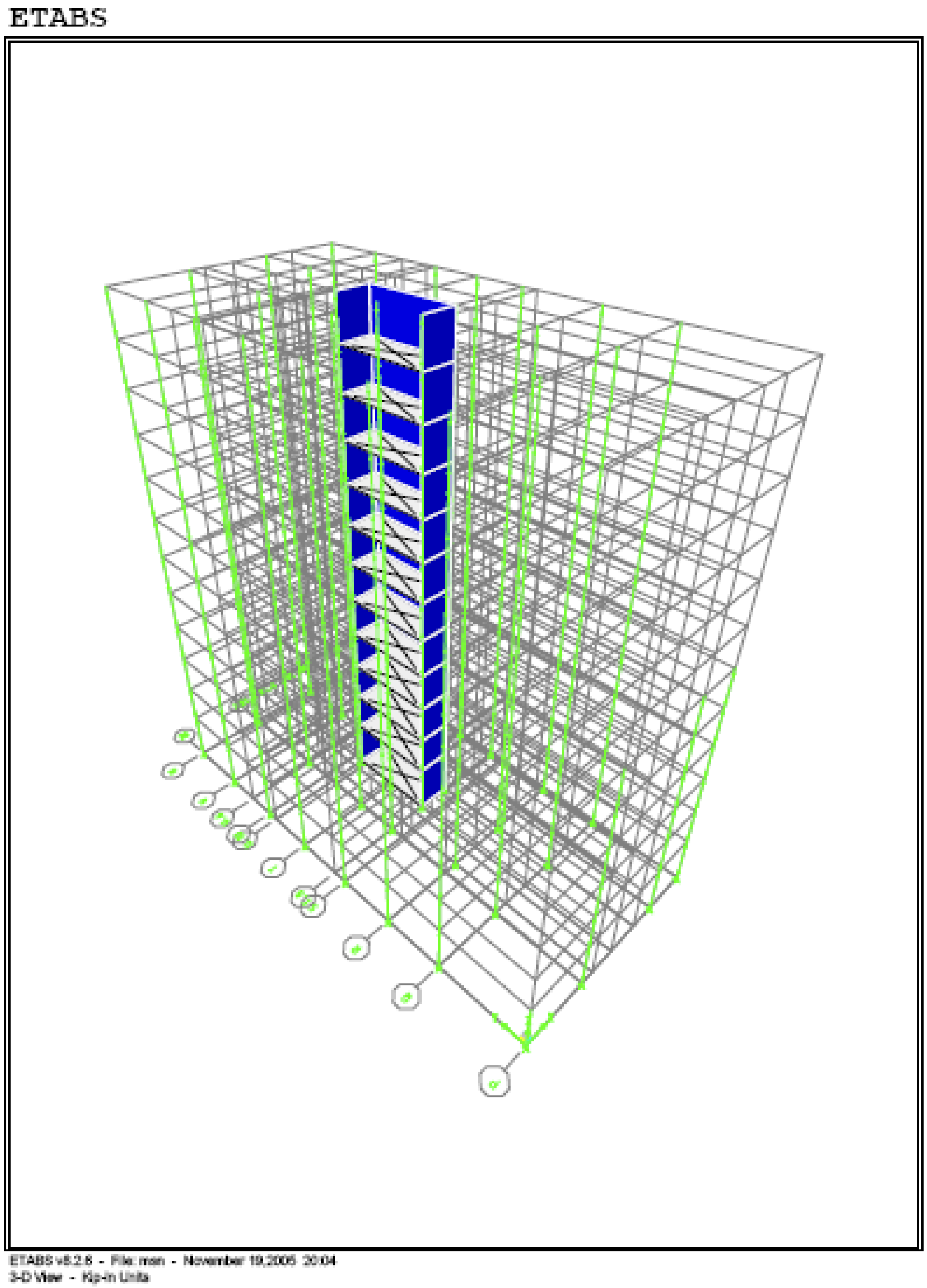

Figure 8.
Lexington II's Shear Wall Core as Modeled in ETABS

Running the ETAB model for all load cases, automatically calculates the wind and seismic loading applied to the building. The results of the loads ETABS calculated are summarized in Appendix 5. The story forces ETABS calculated are less then the previously calculated wind loads, but greater then the calculated seismic loads. The loads however, do not differ great enough to be a large concern that there is an error is one set of the calculations. Loads are displayed as a point load and an equivalent moment load to account for eccentric towards the center of rigidity.

