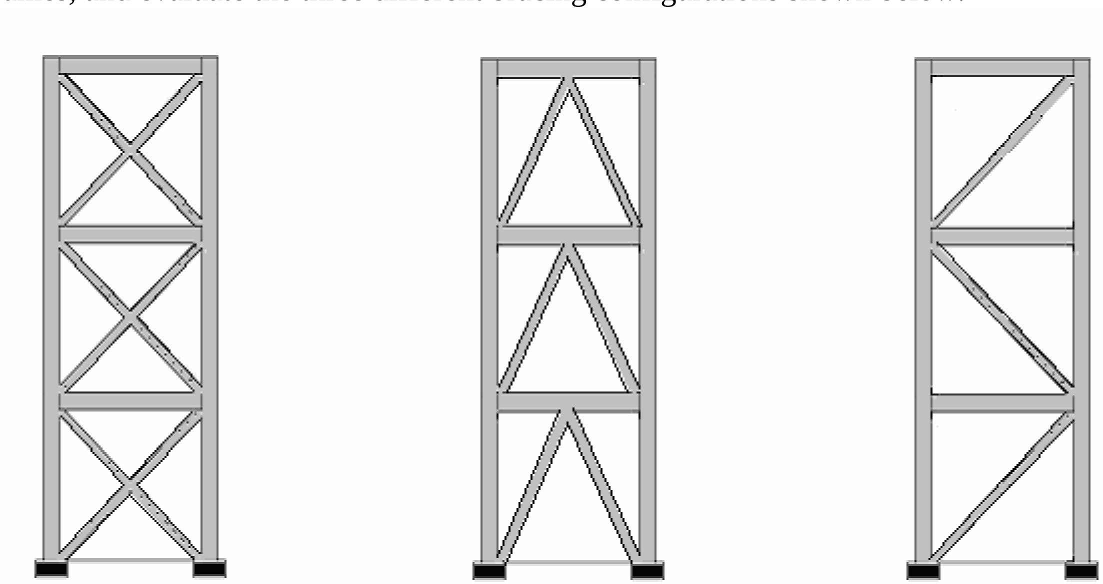
Structural Depth Study

During my work in analyzing this structure, I have come to the conclusion that the best way to further engineer this construction is to attempt optimization. Revisiting the lateral system design and performing a vibration analysis on the gravity system will establish my new performance criteria for a structurally sound and efficient design.

Lateral System Redesign

My first analysis will center on the redesign of the existing lateral system. Currently, lateral resistance is provided by a system of concentrically braced steel frames located on all four faces of the structure. The braces are designed as HSS members acting to resist forces in tension only. For the redesign, I will continue to use steel frames, and evaluate the three different bracing configurations shown below.



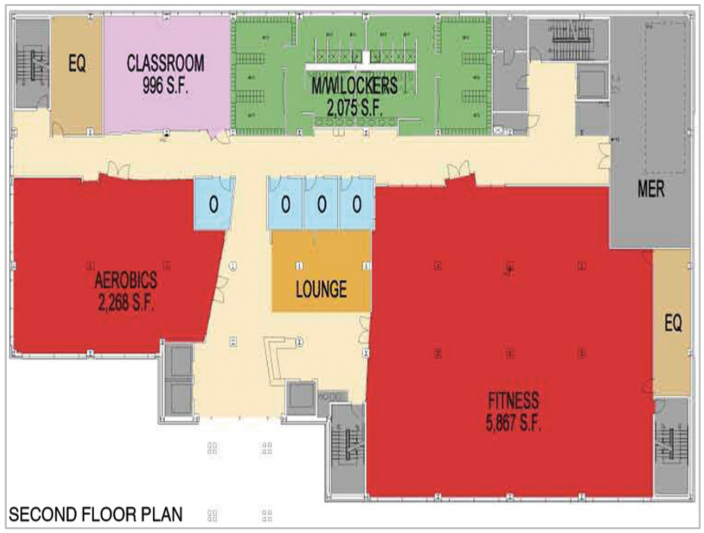
X-Bracing (Tension and Compression) | Chevron Bracing | "K"-Bracing

In performing a lateral analysis during Technical Assignment 3, the frames were checked based on the assumption that the diagonal braces were designed to take the full lateral force in tension. If the frames were designed under this assumption, then it is possible that the diagonal braces are not being used to their full potential. If this is true, then a redesign of the lateral system could result in lighter, more efficient structure.

