Design- Lateral System:

Investigation:

When considering the design of the lateral load resisting system, first a look was taken at the existing shear walls. The existing lateral system in The Lexington consists of a core of shear walls designed as the elevator shaft of the building. Before a new lateral system was investigated, the existing shear walls were again considered as the main lateral force resisting system.

The elevator core is an architectural feature of the building and, as such, will remain unchanged in the new building design. The elevator shaft therefore lends itself nicely as shear walls which will run the entire height of the building uninterrupted. Logic shows that the existing shear wall specifications are adequate to carry the new lateral loads which may be applied to the building. By changing the building system to a steel frame, the floor sandwich depth of each level was doubled resulting in a total height change of approximately 8'. Although this change in height will add additional wind load to the building, it was the seismic lateral loads which were the controlling load case for the upper stories, the wind load on the lower stories will remain comparable to the current load. The seismic load associated with the steel redesign of The Lexington will also differ from the load applied to the existing building. As the Lexington gravity system was redesigned in steel, the weights of the building were changed altering the seismic load the building will receive. By converting the current concrete system to a composite steel system, the building weight will be reduced. In turn, this lighter weight will cause a reduction in the seismic load making the seismic load on the new building design less then the load on the original design. With this considered, the shear walls used in the original design of the Lexington will be able to carry the new load. The reinforcement and materials used in the current shear walls are already minimal and a reduction in the size and reinforcement of the existing shear wall is unnecessary.

To verify the above assumptions, the same ETABS model used when evaluating the existing shear walls was altered to evaluate the shear walls when used with the composite floor system. Changes to the ETABS model include increasing the building height by adding 8 ½" to each floor and reducing the dead load to 32 psf. The results of the shear wall analysis were very similar to the results calculated for the existing structure and are as predicted above.