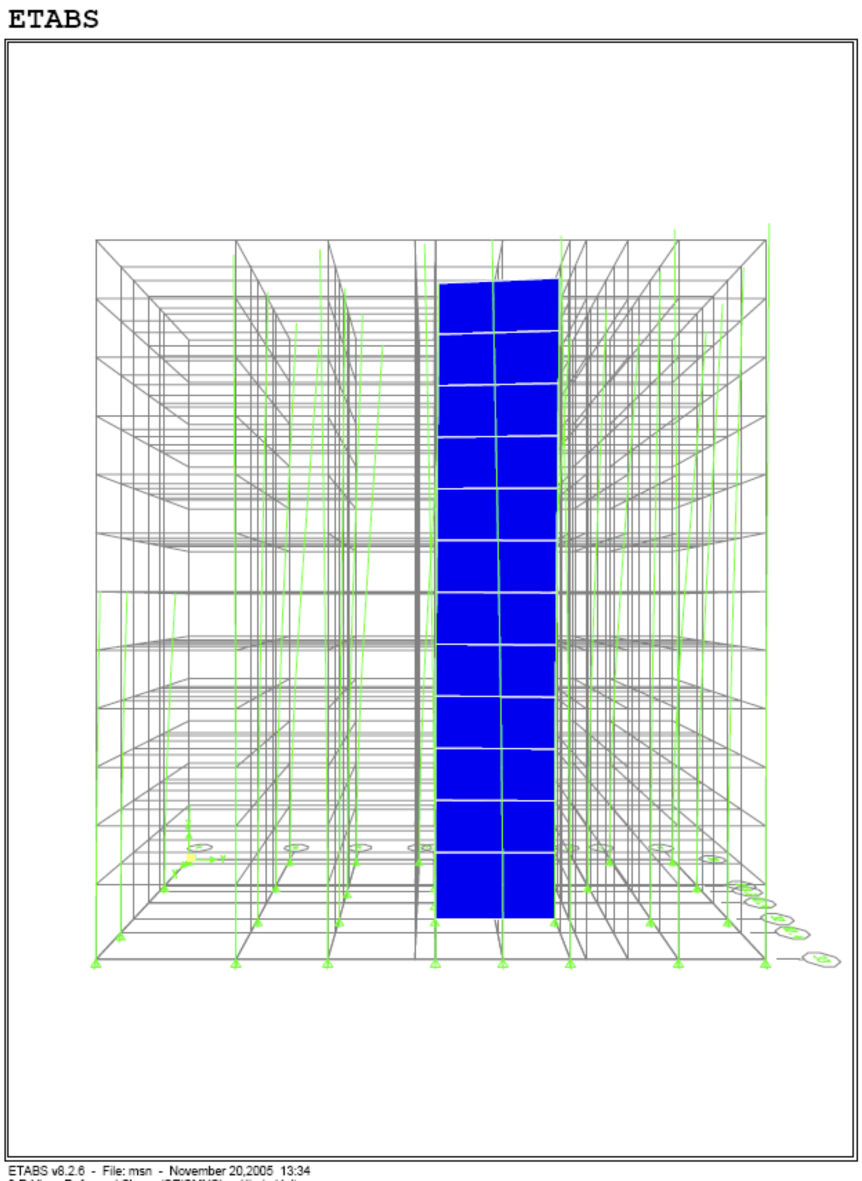

Figure 9.
Typical N/S deflection of Lexington II

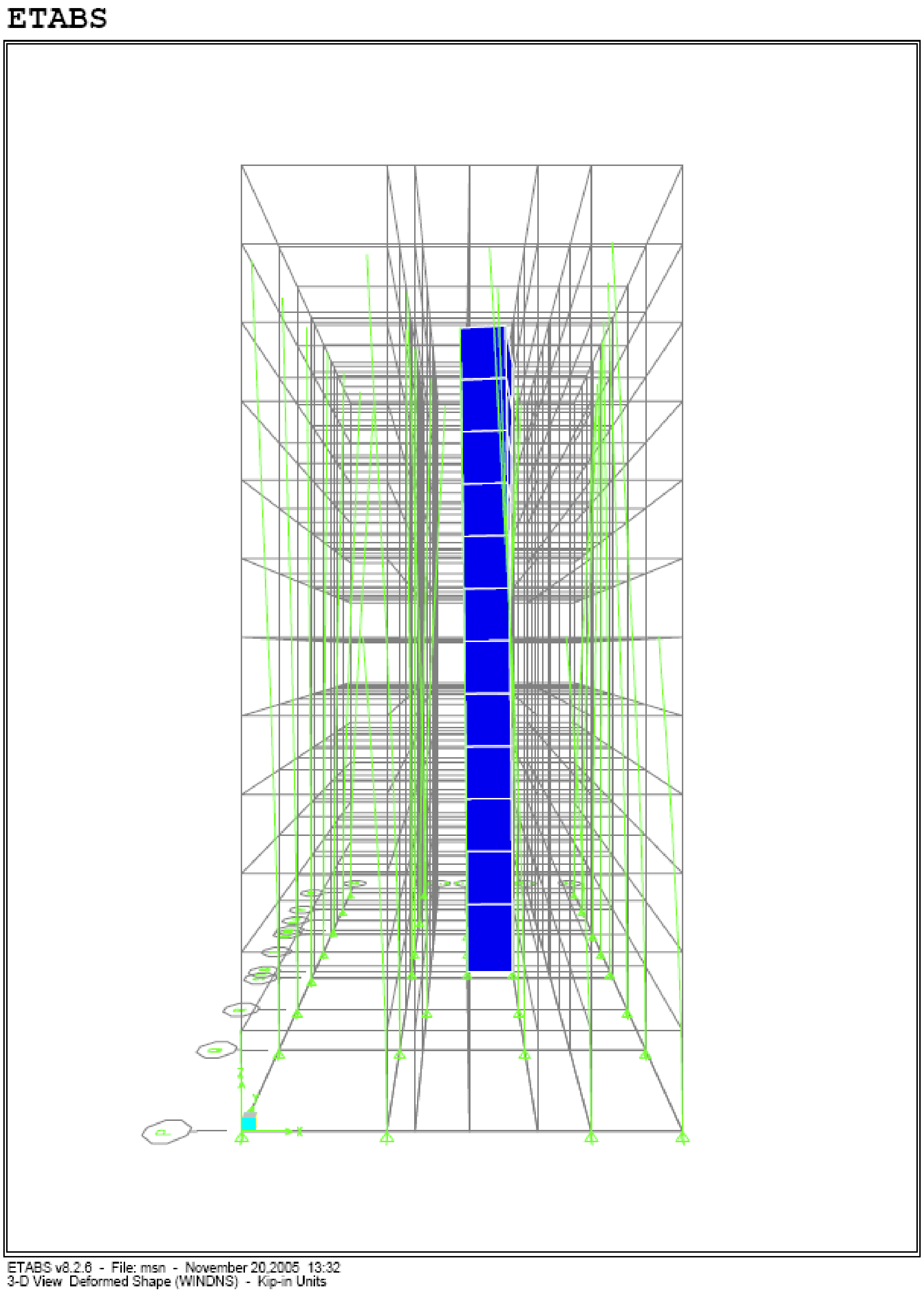

Figure 10
Typical E/W deflection of Lexington II

DRIFT:

Story drift is automatically calculated using ETABS. ETABS will provide the drift per story height of each floor. The data output from ETABS for wind acting in the N/S direction was placed in an excel spreadsheet and summed to produce a value of total building drift, 0.28 feet or 3.36 inches, for wind in the North-South direction. (See Appendix 6) Comparing this number to the allowable drift value of h/400 verifies that Lexington II is within its maximum drift.

