COMPARISON OF STRUCTURAL STEEL LATERAL FORCE RESISTING SYSTEMS FOR A THEORETICAL HOSPITAL GRID SYSTEM

by

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Abstract

In 2006, a research project was being carried out by architects at architecture/engineering firm Cannon Design involving an optimum bay size for a hospital. RISA computer modeling was used to explore a set of lateral force resisting system (LFRS) options for a building based on this optimum bay size and importance category. The structural material was first narrowed down to steel, and then moment frames and braced frames are examined. The LFRS was narrowed down to braced frames, discarding moment frames due to their inordinate story drift. Of the different types of braced frames, the study further narrowed the LFRS system to chevron braced frames. Then the precise arrangement of braces for a particular building size using this bay system was examined. The steel material cost of the final system was compared to a system that only included members sized for gravity loads to demonstrate the rough amount of cost that a lateral system can add to a building.

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CHAPTER 1 - The Hospital Grid System

In summer 2006, the structural engineering division at the St. Louis office of Cannon Design, an architecture and engineering firm with 13 offices worldwide, worked closely with the architecture department on several research initiatives. One of these was based on exploring the concept of creating a prototypical hospital bay. This is referred to as a *universal grid*. This report will explore a set of structural design issues relating to designing a building with this universal grid – specifically, it will analyze a group of lateral force resisting systems and determine the best one for this application. This section will explore this project's purpose, the criteria used in the project, the dimensions determined by these criteria and possible building variations that can affect this system.

Architectural Purpose

The purpose of the *universal grid* is to maximize design efficiency as well as space use flexibility for a variety of applications. The benefits of such a grid include the reduction of design time and cost through the repetition of design elements. Architects will be able to use the *universal grid* as a starting point for their designs (Cannon Design 8).

Criteria

According to Cannon Design architect Natalie Petzoldt, the primary criteria for determining the properties of the *universal grid* were American Institute of Architects clearance requirements for various hospital spaces and pieces of equipment. A standardized bay size for a hospital floor giving the required clearance between walls for a variety of space applications, such as patient rooms, operating rooms, special procedure rooms, offices and others was the goal (Petzoldt, 2006). Within this standardized bay size, the structural columns should fit within the walls - the wall clearances required dictate the column spacing in the structural grid. The structural design of the LFRS carried out in this report uses a lateral drift limit of frame height/400 (h/400), which does not consider the interaction between the structural system and the exterior cladding, whether it be glass, masonry or any other material. The architects at Cannon determined the optimum bay size to be 31.5 feet by 31.5 feet (Cannon Design 9). Figures 1.1, 1.2, 1.3, 1.4 and 1.5 show various space applications considered for the grid system, and how these applications fit in the 31.5 foot bays.

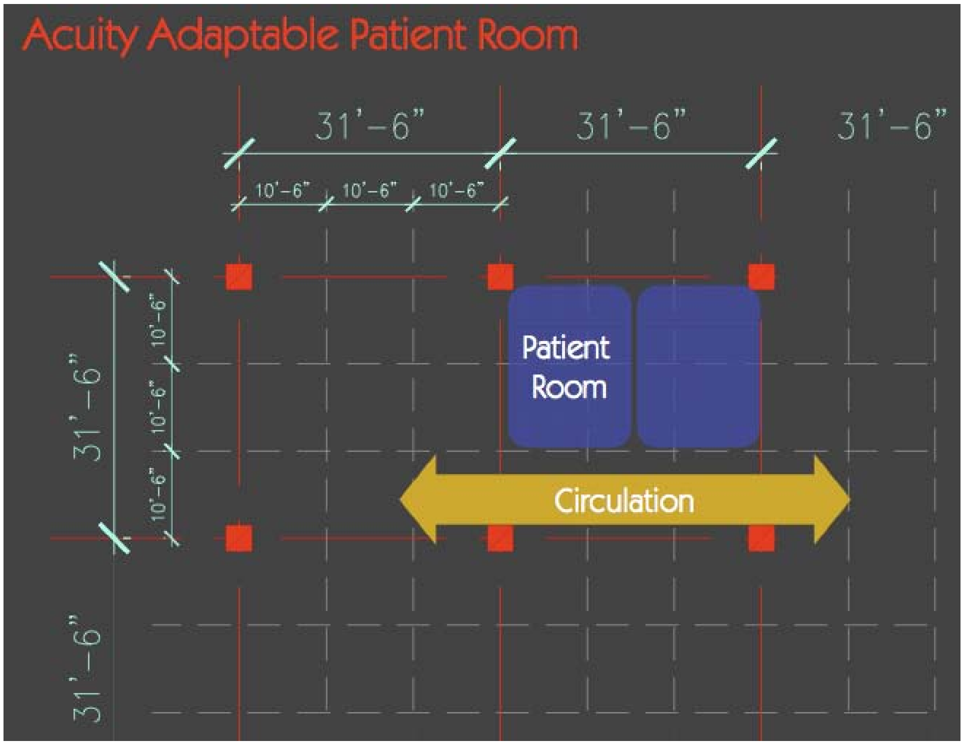

Figure 1.1 Patient Room Diagram (Cannon Design 13)

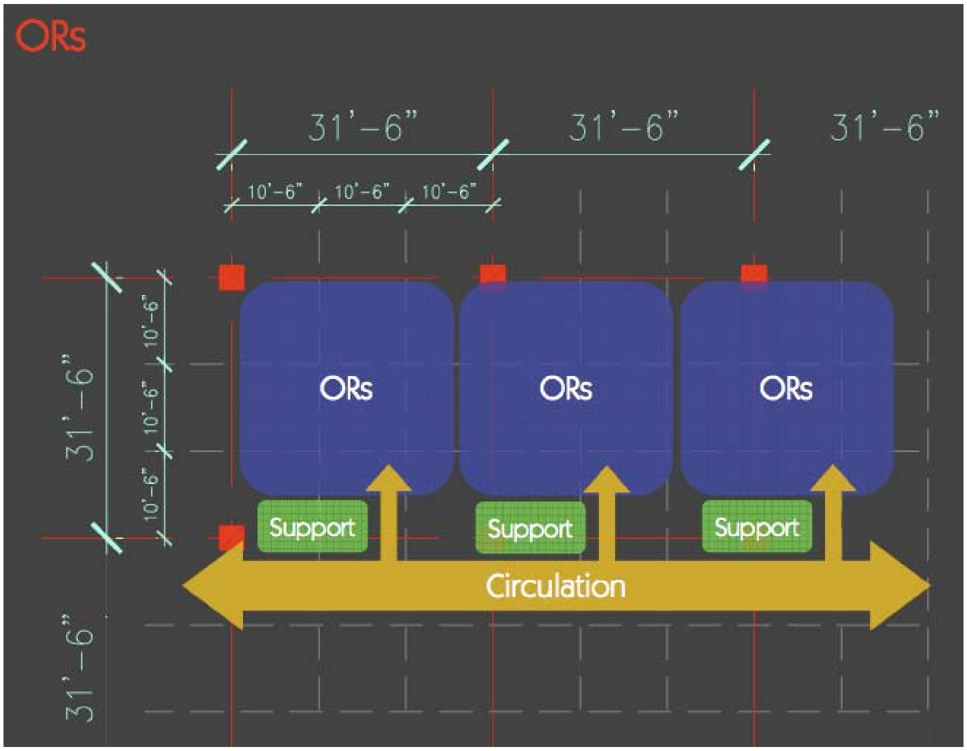

Figure 1.2 Operating Room Diagram (Cannon Design 14)