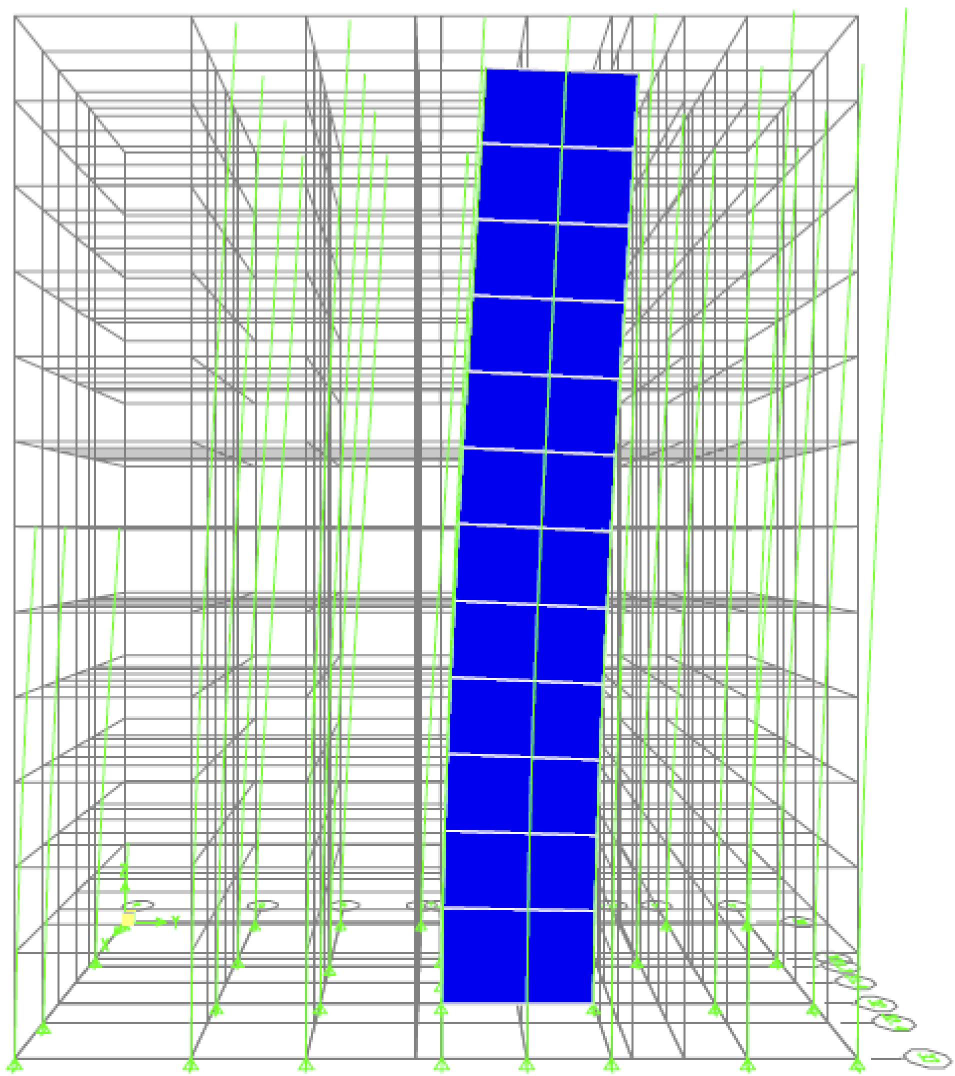


Figure 19
ETABS Model. Shear wall results auto scaled by 3000

Although the existing shear walls work, it may be in the interest of the designer to try other lateral force resisting systems as well. By considering alternatives a more cost efficient solution may be possible. The other systems considered in the redesign of The Lexington include moment and braced frames. Both moment and braced frames have advantages and disadvantages.

Moment frames were the first system I considered for The Lexington. The greatest benefit of using a moment frame in The Lexington is that moment frames are unobtrusive. This will allow for moment frames to be placed in bays that span living areas and other spaces that must remain open. The biggest disadvantage with moment frames in The Lexington, like in most buildings, is the cost associated with them. Moment frames would require very specific connection detailing and assembly.

Braced frames were also a possibility for use in The Lexington. Braced frames are easier to erect from a constructability aspect and have higher strength and stiffness than other lateral systems. The only obstacle in placing braced frames is to find locations where they will not interrupt the architecture of the building and can be concealed in walls without obstructing window or door placement.

The design decided on for The Lexington, was to use braced frames in place of the shear walls. The braced frames will be easier and more cost efficient to construct than the moment frames. Braced frames can be located in the same place as the exiting shear walls, around the building's elevators and stairwells, to avoid interference with the architecture. If additional braced frames are needed, they can be placed along the exterior walls of The Lexington which abut the adjoining building. I have also found several other frames in which braces can be added with minimal conflict with the existing architectural.