H / 400 = 116' / 400 = .29' > .28'   Okay!

To verify that the above is a reasonable deflection for the given loading, hand calculations were preformed. Two types of calculations were preformed. The first set of calculations was to try to determine the drift per story of the building based of wall flexure and shear. Results can be viewed in Appendix 7. A second approximate deflection can be found if the shear walls are assumed to be a cantilever (Appendix 8). Deflections on a cantilever were solved for three ways, each using the loadings provided in Figure 2. First the load was assumed to be an uniform load acting on a cantilever. The uniform load was the average of the wind pressures acting on each story (psf) times the building width on which it acts. The second attempt accounted for the increasing wind pressure on the higher stories of Lexington II. A point load acting at the 2/3 point of the increasing uniform load was used. The third approach was to account for, a deflection due to a moment on the beam. This moment was added to regular deflection since the lateral loads do not act directly on the center of rigidity, but rather 8" to its left at the geographic center of the building.

Conclusions to the hand calculations were slightly lower then expected but reasonable. The values were run twice, once treating the shear walls as a single element and a second time for just wall C since wall C is the only shear wall acting in the N/S direction. ETABS drift values were larger, but this is probably because ETABS is designing the walls to meet H/400. Contradictory, the hand calculations are checking the existing wall's design which may not be drift controlled.

TORSION:

Torsion of Lexington II can be viewed using the ETABS model.