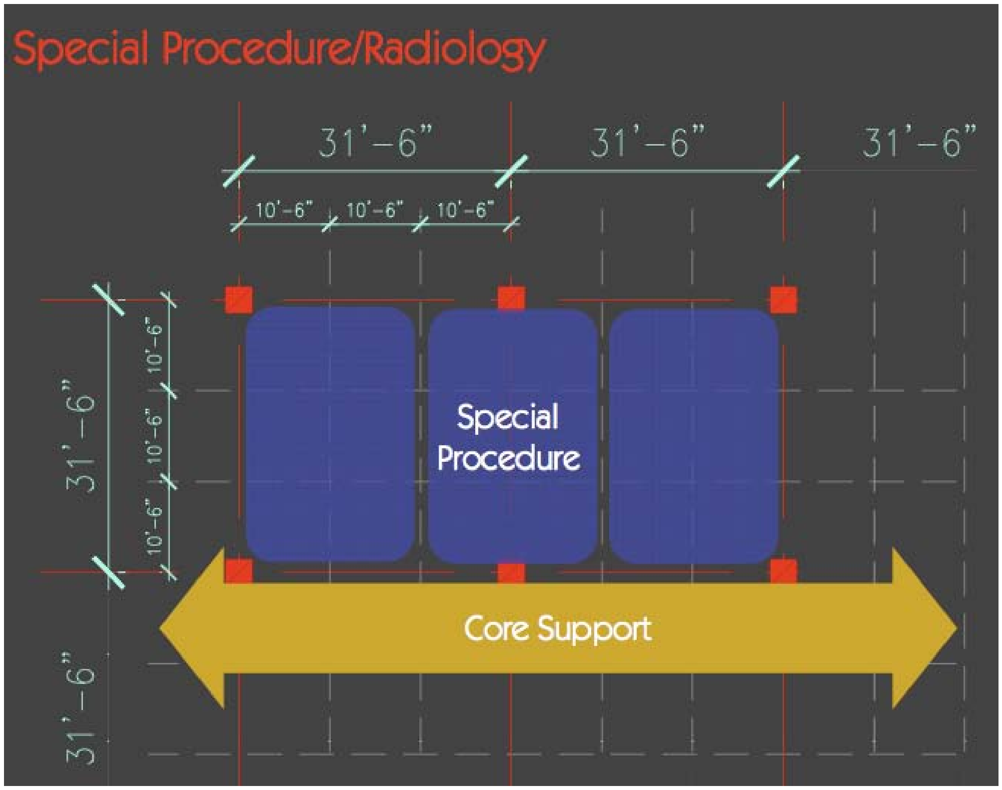

Figure 1.3 Special Procedure Room Diagram (Cannon Design 15)

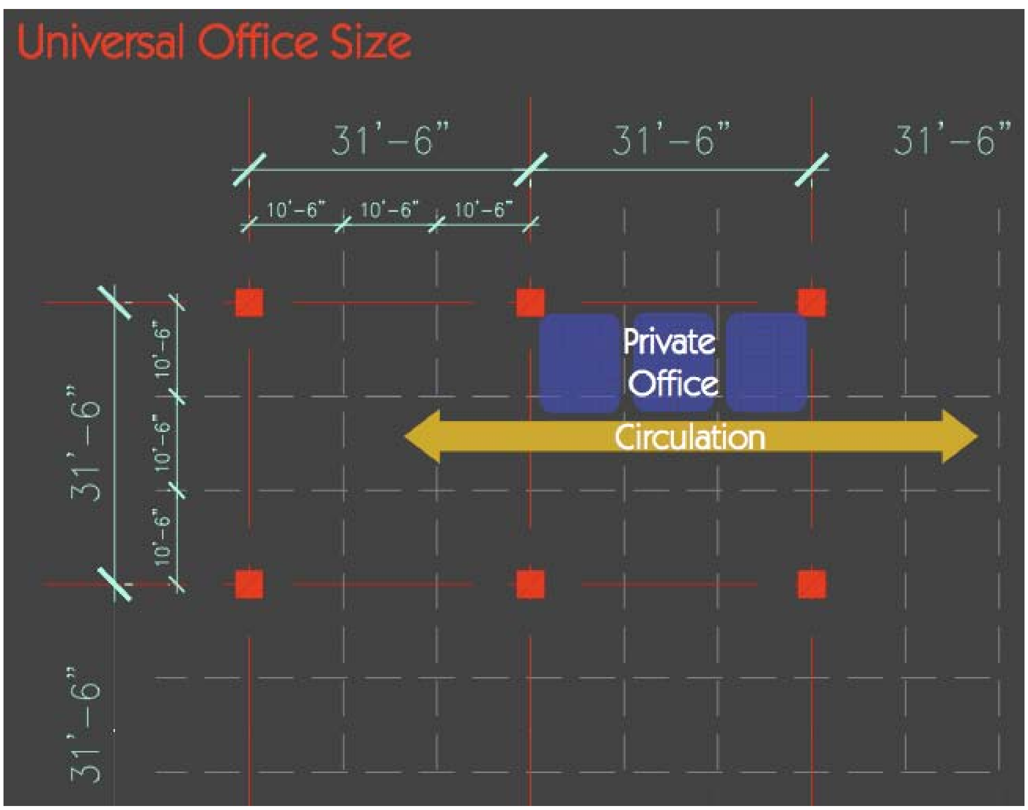

Figure 1.4 Office Diagram (Cannon Design 16)

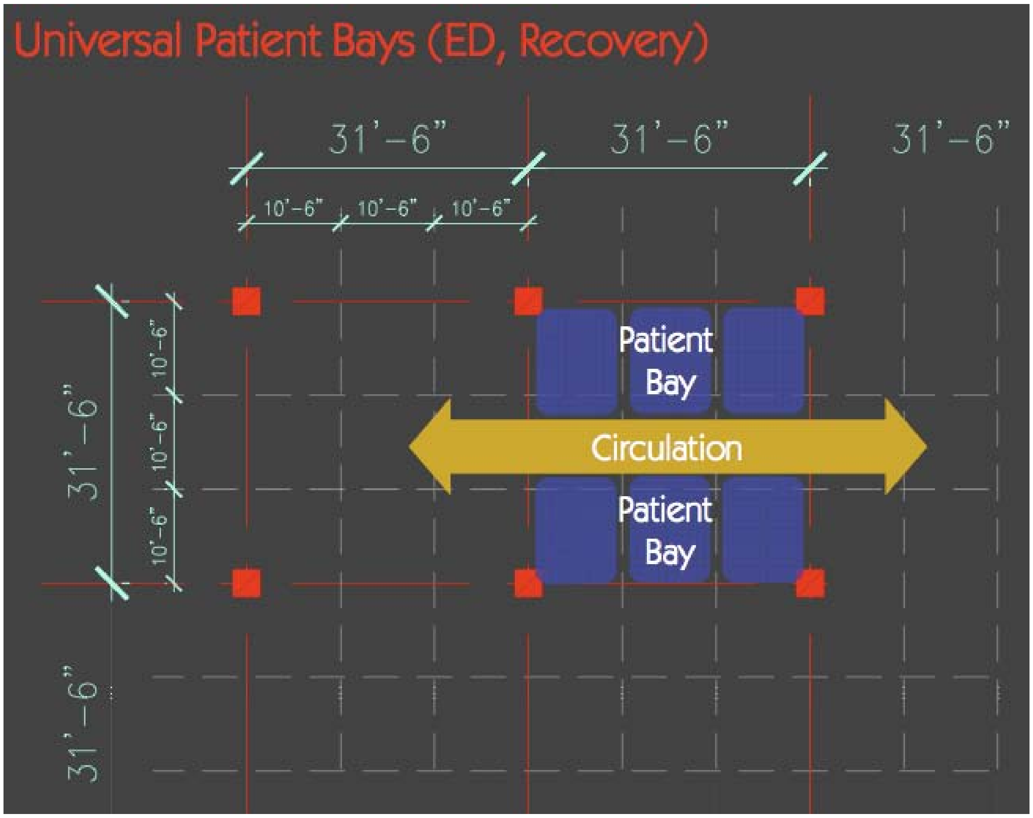

Figure 1.5 Patient Bay Diagram (Cannon Design 17)

Dimensions

For this report, computer building models were constructed using a 31.5 foot by 31.5 foot bay throughout the structure. In addition to a standard bay width and length, a standard vertical dimension (floor-to-floor height) was required in the *universal grid system*. A standard floor-to-floor height is more difficult to determine, as different types of spaces could require vastly different amounts of plenum room for mechanical equipment while the depth required for the structural system would stay approximately constant. The Cannon architects have proposed an 18 foot floor-to-floor height as a starting point (Cannon Design 9). This number is generous for the mechanical plenum space and therefore, conservative for structural design. Structurally, an 18 foot floor-to-floor height is considered difficult to engineer cost effectively. The high floor-to-floor height results in high slenderness of the structural frames, which puts a strain on their ability to transfer the lateral load while minimizing story drift, especially when using moment frames. The higher up the lateral load is, the larger the resulting moment going into the frame, and in order for the frame to distribute the load, the moment needs to be resolved into a couple force at the supports. The closer together these supports are, the higher the couple forces will have to be. Thus, the higher up the lateral load point is, and the closer together the supports are, the more the frame is loaded. This ratio of frame height to frame width is the “slenderness” of the frame.