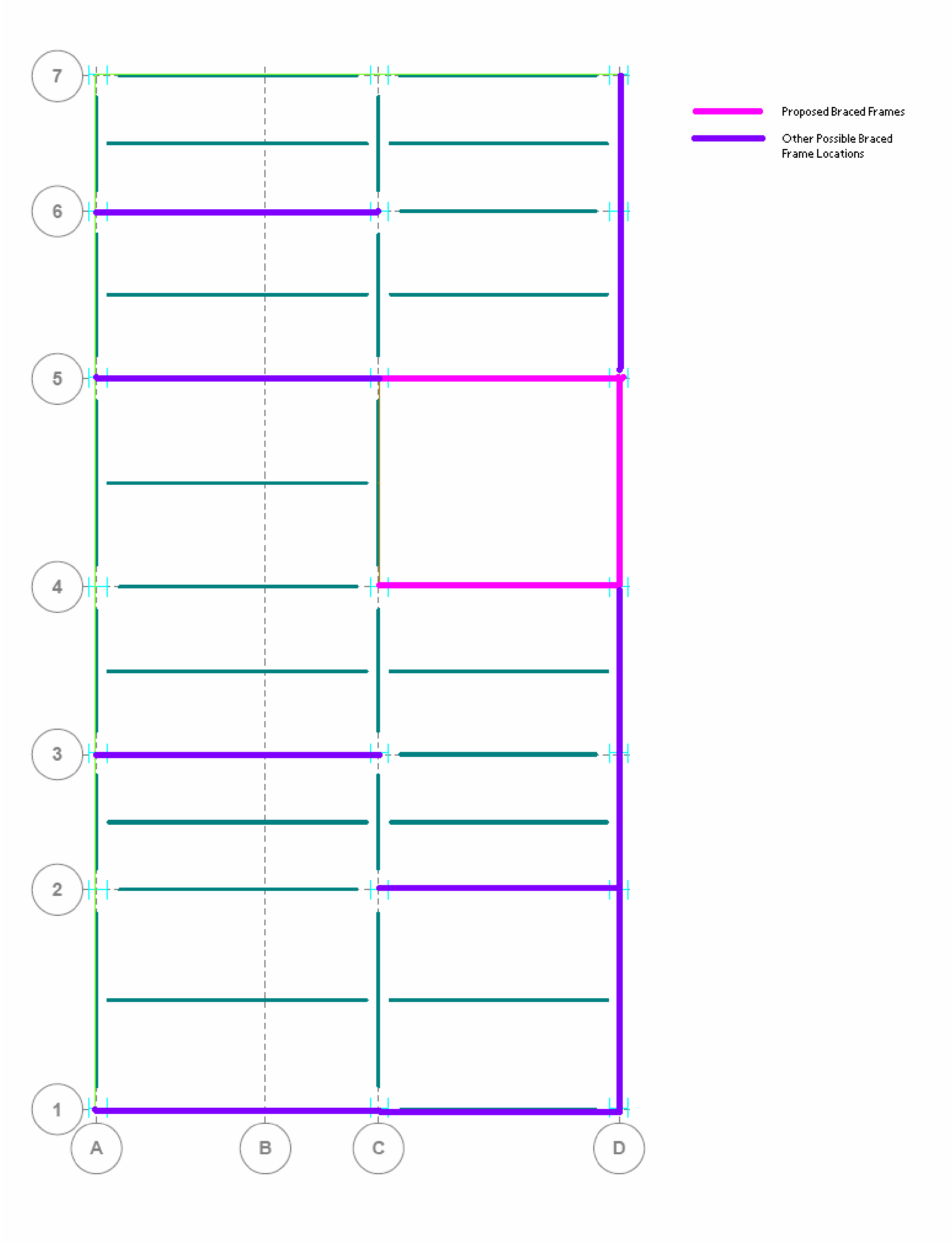


Figure 20
Proposed Brace Frame Locations

Loads:

Before the new lateral system can be designed, the lateral loads affecting the building must be calculated. The lateral loads on the redesign will differ from the loads acting on the current building because of changes in the height and weight of the building.

The height of the building will be increased to accommodate the new floor sandwich associated with the steel gravity system. The total depth of the new system includes 12" deep beams and 2" composite deck topped with a 2.5" concrete slab, or 16.5 total inches. In comparison, the existing building used 8" flat plate two way slab. The total height difference of the building is 8.5" per floor for 12 above grade floors, which add 8.5' to the building height. The new height information was input into the same excel spreadsheet used to calculate the wind loads on the actual building. Results of wind loading on the new height are as below:

N/S direction

Floor	P (net)	Trib Area (ft^2)	Fx (kips)	Vx (kips)	Mx (kip ft)	
ground	21.93	294.00	6.45	159.20	0.00	
1	21.93	526.75	11.55	152.75	138.63	
2	23.01	465.50	10.71	141.20	230.31	
3	24.21	465.50	11.27	130.49	349.40	
4	24.81	465.50	11.55	119.21	467.78	
5	24.81	465.50	11.55	107.66	577.51	
6	25.29	481.06	12.17	96.11	723.95	
7	25.77	496.13	12.79	83.95	890.38	
8	26.25	495.88	13.02	71.16	1038.20	
9	26.61	496.13	13.20	58.14	1186.64	
10	26.97	496.13	13.38	44.94	1338.18	
11	26.97	496.13	13.38	31.56	1473.67	
12	27.57	451.41	12.45	18.18	1496.71	
roof	28.17	203.35	5.73	5.73	736.45	moment total
						10647.84

E/W direction

Floor	P (net)	Trib Area (ft^2)	Fx (kips)	Vx (kips)	Mx (kip ft)	
ground	11.75	600.00	7.05	193.95	0.00	
1	11.75	1075.00	12.63	186.90	151.52	
2	12.83	950.00	12.18	174.27	261.97	
3	14.03	950.00	13.32	162.09	413.07	
4	14.63	950.00	13.89	148.76	562.74	
5	14.63	950.00	13.89	134.87	694.74	
6	15.11	981.75	14.83	120.98	882.41	
7	15.59	1012.50	15.78	106.14	1098.90	
8	16.07	1012.00	16.26	90.36	1296.65	
9	16.43	1012.50	16.63	74.10	1494.75	
10	16.79	1012.50	17.00	57.47	1699.60	
11	16.79	1012.50	17.00	40.48	1871.69	
12	17.39	921.25	16.02	23.48	1926.05	moment total
roof	17.99	415.00	7.46	7.46	959.54	13313.62