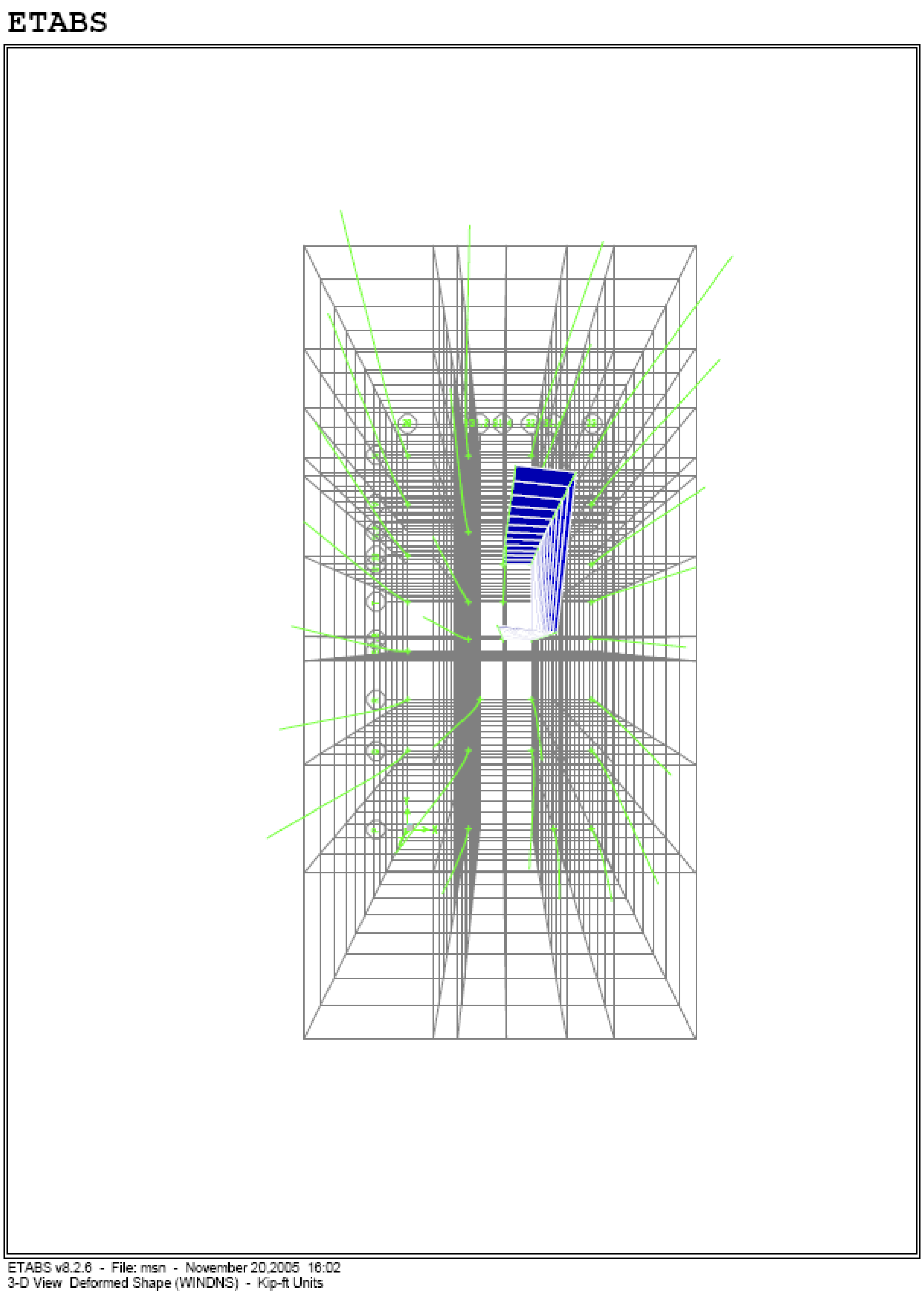

Figure 11.
Torsion in Lexington II.