The roof and floor plans used for the models in this report are shown in Figures 1.6 and 1.7. Figure 1.6 shows the structural roof plan for the hospital model. It consists of structural steel beams and open-web roof joists for the roof system arranged on the standard uniform hospital grid of 31.5 feet by 31.5 feet. Figure 1.7 depicts the structural floor plan, composed of similar structural elements but with tighter joist spacing due to higher floor live loads. Table 1.1 explains the meanings of the beam and joist labels used in the models.

The building columns will be considered spliced every 2 stories, or 36 feet, in order to both keep their lengths to a manageable size while still minimizing the number of column splices necessary. Therefore, individual columns are to be 2 stories tall, and this will affect the column sizing carried out in this project, as each column will carry two stories worth of vertical load and lateral load. Figure 1.8 depicts the vertical column labeling scheme used to identify columns on different floors, and Table 1.2 explains the meanings of the column labels used in the models.

Variables

Variables that could change among hospital projects affecting the application of this *universal grid* system are: preferred structural material (steel versus concrete) and seismic and wind forces. Making the 31.5 foot bay work for concrete structural system and a steel structural system for various seismic and wind forces is a vast task. Therefore, the scope of this report is wind forces based on a 90 mph, 3-second gust basic wind speed which governs over seismic forces, and a structural steel system.