Table 4

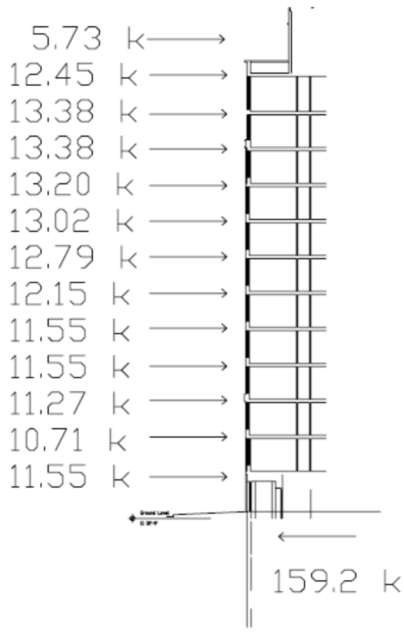
The other lateral load to be recalculated was the seismic load. Like the wind load, the excel spreadsheet used to calculate the load on the existing building was reused with proper adjustments. In the seismic case, data on the spread sheet was changed to reflect the building's new weight. The weight of the composite deck and slab was given from the decking catalog as 34 psf. The average weight of the steel framing system was determined by multiplying the weight of the beam in lb/in by the length of the beam in inches. The total weight for every beam on a floor was added together to find the framing system weight and then divided by the area of the floor to achieve the units of psf. This had to be done twice for the two varying floor plans used on residential levels. Along with the weights, other factors had to be changed to comply with the ASCE 7 code. These include the building height, response modification factor (R), and any variable which was affected by the type of lateral load resisting system used, such as C_t, x, W_o, and C_d. Results of the earthquake loading are as follows below:

Floor	height (ft)	Total Load (kips)	wx*hx^k	Cvx	Fx (kips)	Vx (kips)	Mx (kip ft)
roof	120.25	294.37	258349.75	0.16	4.35		523.40
12	110.125	333.25	258245.95	0.16	4.35	4.35	479.14
11	100	333.25	225301.95	0.14	3.80	8.70	379.58
10	89.875	333.25	193715.33	0.12	3.26	12.50	293.32
9	79.75	333.21	163555.00	0.10	2.76	15.76	219.75
8	69.635	333.25	135010.01	0.08	2.27	18.52	158.39
7	59.5	380.56	123414.56	0.08	2.08	20.79	123.72
6	50	386.63	98024.68	0.06	1.65	22.87	82.57
5	40.5	386.63	72751.46	0.04	1.23	24.52	49.64
4	31	386.63	49839.04	0.03	0.84	25.75	26.03
3	21.5	386.63	29695.68	0.02	0.50	26.59	10.76
2	12	388.90	13088.12	0.01	0.22	27.09	2.65
Ground	0	368.22	0.00	0.00	0.00	27.31	0.00
			1620991.54				

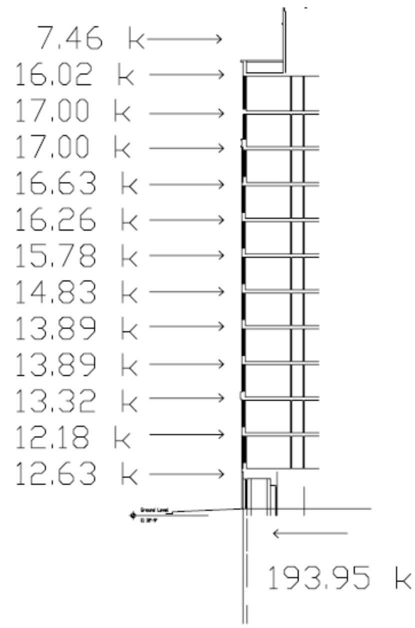
Total Building Weight (kips)	4644.75
Overturning Moment	2348.95