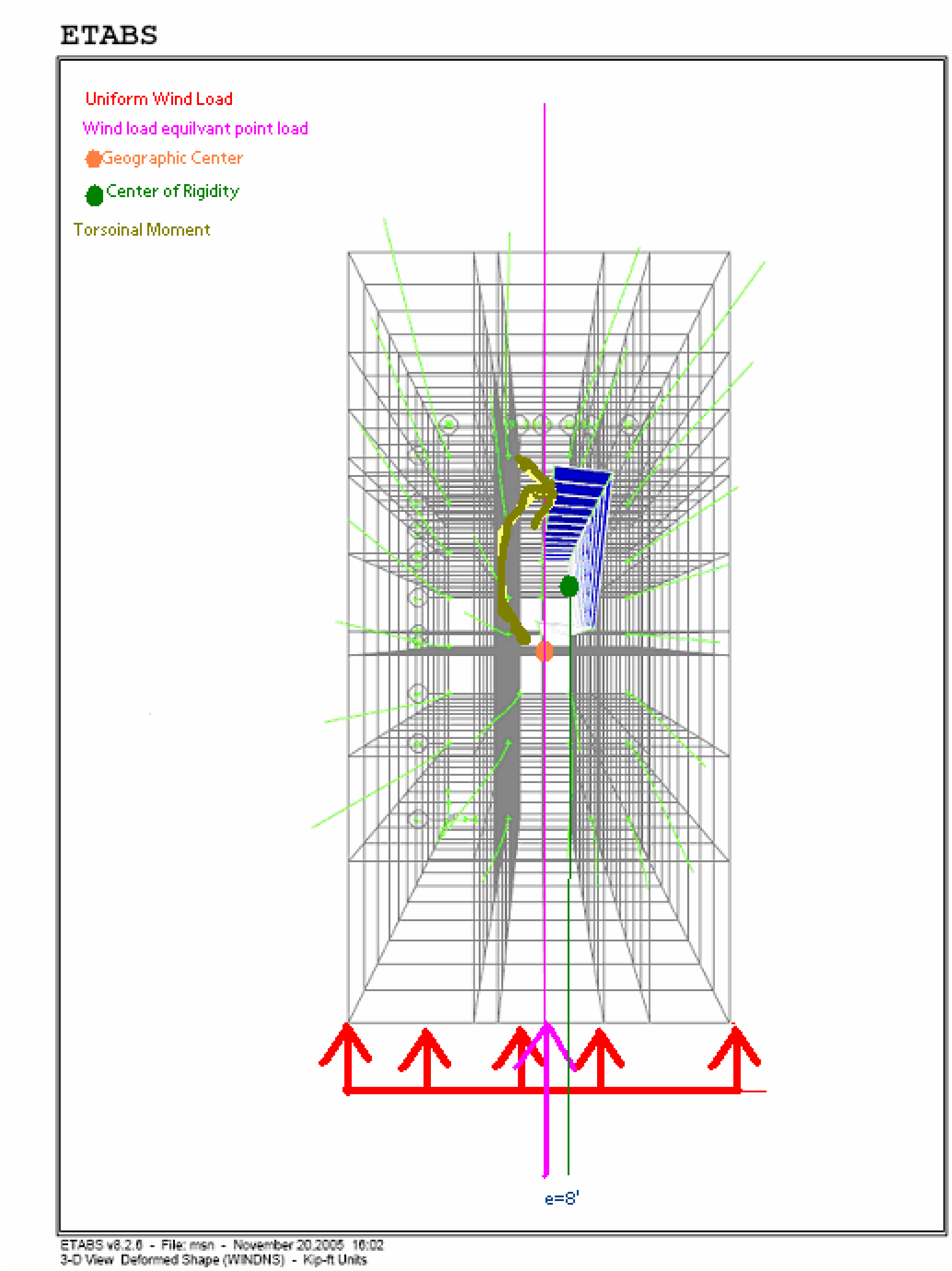

Figure 12.
How lateral loads cause torsional moments.

Torsional load is caused by a building's natural tendency to absorb load based on stiffness and rigidity. In most buildings the center of rigidity is not the same point as the geometric center of the building. A uniform load (shown in Figure 12) will act on the geometric center of the building. In turn, a moment arm is created as the building trys to rotate about its center of rigidity. This moment is considered the torsional moment of the building and is the story load acting on the center of mass of the diaphragm times the eccentric distance to the center of rigidity of the diaphragm. Torsion, like force loads, is distributed based on wall stiffness or rigidity. The proportions of torsional moment that each wall carries are calculated in the distribution of loads section of this report. The building's torsion is another component of the lateral load system and is therefore carried by the same structural elements as the lateral load. Thus, all lateral force resisting systems must be designed to carry the additional shear load produced by torsional moments.

ETABS prepares its own results of the torsional load which each wall sees per-story. An example of the ETABS results for torsion in Wall A are below. For full ETABS results, see Appendix 9.

Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
STORY12	A	WINDNS	Top	-1.71	-6.53	-0.02	-36.053	-0.023	21.79
STORY11	A	WINDNS	Top	-5.08	-3.2	0.02	-36.423	-0.348	-1.408
STORY10	A	WINDNS	Top	-8.36	0.07	0.05	-36.865	-0.403	-13.81
STORY9	A	WINDNS	Top	-11.55	3.31	0.08	-37.128	-0.342	-15.763
STORY8	A	WINDNS	Top	-14.63	6.6	0.11	-36.943	-0.137	-7.553
STORY7	A	WINDNS	Top	-17.55	9.99	0.13	-36.034	0.228	10.672
STORY6	A	WINDNS	Top	-18.72	13.5	0.15	-34.246	0.66	32.16
STORY5	A	WINDNS	Top	-20.57	17.17	0.2	-31.382	1.177	67.094
STORY4	A	WINDNS	Top	-21.93	21.1	0.17	-27.038	2.01	111.46
STORY3	A	WINDNS	Top	-26.7	25.13	0.12	-21.785	2.503	102.607
STORY2	A	WINDNS	Top	-28.98	29.02	0.49	-15.963	2.362	149.224
STORY1	A	WINDNS	Top	-31.85	34.62	-0.44	-6.095	5.587	166.369

The results given for torsion by ETABS are very logical. Walls A and B carry the same amount of the torsional load, which is correct according to their proportions of distributed load.

A hand calculated example of the torsional shear each wall will carry can be found in Appendix 10. These calculations show all work including how the center of rigidity, proportions of shear, and proportions of torsional shear were found for Lexington II.