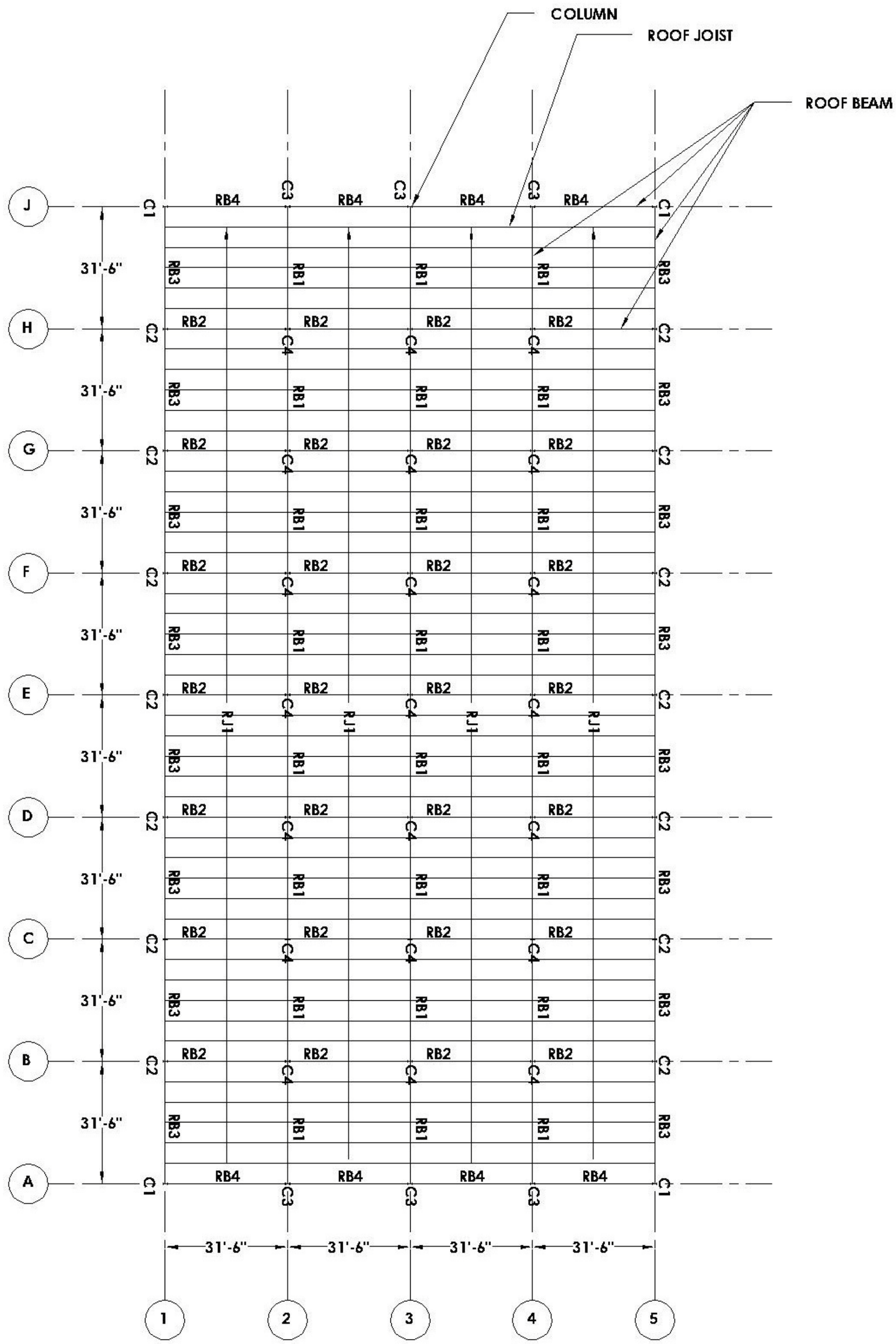

Figure 1.6 Roof Plan

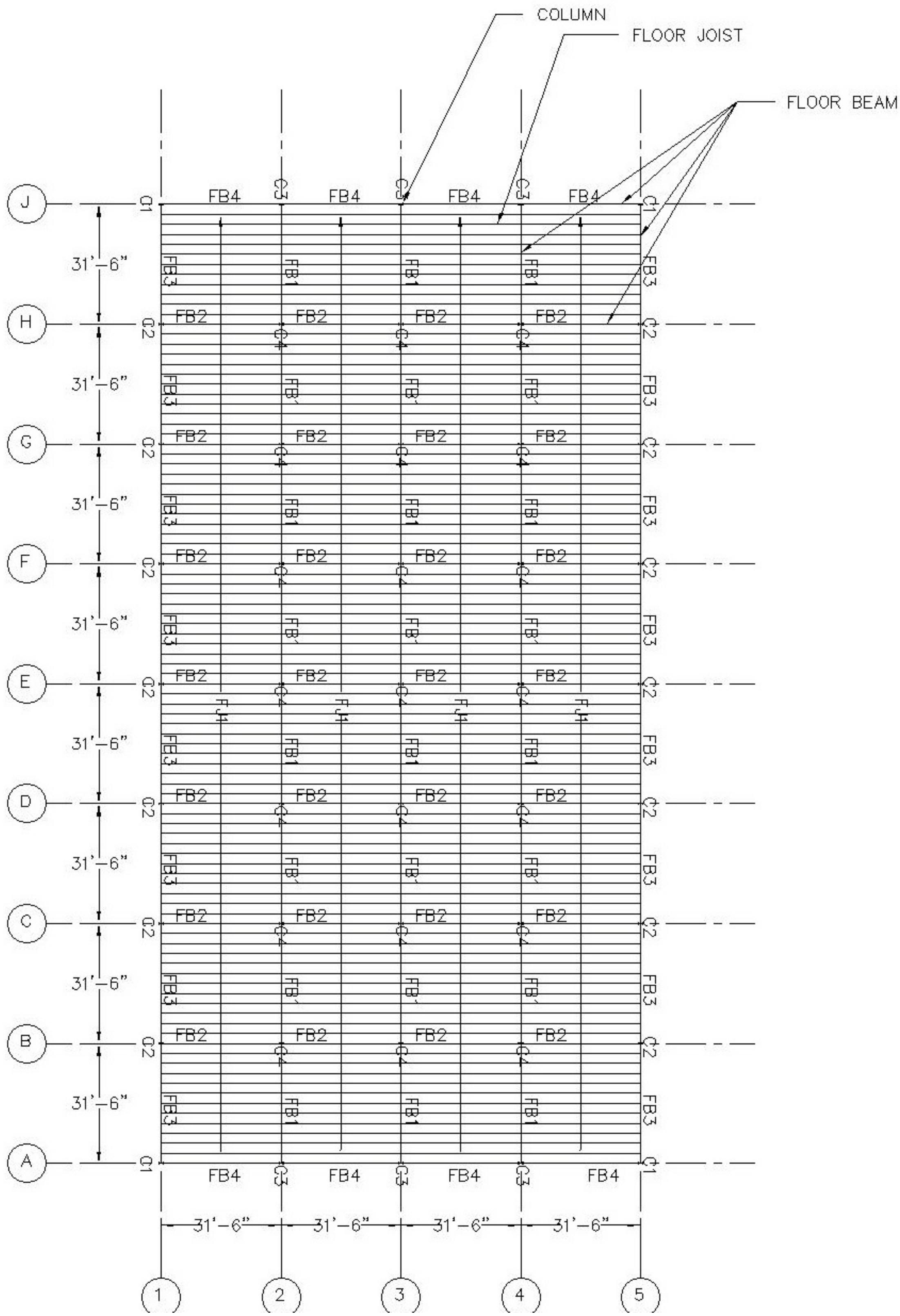

Figure 1.7 Floor Plan

Table 1.1 Roof/Floor Plan Labeling Scheme

RJ1	Roof Joists
RB1	Interior Roof Beams (Joist Bearing)
RB2	Interior Roof Beams (Non Joint Bearing)
RB3	Exterior Roof Beams (Joist Bearing)
RB4	Exterior Roof Beams (Non Joist Bearing)
FJ1	Floor Joists
FB1	Interior Floor Beams (Joist Bearing)
FB2	Interior Floor Beams (Non Joist Bearing)
FB3	Exterior Floor Beams (Joist Bearing)
FB4	Exterior Floor Beams (Non Joist Bearing
C1	Exterior Column Line (Building Corners)
C2	Exterior Column Line (Long Building Face)
C3	Exterior Column Line (Short Building Face
C4	Interior Column Line

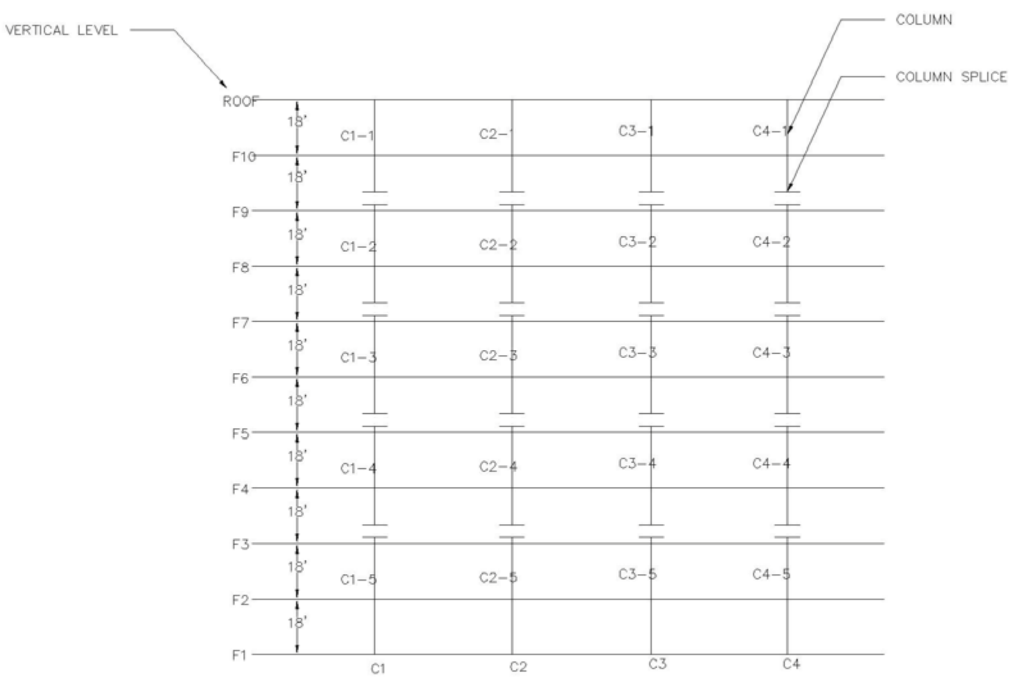

Figure 1.8 Column Elevation

Table 1.2 Column Elevation Labeling Scheme

C1-1	Exterior Column (Building Corners), Floors 9 and 10
C1-2	Exterior Column (Building Corners), Floors 7 and 8
C1-3	Exterior Column (Building Corners), Floors 5 and 6
C1-4	Exterior Column (Building Corners), Floors 3 and 4
C1-5	Exterior Column (Building Corners), Floors 1 and 2
C2-1	Exterior Column (Long Building Face), Floors 9 and 10
C2-2	Exterior Column (Long Building Face), Floors 7 and 8
C2-3	Exterior Column (Long Building Face), Floors 5 and 6
C2-4	Exterior Column (Long Building Face), Floors 3 and 4
C2-5	Exterior Column (Long Building Face), Floors 1 and 2
C3-1	Exterior Column (Short Building Face), Floors 9 and 10
C3-2	Exterior Column (Short Building Face), Floors 7 and 8
C3-3	Exterior Column (Short Building Face), Floors 5 and 6
C3-4	Exterior Column (Short Building Face), Floors 3 and 4
C3-5	Exterior Column (Short Building Face), Floors 1 and 2
C4-1	Interior Column, Floors 9 and 10
C4-2	Interior Column, Floors 7 and 8
C4-3	Interior Column, Floors 5 and 6
C4-4	Interior Column, Floors 3 and 4
C4-5	Interior Column, Floors 1 and 2

CHAPTER 2 - Discussion of Lateral Force Resisting System

Structural Issues

The basic framework of the *universal grid* – a 31.5 foot by 31.5 foot bay size, with an 18 foot floor-to-floor height – is used to explore the structural design implications for a steel structural system for a building with wind-governed lateral forces. The lateral force resisting system is the focus of this report, and the gravity system will be largely left undefined, as its effect upon the performance of the lateral system members is minimal. The diaphragm assumed for this building is steel deck with concrete fill, which will probably act as a rigid diaphragm, but the symmetric loading and building layout considered in this report will result in the same load distribution as a flexible diaphragm. The theoretical hospital is ten stories high and four bays wide by eight bays long in plan.

This chapter of the report will focus on narrowing down the available LFRS choices by considering isolated vertical frames using different LFRS types and applying unit loads to them to compare their performance and efficiency. The actual building model will be analyzed in chapter 3.

Lateral Force Resisting Systems

A continuous load path for lateral loads is required for any structure to be stable. The stability of the structure to resist wind horizontal forces and distribute these forces into the supporting soil is in the lateral force resisting system. The lateral force resisting system consists of two separate but integrally connected components of the structure – the diaphragm (horizontal elements) and the frames or shear walls (vertical elements). The frames or shear walls behave as cantilevers subjected to lateral loads. Frames may be braced or rigid (moment-resisting). Braced frames resist lateral loads by truss action in the vertical plane. Rigid frames resist lateral loads by the virtue of the moment-resisting joints. For a structural steel building, two lateral force resisting systems are common – moment-resisting frames and braced frames. Variations on these systems abound; for example, special moment resisting frames differ from ordinary moment resisting frames in their connection quality, and braced frames come in a variety of shapes, such as inverted V braces (also known as chevron braces) and X braces. See figures 2.3 and 2.4 for illustrations of these braced frames.

Moment Frames

A moment frame is a lateral force resisting system consisting of members designed to carry the flexural forces induced by lateral loads, as well as the axial forces. The connections between these members are considered to be "fixed," which indicates the ability of the connection to transfer bending forces between members. Resistance to lateral loads is provided primarily by the flexural resistance of columns, girders and rotation of beam-column joints. If a member bends at one end, the fixed connection is considered to bend along with the member so that the angle between members remains constant (for a fully fixed connection). Figure 2.1 shows the deflection and load transfer through a moment frame.