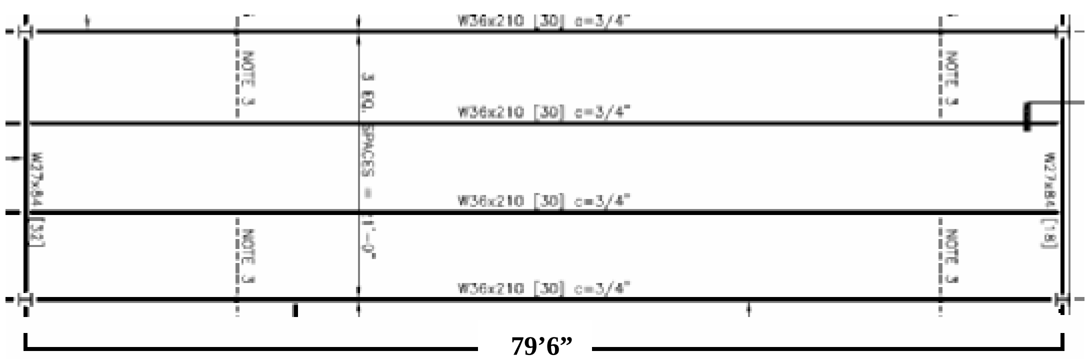
Previous research from Technical Assignment 3 yielded high torsional forces acting on the upper stories of the structure. After verifying my calculations, I will investigate possible solutions to reducing these torsion forces. Also, analysis results from RAM Frame software will be further scrutinized as a further confirmation of my calculations.

Gravity System Analysis

My second analysis will focus on the gravity framing system. As constructed, the floor framing is composite steel wide flange beams and girders. Throughout the building, different floor areas are used for a variety of activities. Some areas that are used for aerobic or athletic activities are near (above/below/next to) office, classroom, or retail spaces. I will analyze these athletic spaces based on acceptable vibration criteria and make changes accordingly.



Along with intermixed activities, the fourth floor gymnasium and fifth floor ballroom are framed with long spanning members, and may be more susceptible to unacceptable vibration conditions. These conditions will be analyzed just as the other spaces listed above.

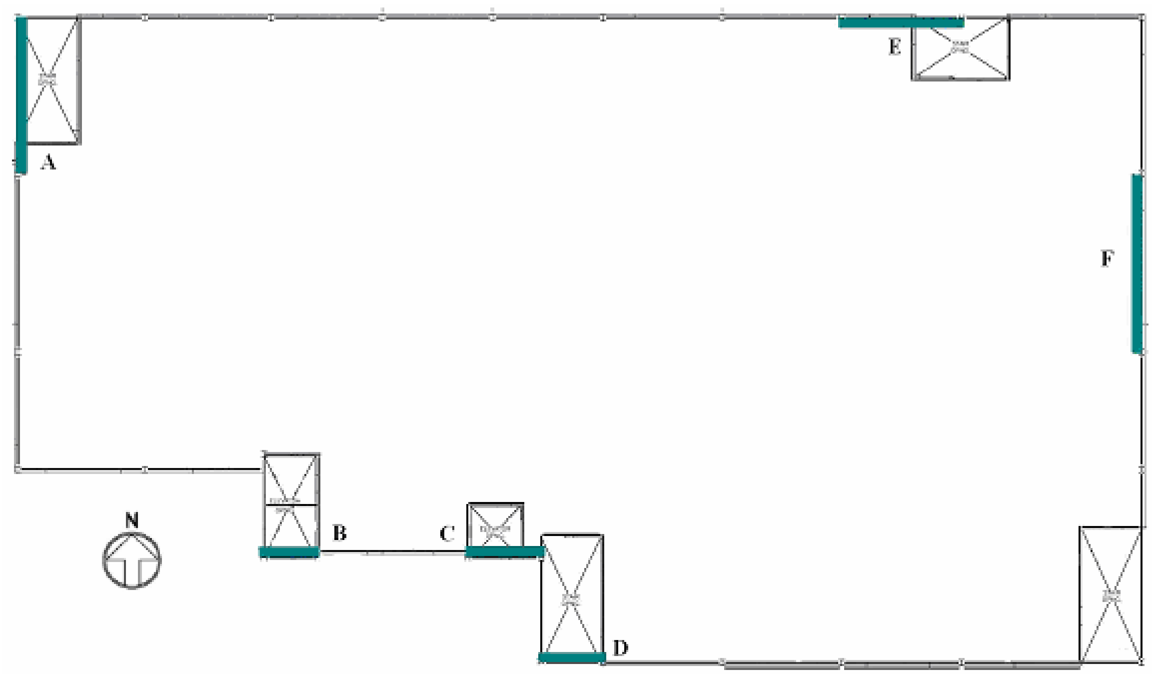


Within all of the previous research topics is the subject of optimizing each component of the structural system. Therefore, the overall goal of my research will be to create a more cost effective and structurally efficient building, without reducing the quality or efficiency of the other building systems.