OVERTURNING MOMENT:

A third effect of lateral loads which must be considered in building analysis is overturning moment. Overturning moment is moment created by lateral loads affecting a building at varying heights creating moment arms.

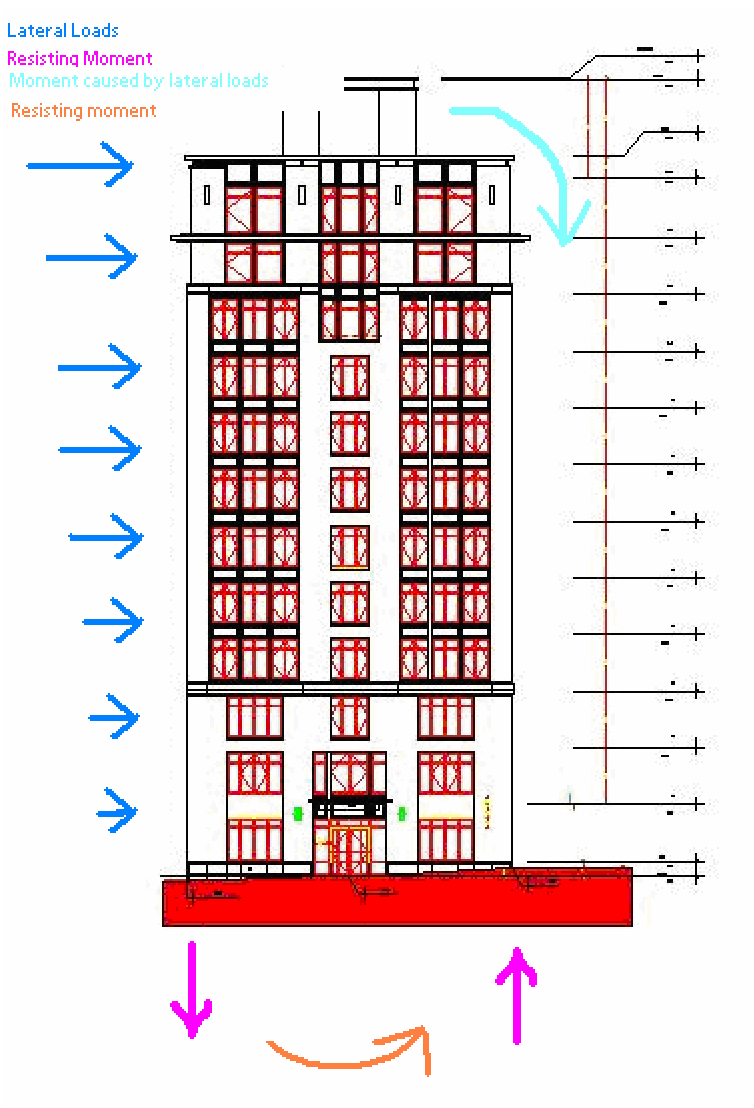

Figure 13.
Overturning Moment and resisting forces.

Over turning moment can be calculated by multiplying the story force at each level by the distance that level is from the building's base. Alternatively, moment on each level can be found by multiplying the shear force at each level by the floor to floor height of that level. The moment at the bottom story will be equal to the over turning moment. This is true because shear loads add downward so that the shear at any level is equivalent to the sum of all lateral loads acting above it. The same calculations can be preformed on a wall to wall basis. Using the second method described, the over turning moment of each shear wall was determined and can be seen in Appendix 10 under the Base Moment Section.

The overturning of the building must then be checked with a resisting force, caused by the building's dead load. Similarly, the over turning moment of each wall can be checked using the gravity loads on that wall. In Lexington II the only dead load applied to the shear walls is self weight; it is this self weight which will cause the resisting moment.

Self Weight of Wall A:
150pcf * 12' thick * 116' tall * 7.5' wide= 1566 kips

Over Turning Moment on Wall A (from Appendix 10):
4034 kip ft

Resisting Moment (caused by self weight):
1566* (7.5/2) = 5872.5 kip-ft

Over Turning Moment < Resisting Moment okay!