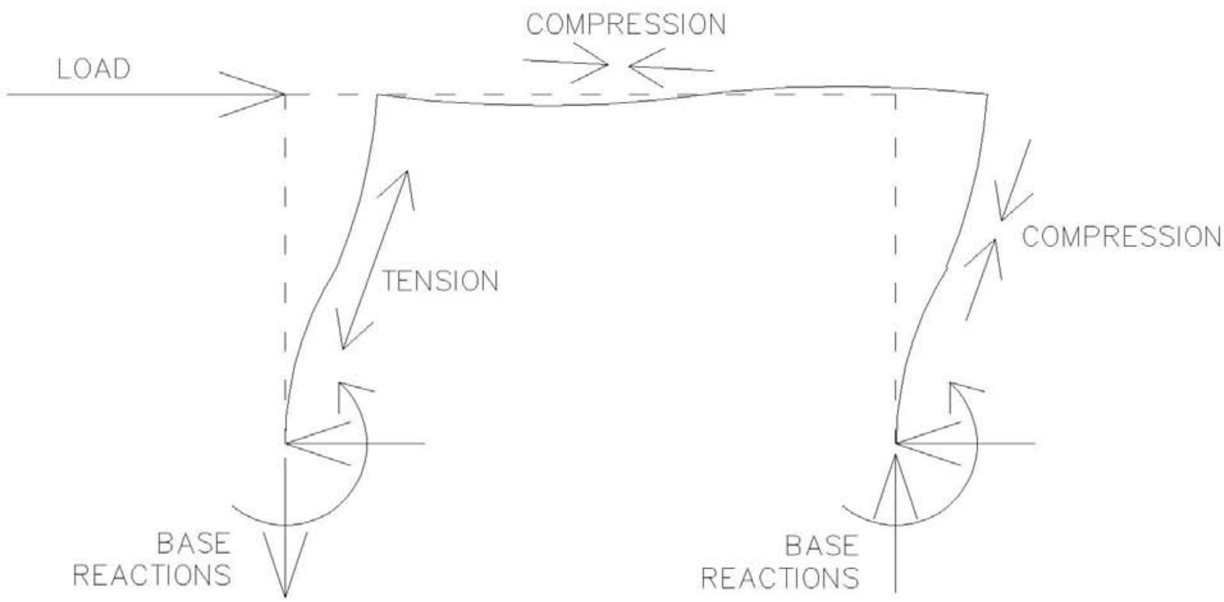

Figure 2.1 Moment Frame Load Path

The level of "fixity" of the connection can vary between "fully fixed" and "partially fixed" connections. For this report, fully fixed moment frame connections are examined. Refer to Figure 2.2 for a multi-story diagram of the type of moment frames used. The solid triangular symbol in the upper corners of the frame indicates a fixed connection that can transfer moment as well as axial loads. The connections between the lower ends of the columns and the ground are also fixed.

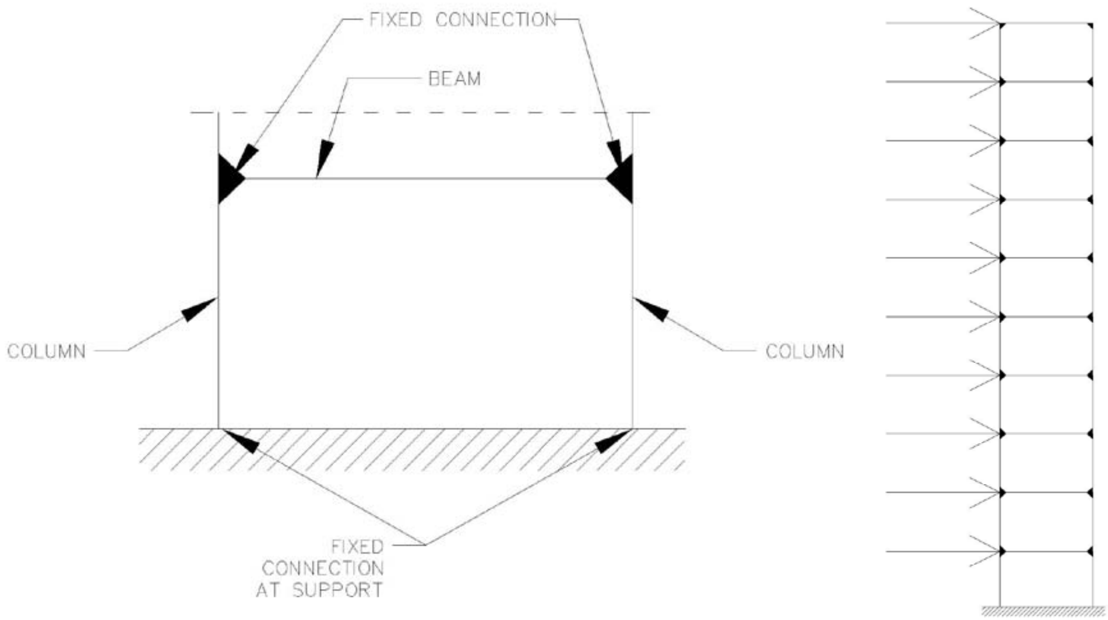

Figure 2.2 Moment Frame Diagram

A moment frame by definition is unbraced: horizontal displacement of the frame is allowed when loaded laterally, and the columns are subjected to sidesway. The ability of moment frame members and connections to bend allows for a certain level of "drift" (horizontal displacement of the upper nodes) to occur. When a lateral force is applied to the top of a moment frame, the columns bend as the top of the frame deflects in the direction of the applied load. Because of the fixity of the connections, this deflection forces rotation in the connections in order for the angles between the members to be maintained. This transfers load into the upper beam of the moment frame axially, and shears are induced in the tops of the columns. The rigidity of the connections causes the members to bend in reverse curvature as these loads are transferred. As the members bend, the top of the frame can shift substantially, depending on the members, the connections and the forces. Orienting the members so as to force the bending to occur about the major axis can decrease the deflections of the members. The ratio of the distance a frame deflects laterally under a given lateral load to the height of the frame is the "story drift ratio," and this is an important value to consider in the serviceability design of a building. High moment frame deflections can have a variety of negative effects on buildings. For example, they can damage glass, masonry and other building materials supported by the frames. They can also induce P-delta effects - additional moments induced by loads being applied at a distance from member axes due to deflection. For example, when a frame deflects, vertical loads that had originally been along the column axes can move off the axes due to the lateral movement of the upper part of the frame, and this can put additional bending moments into the frame.

The primary advantage to a moment frame is architectural – the area within a frame consisting simply of two columns and a beam is clear of structural members. This gives the architect a great deal of flexibility in his or her use of that space.

Braced Frames

Braced frames resist lateral loads by truss action (axial loads) in the vertical plane. Braced frames are comprised of horizontal members (beams or girders), vertical members (columns) and inclined members (braces), which are not present in moment frames. The joints between beams, columns and braces are flexible and assumed pin-connected theoretically. This allows rotation at the beam-column joint. Since the beam-column joints are unable to resist rotation in order to transfer lateral loads, the frame would be unstable without the presence of the braces. The beams, columns and braces form a vertical truss configuration and are analyzed as a vertical truss. The braces tend to prevent the columns from swaying. Because the joints of braced frames are not designed to carry moment, all the forces in braced frames are transmitted through the structure in the form of axial loads – tension and compression. Steel is extremely resistant to tensile forces in comparison to other structural materials, and braced frames are an attractive option for redirecting lateral loads. Two primary advantages of braced frame systems over moment frame systems are the simplicity of their connections and the increased resistance to story drift. In general, a braced frame is far stiffer than a moment frame with similar dimensions and member sizes. To demonstrate the differences between the different frame systems, I modeled a sample 31.5 foot x 18 foot frame from my hospital system as a moment frame (with fully fixed upper and lower end connections), an X-braced frame (once with braces designed to take tension forces only, and once with braces designed to take tension and compression forces), and a chevron braced frame. See Figures 2.3 and 2.4 for the two braced frames.