Existing Lateral System

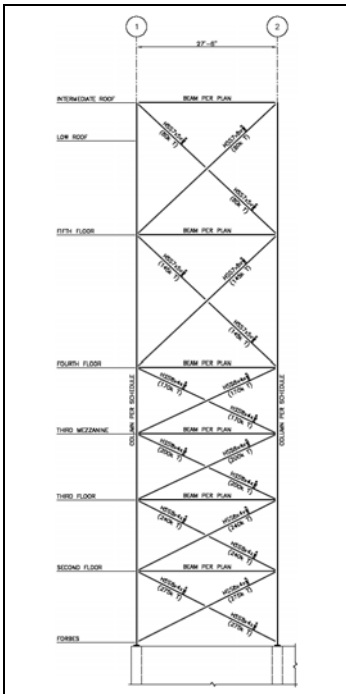
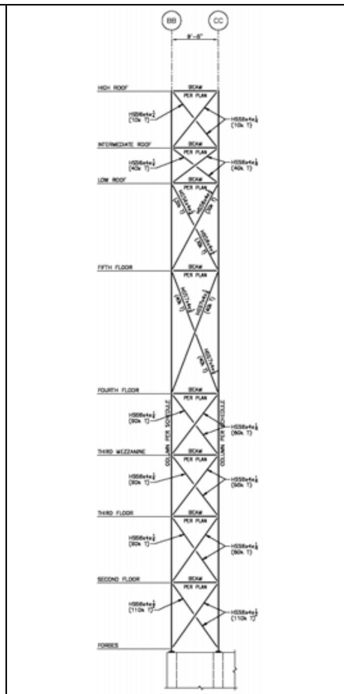
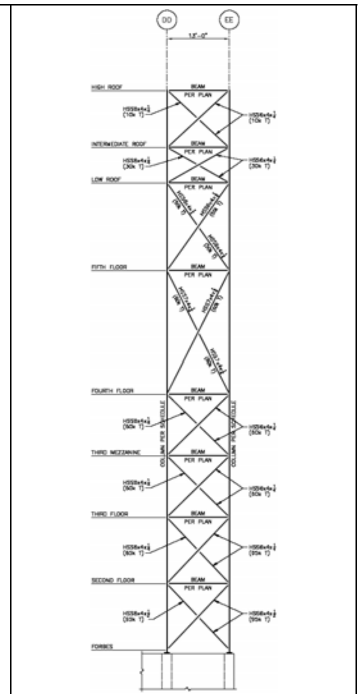
The Duquesne University Multipurpose Facility uses concentrically braced steel frames to resist lateral loads. Each lateral element or frame is located along the perimeter of the structure (as shown below). The upper level interior spaces, gymnasiums and ballroom, are not as favorable for lateral elements because they require so much open space. Exterior locations such as stair wells and elevator cores lend themselves as unobstructed positions for the braced frames. These areas are devoid of windows and other openings allowing the frames to be well hidden from view. Where other frames are needed, exterior elevations without windows or openings were again chosen to hide these elements.

On the South face of the building, frames are constructed around both elevator shafts and a stair tower. The same is true on the North and West faces of the building where bracing is positioned at stair towers. The typical columns used in each of bracing elements are W14's ranging from W14x53 to W14x132. Each floor to floor section makes use of a series of concentrically braced HSS members ranging in size from HSS6x4's to HSS8x4's, 1/4" to 5/8" thick. Each bracing member is designed to see 30 – 275 kips in tension only.