Table 5

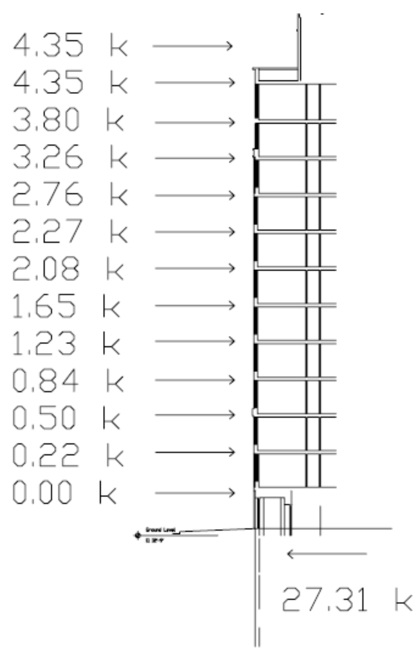
Story forces must be found. Story forces are the forces which will act on each floor and are called F_x in the wind and seismic load tables.



N-S wind forces



E-W wind forces



Earthquake Forces

Figure 21

To determine the worst case lateral load experienced by The Lexington, the load combinations below from ASCE 7 were analyzed.

$1.4D$
$1.2D + 1.6L + 1.6L_r$
$1.2D + 1.6L_r + (L \text{ OR } .8W)$
$1.2D + 1.6W + .5L + .5L_r$
$1.2D + 1E + .2S$
$.9D + 1.6W + 1.6H$
$.9D + 1E + 1.6H$

It is obvious from observation of the story forces, that the wind loading is the more critical loading case for The Lexington. In the above load combinations, the wind load will have an additional increase of 1.6 while the earthquake load would remain the same with a factor of 1, making the wind load case all the more critical. The controlling load combination for the new design of The Lexington in both directions is:

$$1.2D + 1.6W + .5L + .5L_{r.}$$

Distribution of Loads:

The loads were distributed to each frame based on rigidity. The rigidity of the lateral elements is affected by their member sizes, moments of inertia, and geometry. The first consideration in picking a braced frame shape is the building's architecture. However, the proposed frames will not obstruct any doors or windows and can be used with any frame geometry. A simple analysis in STAAD was run to compare the rigidity of braced frames. This resulted in the X brace as having the greatest stiffness, followed by the chevron, single diagonal and finally inverted chevron. Using an X or K frame will result in more connections and may therefore be more costly if the extra stiffness is not needed. The first braced frame model analyzed was for chevron frames in only the frames around the elevator core. The two frames spanning east to west will be identical to each other, and the north- south frame will only differ due to length. Distribution by rigidity also depends on the distance of the frame to the center of rigidity. Because there is only one frame spanning N-S, it will be the x coordinate of the center of rigidity. The y coordinate will be directly between the two walls spanning the e-w direction since they have the same stiffness as each other.

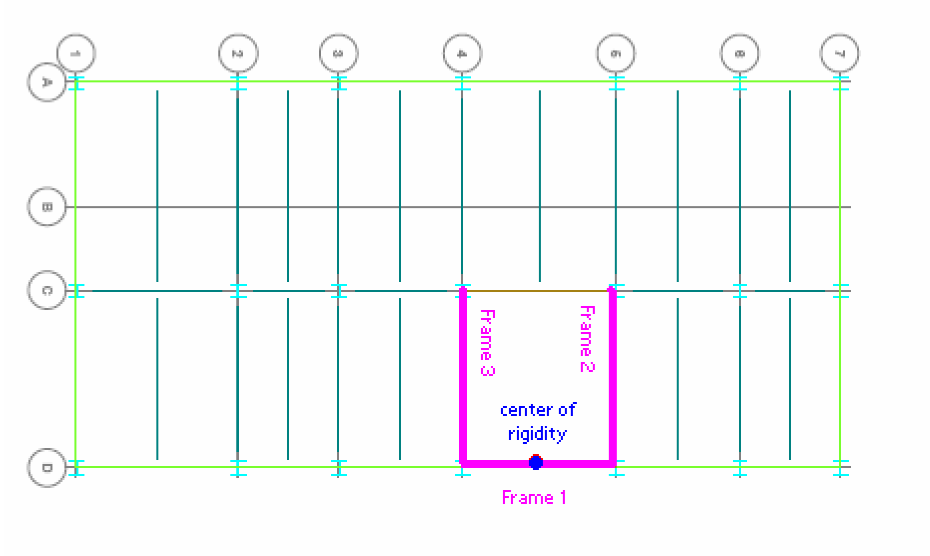


Figure 22

N-S

F_{direct}

Frame 1 = 100%$F_{n\text{-}s}$
Frame 2 = 0%$F_{n\text{-}s}$
Frame 3 = 0%$F_{n\text{-}s}$