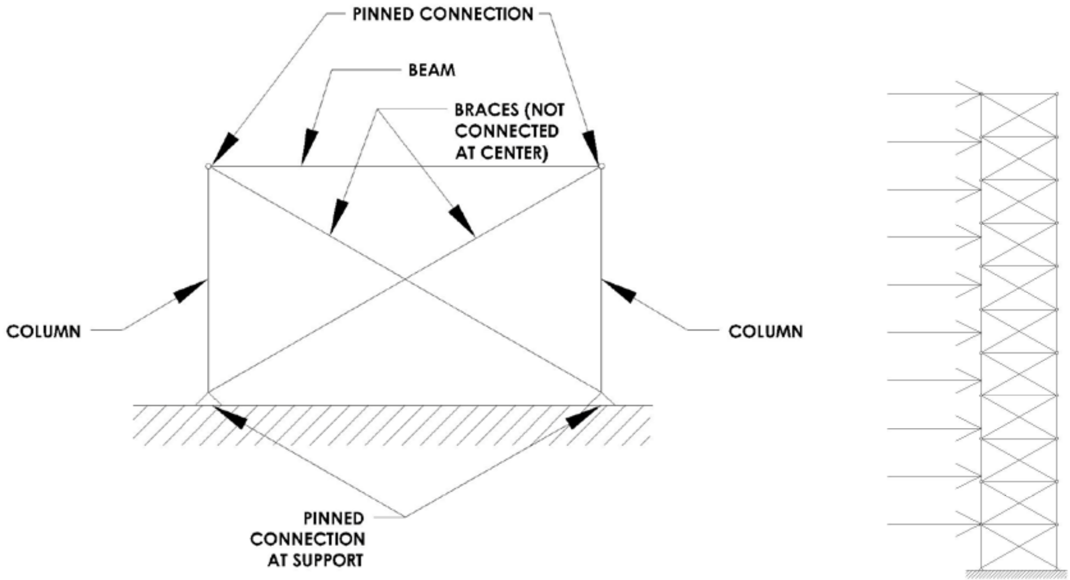

Figure 2.3 X-Braced Frame Diagram

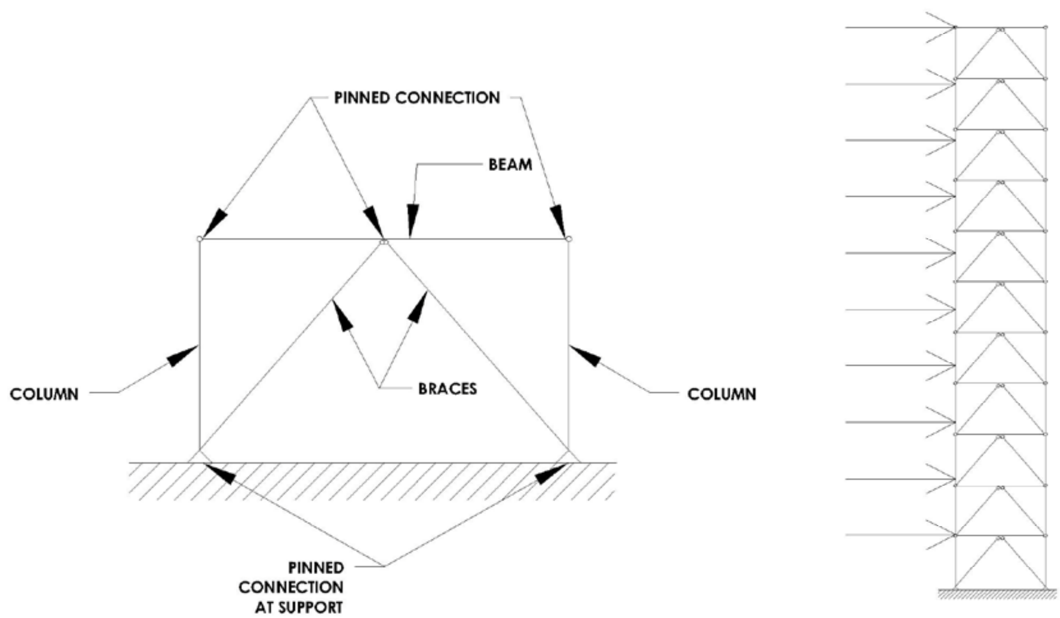

Figure 2.4 Chevron Braced Frame Diagram

Frame System Comparison

A horizontal load of 10 kips was applied to the top of each single-story frame in the plane of the frame (in each direction). The size of the load is an arbitrary figure chosen to give a comparable set of displacement results between the different frame systems. After designing the steel members of all the frames for strength requirements, as well as a maximum lateral drift requirement of h/400 (Minimum Design Loads for Buildings and Other Structures 2002 366), which is .54 inches for the universal grid height of 18 feet, the chevron braced frame turned out to be the most economical option. The chevron braced frame required 1800 pounds of steel in a single story frame, while 3200 pounds of steel were required for the moment frame. The X-braced frame required 1900 pounds of steel when the braces were designed to take tension only, and 2000 pounds of steel when the braces were designed to take tension and compression forces. The moment frame required such a large amount of steel because it was heavily governed by drift – it could satisfy the strength requirements alone with much smaller members, but the lateral drift was much larger than the h/400 serviceability requirement. In order to drastically reduce the lateral deflection, much stiffer members were needed. The braced frames, however, each required little or no adjustment once designed for strength requirements – they were naturally able to fulfill the horizontal drift requirements without increasing the member stiffness. Each braced frame uses almost the same amount of material to resist the 10 kip load.