Letters correspond to the elevations on the following page

Existing Braced Frame Elevations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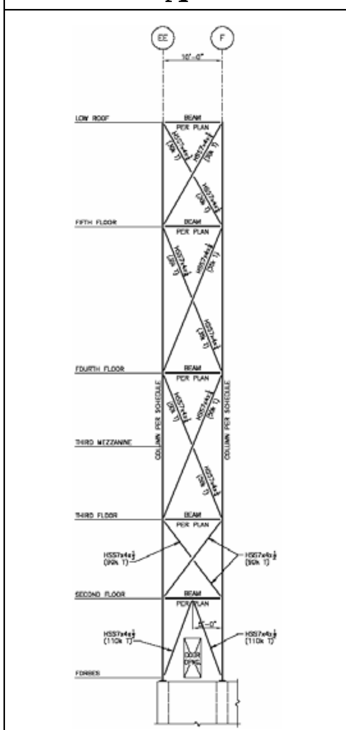	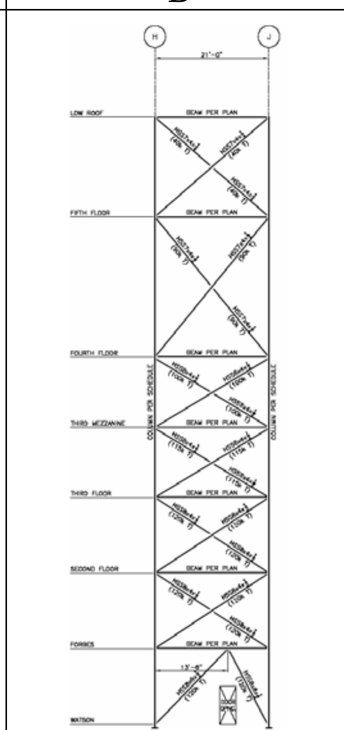	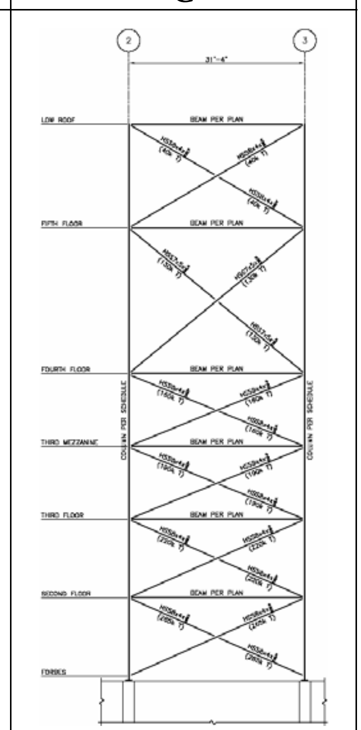
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Design Criteria

Building Code: International Building Code, IBC 2003
Referencing ASCE 7-02

Structural Concrete: Code Requirements for Structural Concrete, ACI 318
Specifications for Structural Concrete, ACI 301

Structural Steel: Manual of Steel Construction
AISC, 13th Edition LRFD/ASD

Applicable Loadings: Gravity Loads

Live Loads (ASCE 7-02, Table 4.1)

Lobbies and Public Spaces………….............100 PSF
Corridors (above first floor)………………...80 PSF
Mechanical…………………………………75 PSF (assumed)
Athletic Floors………………………………100 PSF
Stairs and Exits…………………………….. 40 PSF
Offices………………………………………50 PSF

Dead Loads

Partition Allowance……………………….. 20 PSF
Reinforced Concrete Slab………………….. 150 PCF
Curtain Wall System……………………….. 15 PSF
MEP……………………………………….. 5 PSF
Metal Decking…………………………….... 2-3 PSF
Joist/Beam Weight…………………………Specific to each member

Snow Loading (ASCE Section 7, Figure 7.1)

Ground Snow……………………………….30 PSF
Flat Roof Snow…………………………….. 21 PSF
All other factors = 1.0

$p_f = 0.7CeCtIp_g = 0.7(1.0)(1.0)(1.0)(30psf) = 21$ PSF

Applicable Loadings: Lateral Loads

Seismic Loads (ASCE7-02)

Seismic Design Category…………………….. A
Seismic Use Group…………………………. II
Importance Factor (I_E)…………………….. 1.25
S_S……………………………………………0.128
S_1……………………………………………0.057
S_{DS}…………………………………………..0.102
S_{D1}…………………………………………..0.065
Site Class……………………………………C
Response Coefficient
N-S…………………………. 0.0231
E-W…………………………. 0.0231
Response Modification Factor
N-S…………………………. 5
E-W…………………………. 5

While seismic forces were calculated during the initial lateral analysis of the building, they will not control the lateral design. Under IBC2003 section 1616.6, it states that an analysis must be performed except when structures are assigned to Seismic Design Category A, which includes this structure. However, when the seismic classifications of the building were entered into RAM, the result was that the seismic forces typically did not control the design of the members. The lateral forces from the wind caused the highest stresses in the lateral system. In the end, the controlling design factor for the lateral system was mostly drift of the structure, not stress.