$F_{torisonal}$

Frame 1 = 0
Frame 2 = 5%M
Frame 3 = 5%M

E-W

F_{direct}

Frame 1 = 0% $F_{e\text{-}w}$
Frame 2 = 50%$F_{e\text{-}w}$
Frame 3 = 50%$F_{e\text{-}w}$

$F_{torisonal}$

Frame 1 = 0
Frame 2 = 5%M
Frame 3 = 5%M

When wind loading is the controlling lateral load case, ASCE 7 prescribes 4 wind cases which should be checked for each building. These 4 load cases vary by percent of wind load acting on the building, and by eccentricity. All 4 cases have been checked; case 1 is the controlling wind case for frame 1, and case 3 is the controlling load case for frames 2 and 3.

Solution:

A rough estimate of the stiffness of the building can be computed based on allowable deflection. The deflection criteria used for buildings is called the drift index, where Δ/Story Height. It is common practice to use allowable Δ = H/400. For the new height of The Lexington,

$$H/400 = 120.25/400 = .3 \text{ ft or } 3.6 \text{ inches}$$

Using allowable drift, the stiffness needed in each frame can be calculated.

$$\Delta = \text{Story shear}/\text{Stiffness}$$

$$K = AE\cos^2\theta / L$$

By solving for stiffness and then for area, a rough size for the bracing members was determined. Using STAAD, the frame was modeled and analyzed for the wind load case. The wind load was applied as a point load at each level. Because ASCE 7 wind load case one controlled, the story forces were as calculated in the excel wind load spreadsheet. (For the E-W frames, the story loads are taken at 75% but applied in both directions creating a larger moment to be resisted, in compliance with ASCE 7 wind load case 3).

To design the braced frames, the allowable stress on each member as well as the overall deflection of the frame must be considered. By running a model in STADD, the average stresses and hence axial loads in the columns can be found as approximately 1200 kips. The columns must be able to support both this wind load, as well as the load contributed by the gravity loading. The column is then sized by Table 4-2 of the LRFD manual, design strength in axial compression. This same method of sizing beams is applied to each member in the frame, starting at the top and working downward. The frame is then re-tested in STADD for the wind load. The final design is:

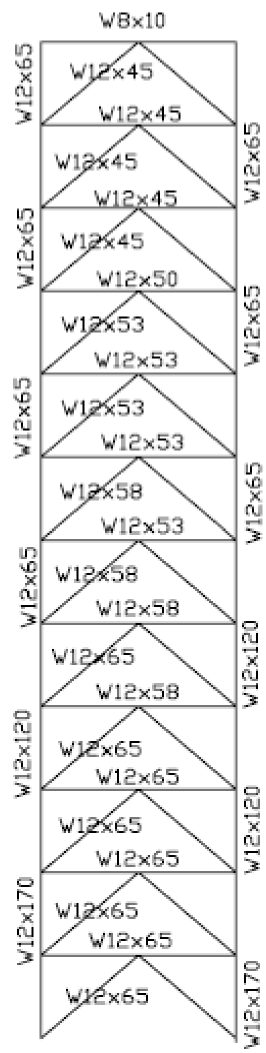


Figure 23
The frame is symmetric, any beam not labeled is the same as its counterpart.

The same frame design is also being used in the N-S direction. Although the N-S direction has smaller loads acting on the frame then the E-W direction, the change in member size is slight compared to the benefit of using repetitive members.

Before this frame can be used, the columns' members must be checked for biaxial bending since two of the columns are used in both E-W and N-S frames. In these columns, biaxial moment will control and there will be bending around the weak axis as well as the strong axis. To design these columns the AISC code was used with equation H1-1b: $\frac{Pu}{2\phi Pn}+\left(\frac{Mux}{\phi Mnx}+\frac{Muy}{\phi Muy}\right)\leq 1$. The final column designs for the biaxially loaded columns are much larger.

Story	Design
1	14 x 342
2	14 x 342
3	14 x 311
4	12 x 336
5	12 x 305
6	12 x 279
7	12 x 252
8	12 x 230
9	12 x 190
10	12 x 152
11	12 x 106
12	12 x 65

Table 6

The frame has total deflection of 2.7", which is less then the allowable 3.6".