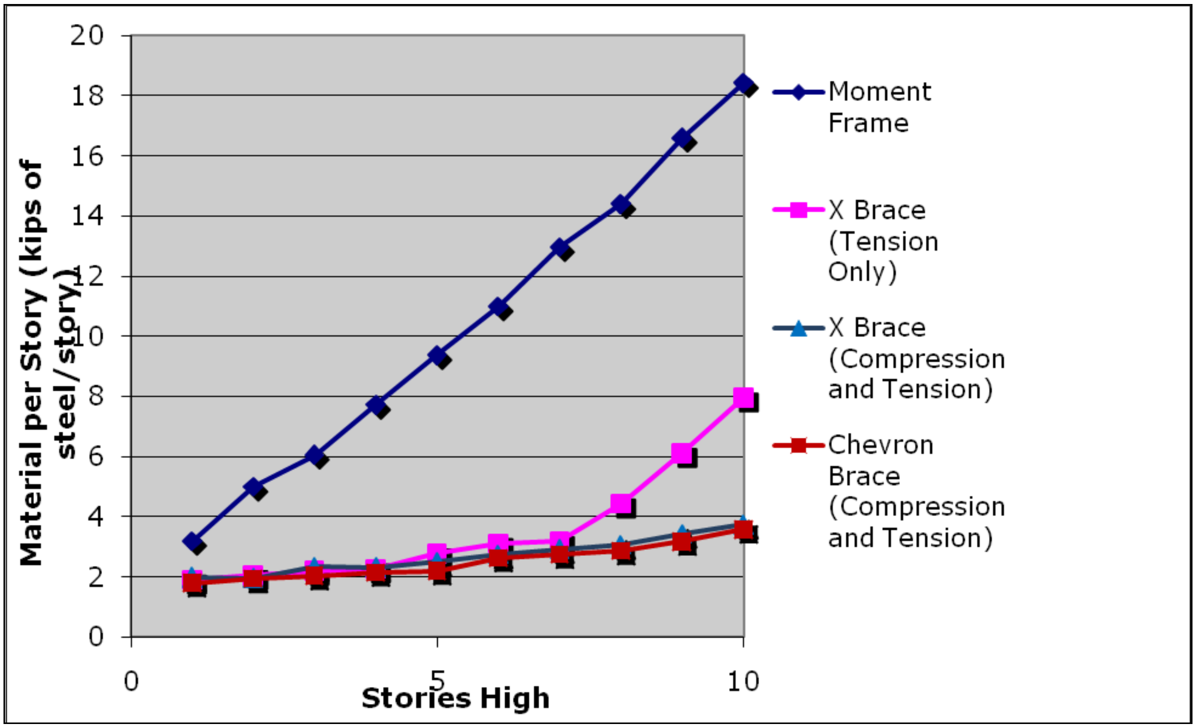

Figure 2.5 Sample Frame Material Use vs. Height

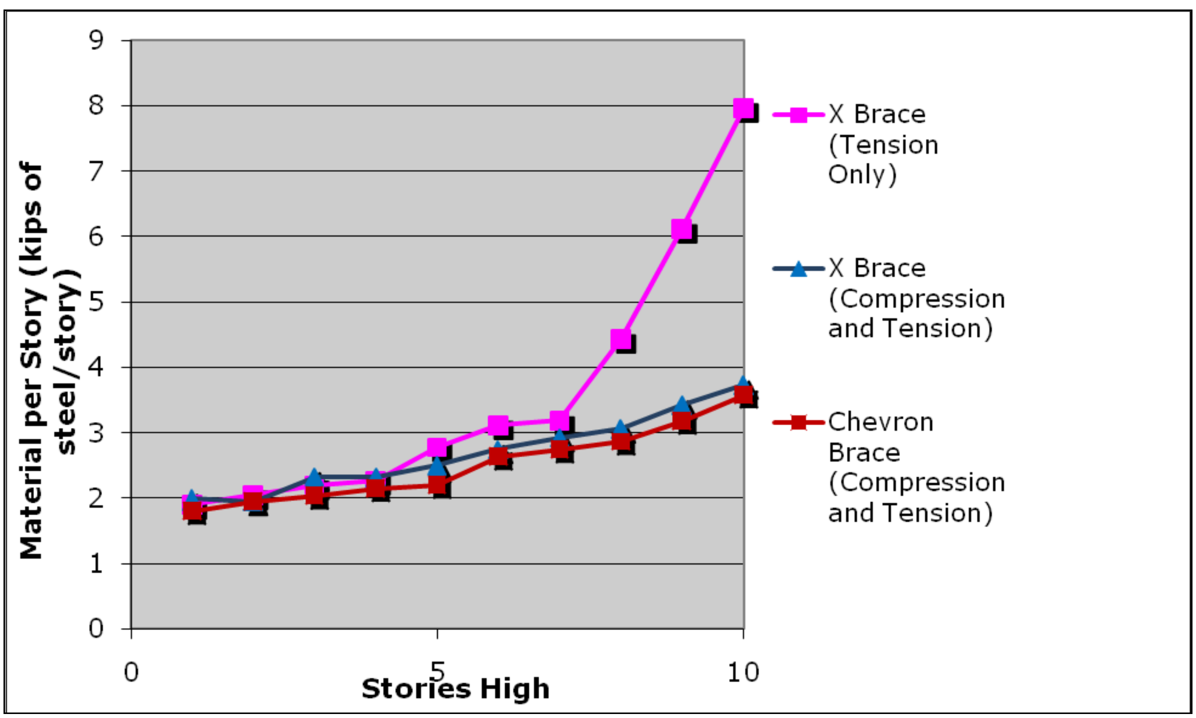

Figure 2.6 Sample Frame Material Use vs. Height (Braced Frames Only)

To provide a more comprehensive demonstration, additional models with similar frames were examined which varied from one to ten vertical stories. For each iteration, a 10 kip horizontal load at each story was used to provide a simple comparison between the performances of the different lateral bracing systems. In order to simplify the design and make it more realistic, column sizes were limited to W14 members and shallower, and beam sizes were limited to W30 members or shallower. The results for basic material efficiency (steel required per floor) are shown in Figures 2.5 and 2.6. The braced frames use the least amount of steel, as their member sizes are all governed by strength requirements – once these requirements are met, their lateral drift turned out to be lower than the maximum allowable of .54 inches per story, and no resizing was required. The percentage of actual drift to allowable drift started as low as 1-2% for a small number of stories, and increased to 75-85% for large numbers of stories. The exception to this pattern is in the X-braced frames with tension-only braces. Starting at about 8 stories, the member sizes of these braced frames are governed by drift rather than strength, and the material efficiency curve of the tension-only X-braced frames begins to parallel that of the moment frames. Moment frames perform less efficiently in comparison at all heights, as the member sizing of the moment frames are governed by drift, rather than strength, at all frame heights. The optimum member sizes for all frames at all heights, determined with the aid of RISA 3D, can be seen in Table 2.1.

Number of Stories	Moment Frames				X-Braced Frames (Tension Braces Only)			
	Beam Sizes	Column Sizes	Brace Sizes	Material Takeoff (thousands of lbs of steel/story)	Beam Sizes	Column Sizes	Brace Sizes	Material Takeoff (thousands of lbs of steel/story)
1	W14x48	W14x48	n/a	3.2	W6x20	W8x24	HSS4x4x1/8	1.9
2	W30x90	W14x61	n/a	5	W8x24	W8x24	HSS4x4x1/8	2.05
3	W30x90	W14x90	n/a	6.07	W8x28	W8x24	HSS4x4x1/8	2.2
4	W30x108	W14x120	n/a	7.73	W8x31	W8x24	HSS4x4x1/8	2.275
5	W30x116	W14x159	n/a	9.38	W8x31	W14x38	HSS4x4x1/8	2.78
6	W30x148	W14x176	n/a	11	W8x35	W14x38	HSS4x4x3/16	3.12
7	W30x191	W14x193	n/a	12.97	W8x31	W14x38	HSS4x4x1/4	3.19
8	W30x191	W14x233	n/a	14.4	W10x54	W14x53	HSS4x4x1/4	4.44
9	W30x261	W14x233	n/a	16.61	W14x74	W14x82	HSS4x4x1/4	6.12
10	W30x261	W14x283	n/a	18.43	W18x76	W14x132	HSS4x4x1/4	7.97

Number of Stories	X-Braced Frames (Compression and Tension Braces)				Chevron Braced Frames			
	Beam Sizes	Column Sizes	Brace Sizes	Material Takeoff (thousands of lbs of steel/story)	Beam Sizes	Column Sizes	Brace Sizes	Material Takeoff (thousands of lbs of steel/story)
1	W6x15	W8x24	HSS5.5x5.5x1/8	2	W6x20	W8x24	HSS4x4x1/8	1.8
2	W6x15	W8x24	HSS5.5x5.5x1/8	1.95	W8x24	W8x24	HSS4.5x4.5x1/8	1.95
3	W6x15	W8x24	HSS6x6x3/16	2.33	W8x24	W8x24	HSS5.5x5.5x1/8	2.03
4	W6x15	W8x24	HSS6x6x3/16	2.33	W8x28	W8x24	HSS5.5x5.5x1/8	2.15
5	W6x15	W8x24	HSS7x7x3/16	2.5	W8x28	W8x24	HSS6x6x1/8	2.2
6	W6x15	W8x31	HSS7x7x3/16	2.75	W8x31	W8x28	HSS6x6x3/16	2.63
7	W6x15	W8x31	HSS8x8x3/16	2.93	W8x31	W8x31	HSS6x6x3/16	2.74
8	W6x15	W8x35	HSS8x8x3/16	3.06	W8x31	W8x31	HSS8x6x3/16	2.86
9	W6x15	W10x45	HSS8x8x3/16	3.43	W8x31	W8x40	HSS8x6x3/16	3.18
10	W6x15	W10x54	HSS8x8x3/16	3.74	W8x31	W8x48	HSS8x8x3/16	3.58

Table 2.1 Sample Frame Member Sizes

The drawback to using braced frames is purely architectural – the area available beneath the braces in the frame decreases the closer the frame is to an X-braced frame (that is, the larger the angle is between the columns and the braces). Generally, the space beneath the braces is the space available for people and equipment to move freely through, though the space above the braces may be architecturally useful for other purposes, such as windows. See Figure 2.7 for the relationship between the brace angle in a frame and the available area beneath the braces in the frame.

Based on the results of these analyses, moment frames are discarded as an option for the full building lateral system. Moment frames undergo lateral drifts far higher than braced frames, especially for buildings as tall as 10 stories, such as my theoretical hospital model. The moment frames required large increases in member sizes to meet drift requirements, and subsequently, the material efficiency of the moment frames suffered greatly, as is evident in Figure 2.5. The most efficient lateral system considered in terms of both strength and story drift is the chevron braced frame. The chevron braced frame also has greater available space beneath its braces than the X-braced frame. As a result of these analyses, the chevron braced frame is exclusively used in the full building model.