Wind Loading (ASCE 7-02)

Basic Wind Speed…………………………..90 MPH
Exposure Category………………………….III
Enclosure Classification…………………….Enclosed
Building Category…………………………..B
Importance Factor………………………….. 1.15
Internal Pressure Coefficient………………..0.18

Base Shear (N/S): 435 kips
Overturning Moment: 26845 ft-kips

Base Shear (E/W): 219.1 kips
Overturning Moment: 13640 ft-kips

Duquesne Multipurpose Facility Story Forces (Kips)					
		Hand Calculations		RAM Output	
Level	Height	Wind (x-direction)	Wind (y-direction)	Wind (x-direction)	Wind (y-direction)
High Roof	120	2	11	2.07	7.46
Int. Roof	108	16.2	29.1	10.24	27.95
Low Roof	100	31.8	48.3	29.83	-3.37
5	80	50.7	109.6	51.31	138.45
4	54	41.4	82.8	44.04	80.82
Mezzanine	41	27.4	52.2	9.24	50.65
3	28	25.7	53.2	38.83	50.47
2	14	23.9	48.8	22.29	47.43
Forbes Avenue	0	0	0	33.7	7.27
Base Shear (k)		219.1	**435**	241.55	**407.13**
Overturn Moment (ft-k)		13,638.8	**26,845**	12,598.42	**23,170.91**

The lateral loads imposed on the building are distributed into story forces and then further distributed to each frame on the basis of relative stiffness. Because there seems to be no practical way to reposition the existing frames, the existing locations will be used in the redesign. Leaving the frames in place also will allow for a more direct comparison between the different bracing configurations.

Analysis Methods

Because the original frames were designed using allowable stress design, I will use ASD combinations to check the new frames. Using allowable stress analysis will allow for a more direct comparison between the existing frames and the alternates. The following combinations were checked:

- D + L
- D + (W or 0.7E)
- D + 0.75L + 0.75S
- **D + 0.75L + 0.75W   **Controls****
- 0.6D + W

After finding the controlling load combinations, RAM Advanse was used to analyze and design each individual frame. RAM's "Optimize Model" command was used to determine the new member sizes in the alternate bracing configurations. The optimize/code check commands choose the appropriate members based on multiple analytical iterations, selecting a member with adequate strength and minimal weight.

Upon completing the individual frame design, each alternate system was checked using RAM Structural System's Frame module.

Torsion Revisited

During previous study, the question of excessive torsional forces arose. Hand calculations suggested that the excessive forces were confined to the upper 3 stories of the building. During the analysis, the relative stiffness of the each full frame was considered. In doing so, the extra, one story, frames for the intermediate and low roofs were omitted. The omissions of these frames are a possible reason that the torsional forces at the upper stories were calculated to be so large.

When recalculating the torsional forces associated with the upper stories of the structure, including the stand alone frames (for the intermediate and low roof levels) dramatically reduced the previously calculated forces. These new forces will be included with the existing shear in the redesign of each lateral frame. Because the torsion forces have turned out to be relatively small when compared to the wind forces imposed on the structure, I expect them to have only a small impact on the overall design of the alternate systems.

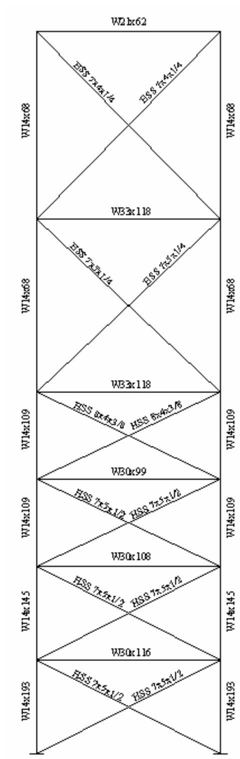
Lateral Alternates

Information contained in the following pages includes:

- Frame Elevations
    - Alternate #1: Modified Concentric Frames
    - Alternate #2: Chevron Bracing
    - Alternate #3: K – Bracing

- Lateral Analysis Results (found on page 17)
    - Alternate frame weight comparison
    - Alternate frame drift comparison
    - Conclusions

Alternate #1: Modified Concentric Frames



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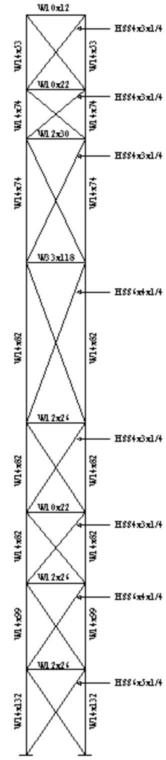