(When known, uplift forces should also be considered.)

Member Checks:

After the lateral loading conditions have been calculated and shear from loads and torsion are know, the members can be designed. ETABS has the capability of designing members as well as analyzing them. As stated earlier, the ETABS model was designed so that columns in the moment frames would not carry any lateral load. The shear walls were modeled to their existing size and ETABS was used to calculate the shear reinforcement which is needed. The ETABS design of the shear walls included a longitudinal steel reinforcement ratio of $\rho = .003$ at the top of the shear wall to $\rho = .017$ at the base and a required shear steel area of .36 sq in per foot. Results are displayed in Appendix 11.

The current design of #4 bars every 12” each way, each face, means that the total steel area per foot is .4 sq in. This meets the .36 sq inch designed for by ETABS. Designing for steel with a lower area than .4 sq in would not be reasonable in that it is impractical to use bars small then #4’s and because lateral load must be considered in both directions, therefore both wall faces will have tension forces and must contain reinforcing steel.

The longitudinal reinforcement area of $\rho = .017$ is not as easily met. The current design (#4’s 12” o.c., EW EF) has a steel area of .4 square inches. This only provides a reinforcement ratio of .4/ (12*12) = .00278.

Hand calculations of member and member strengths were preformed in technical report #1 according to ACI 02. To check member strengths and design the reinforcement in the shear walls, the shear load carried by the concrete wall was determined. This concrete shear load was then subtracted from the total calculated shear load on the wall. Any remaining load, not carried by the concrete, must be carried by the steel reinforcement.

Appendix 12 shows hand calculations for Wall A’s reinforcement. These calculations were done for wind in the N/S direction, the controlling load case based on manual calculations (ETABS found seismic to control). Wall A was designed for the first story since this the floor which will see the most shear load. Shear load was taken from Appendix 10 calculation. The torsional load applied to the wall was found from ETABS data (which is shown in the table on page 18 of this report). Calculations show that that shear load needs reinforcement, but the minimum amount is acceptable. Required reinforcement for the shear and torsion loads is .4 inches squared per foot. This is exactly the provided area of #4’s at 12 inches each face, each way. Checking for longitudinal reinforcement in wall A found $\rho = .0025$ inches squared, which is a steel area of .18 square inches. Therefore, #4 bars should be used every 12” (As= .2 sq in) as in the current design of the shear wall. Again, because the direction of the load changes both faces should be reinforced since both will see tension.

Conclusion:

ETABS data and hand calculations are not always perfectly equivalent as would ideal. This report shows many examples of data not perfectly coinciding which can largely affect the outcome of a building design. Hand calculations found 1.2D + 1.6W +.5L + .5Lr to be the controlling load case. Loads given from ETABS varied slightly from the hand calculation. It is possible that this variation in load forces may cause a different loading combination to be the critical case. With more time it would be beneficial to perform these additional calculations.

Using both ETABS and hand calculations is a good way to ensure that values are correct. Recalculating ETABS and hand data until they are relatively similar is a good process to help eliminate errors. Member strengths and reinforcement design will be more accurate if there is no question about which data is more correct. For this report,

member design used the values of shear and torsion acting a wall based on which value (ETABS or hand data) made more logical sense. Shear was taken from the hand calculation while ETABS data was used for the torsional shear force. ETABS was taken to be more accurate for the torsional shear because in a building the torsional force should decrease as one goes lower in floor. The hand calculations showed an increase in torsional shear at the lower floors, and is therefore through to be calculated incorrectly.

This report found, that when using values logically thought to be the most correct, the shear walls of Lexington II effectively carry the building's entire lateral load, however are not more conservative than the factors of safety included in the ACI and ASCE 7 codes. When originally designed for BOCA 1997, the building may have been more conservation or had differing critical load cases. Also, it should be recalled that the frame of Lexington II is monolithically poured concrete. Being such, Lexington II features a moment frame which in reality probably carries at least a part of the lateral load. Other differences which were not included in this model are edge beams located sparingly thought out Lexington II which would affect the load paths in the building.