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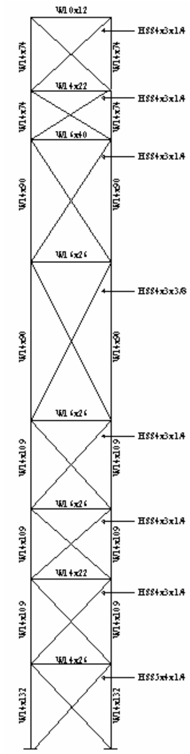

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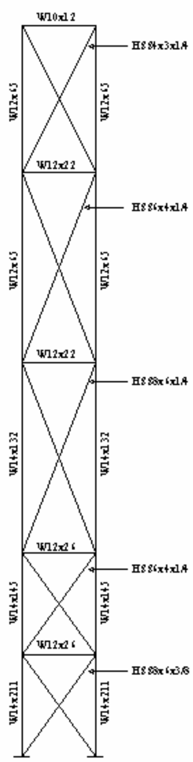

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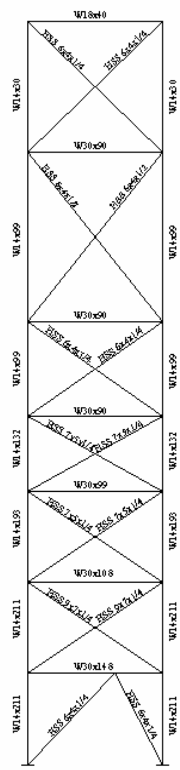

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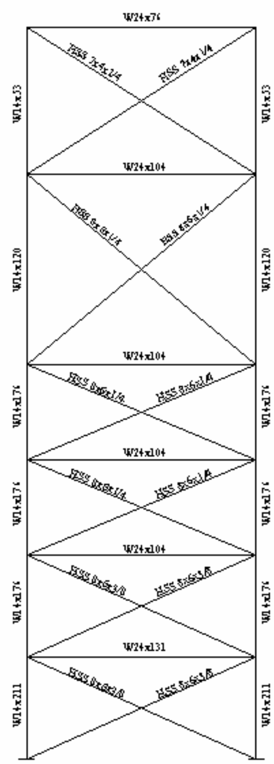

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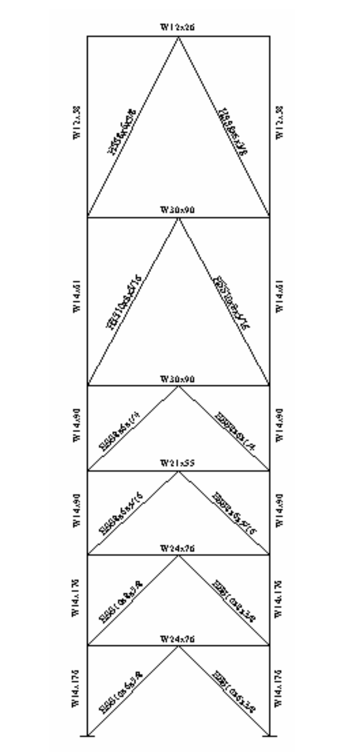
Alternate #2: Chevron Bracing



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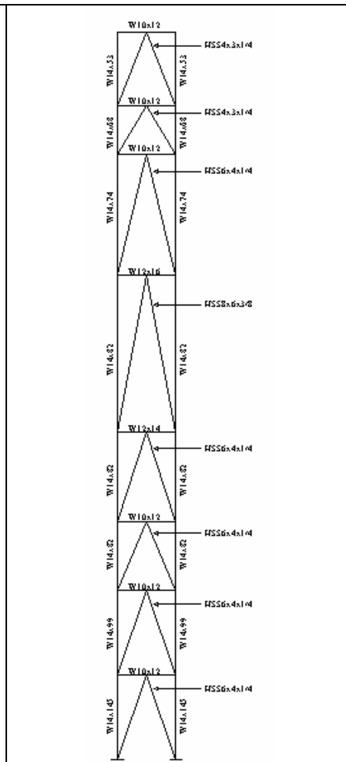

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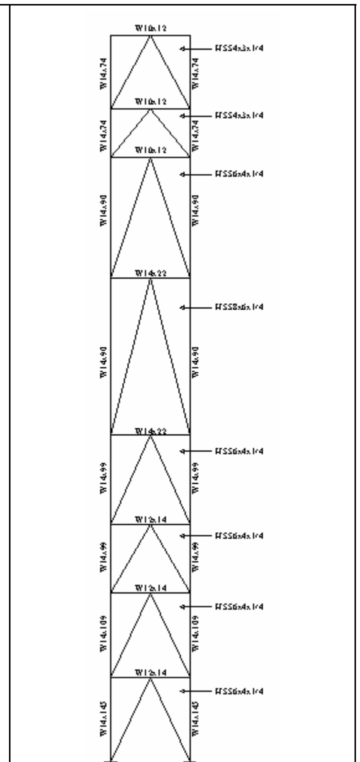

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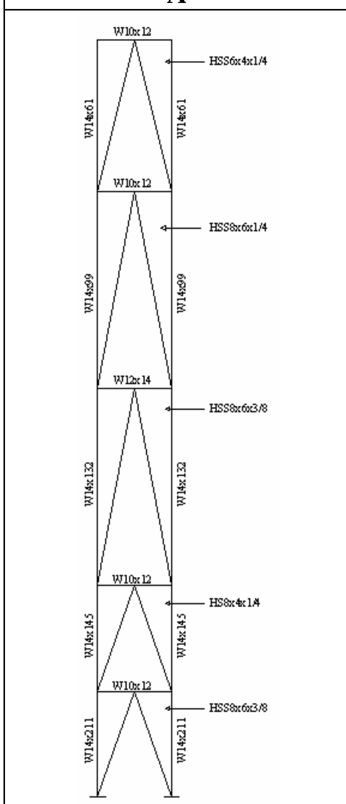

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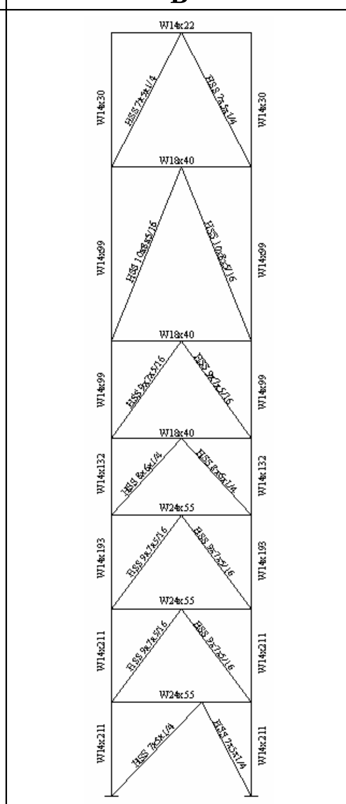

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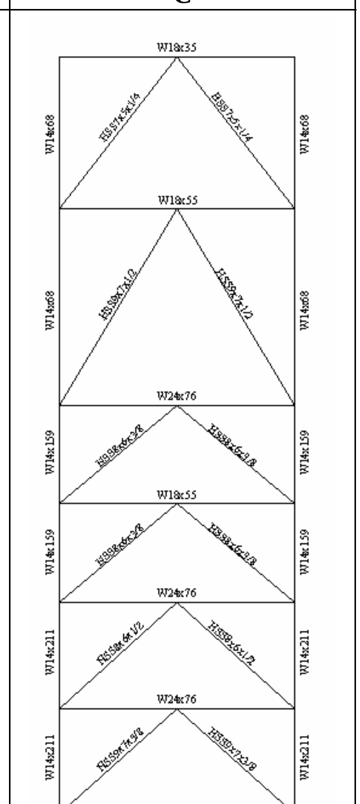

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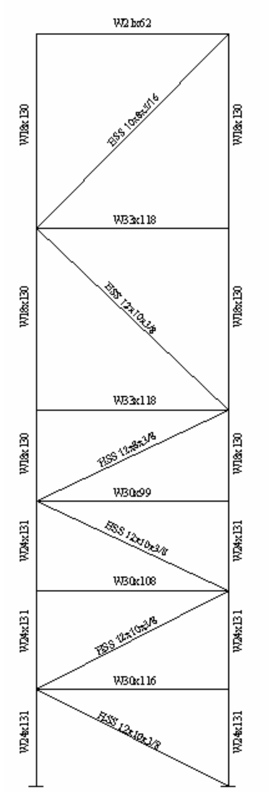
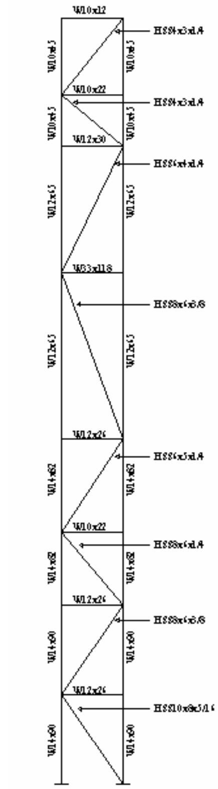
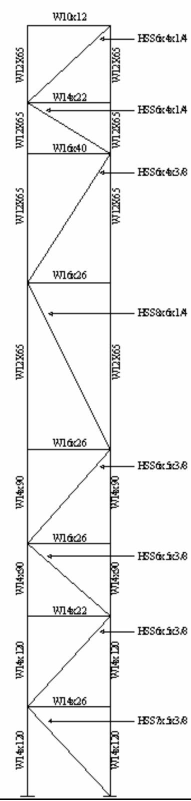
Alternate #3: K-Bracing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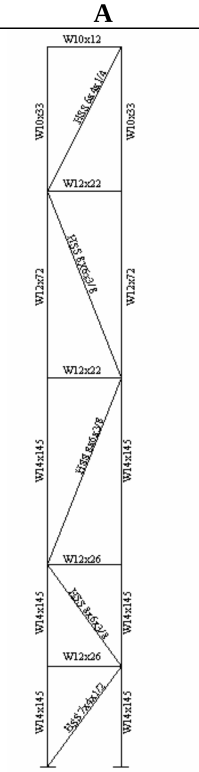	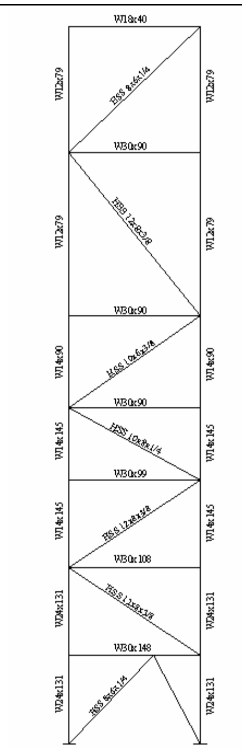	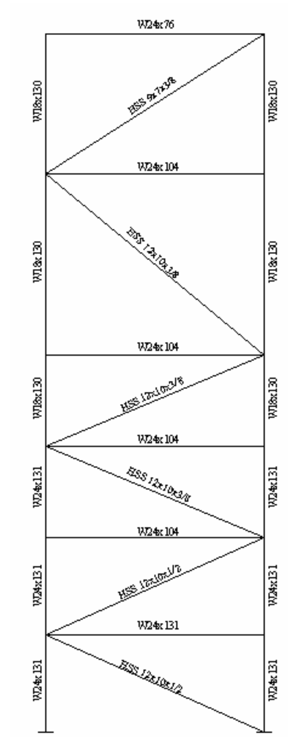
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Lateral Analysis Results

Lateral System Weight Comparison (kips)			
	Component weights		
Bracing Layout	HSS Braces	W shapes	Totals
X-Bracing (T only)	46.5	198.7	245.2
X-Bracing (T-C)	38.4	202.5	240.9
Chevron Bracing	37.9	178.2	216.1
K-Bracing	37.4	192.4	229.8

Overall Building Drift (in.)				
Bracing Layout	Drift @ HR	Drift @ IR	Drift @ 5th	H/400
X-Bracing (T only)	5.6	3.3	2.1	3.96
X-Bracing (T/C)	4.6	2.6	1.5	3.96
Chevron Bracing	4.8	2.7	1.5	3.96
K-Bracing	5	2.9	1.6	3.96

The results of my analysis indicate that the chevron bracing scheme is clearly the lightest of the four systems studied. While the HSS bracing members are of a similar weight in each alternate system, the wide flange beams in the chevron configuration are able to be dramatically reduced. This reduction is possible because each set of braces halves the span of each beam. Each beam supports the masonry façade, and thus its design is controlled by masonry deflection limits (L/600 or 0.3”) and not shear or flexural stress. Initially, I had concerns that the beam sizes would increase due to added shear stresses caused by the chevrons distributing their forces into each frame. In this case however, the beams are oversized leaving most members at approximately 30% of their shear capacity.

One concern that arose in conjunction with all four designs was that of overall building drift. A bridge structure connects this facility to an adjacent parking structure at the 5th floor/ballroom level. The original construction documents call for the two structures to be kept separate by a minimum 1” expansion joint. At the 5th floor level, the minimum deflection (of the four systems) was found to be 1.5”, as can be seen above. Although it would be unfavorable if the building was to push or lean against the bridge, one would assume that an extra 1/2” of drift would not be cause for great concern. In addition, the building drift is calculated for a worst case wind loading scenario and would not likely happen often enough to cause damage or undo stresses in the bridge structure.

Another drift question arises at the HR level. At this height, the practical drift limit of H/400 is exceeded by 0.6”-1.6”. This seems to be of no consequence due to the following:

- H/400 is an accepted practical standard and not part of any structural code
- The HR level is part of an “atrium” space, and unoccupied