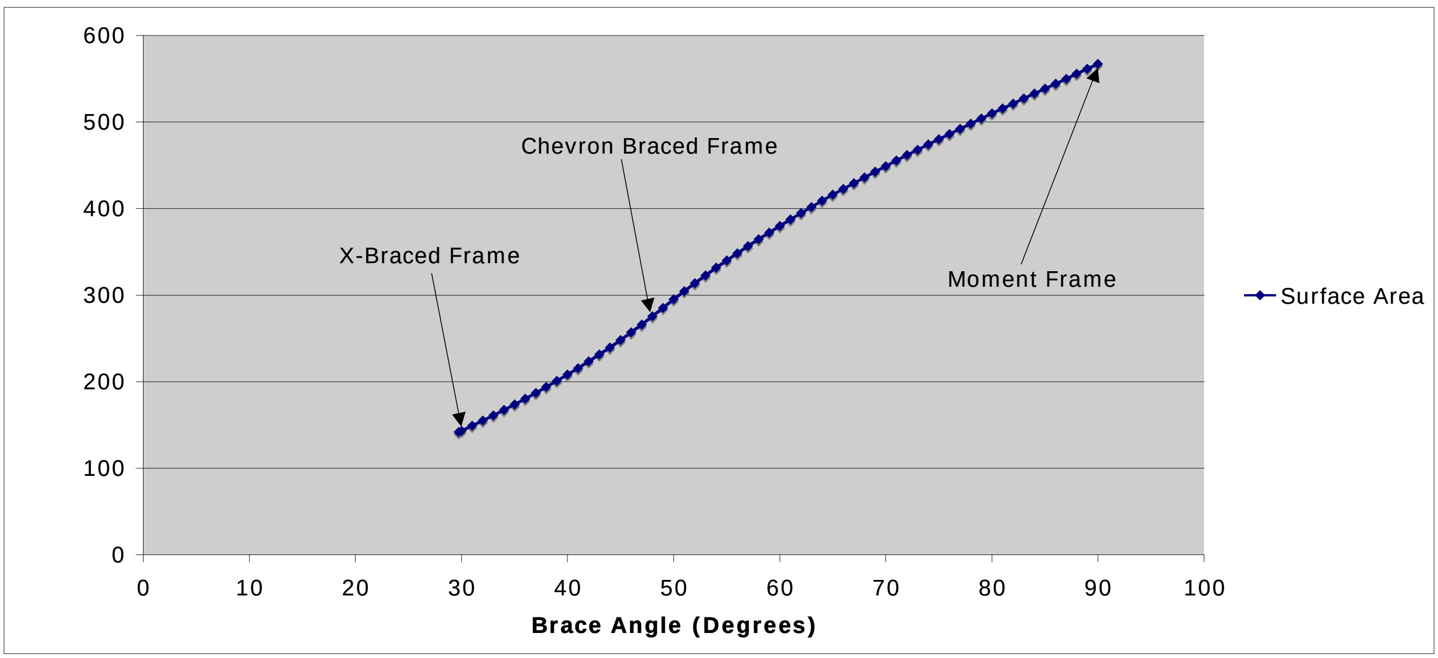

Figure 2.7 Frame Space vs. Brace Angles

CHAPTER 3 - ANALYSIS AND CALCULATIONS

The structural analysis calculations for the full building model have been done with the aid of RISA 2D software. The configuration and dimensions of the hospital layout are shown in Figures 1.6, 1.7 and 1.8. The general analysis procedure consisted of the following steps:

i. Determine a theoretical building size.
ii. Calculate basic assumed gravity loads that would apply to many hospital building spaces.
iii. Determine minimum member sizes based on these gravity loads.
iv. Determine wind loads for an area with average wind speeds.
v. Model lateral force resisting as chevron braced frames and size members to take the calculated wind loads.
vi. Estimate material cost for any feasible building designs determined by the analysis.

Each of these steps is presented in detail in the following sections.

Building Size

A 4 bay by 8 bay, 10 level building was analyzed. This gives a fairly large hospital to analyze, but it is not so large that it needs to be separated into two separate structures by construction joints. If the floor and roof diaphragms had to span too large a distance, their structural performance would suffer. When a diaphragm taking lateral load is very wide (parallel to the load) compared to its "depth" (perpendicular to the load), it is like a long shallow beam – the moment increases with length, and the couple forces generated in the chords required to resolve the moment are large due to the short distance between them. With 31.5 foot bays, 4 bays by 8 bays means a 126 foot by 252 foot building, which was pushing the acceptable boundary of a single diaphragm span. It was important to analyze a large building, however, as the point of the universal grid is to streamline the design of large hospitals by eliminating design variations between different areas. The 2 to 1 ratio of the building's length to its width is based upon Cannon Design case studies, which base their room layouts upon an approximately 2 to 1 floor plan ratio.

Gravity Loads

In order to determine minimum member sizes for the building's beams and columns, gravity loads were applied to them and they were sized based upon LRFD strength criteria and a maximum total deflection criteria of L/240 (International Building Code 280). The gravity loads applied to the building are as follows:

Roof dead load = 20 psf (pounds per square foot)

Roof live load = 20 psf

Roof snow load = 24 psf

Floor dead load = 60 psf

Floor live load = 100 psf

Wall dead load = 10 psf

These loads are based on a location of Chicago, Illinois, and the Minimum Design Loads for Buildings and Other Structures 2005. See Appendix A for the calculations determining these loads. The live load reduction provisions to reduce live loads for certain members with large tributary areas were used (Minimum Design Loads for Buildings and Other Structures 2005 10).

Chicago has a ground snow load of 25 psf (85) and a wind speed of 90 mph (33). Figure 3.1 combines the Minimum Design Loads for Buildings and Other Structures 2005 wind and snow load maps of the United States to show the areas where loads of these magnitudes or lower would apply. The purple area of the map is the area that these load assumptions could apply to. However, high seismic areas such as California would usually not use wind as a governing lateral case. Figure 3.1 does not account for seismic loads that would govern over a 90 mph wind speed.

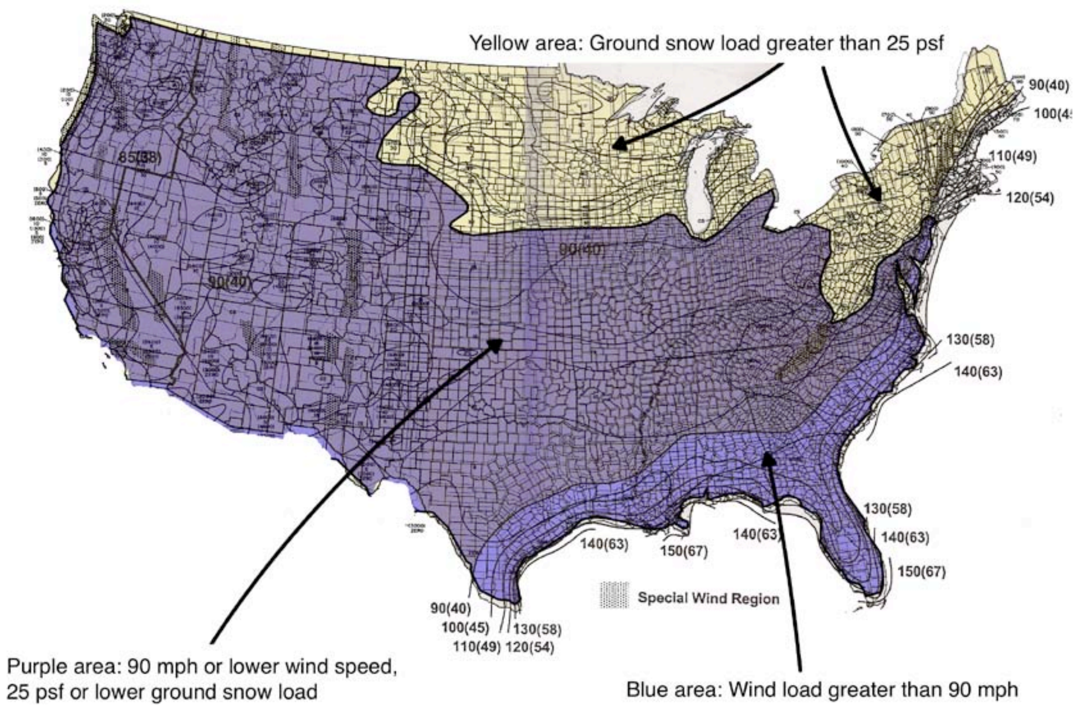

Figure 3.1 Combined ASCE 7-05 Snow and Wind Maps

Gravity Member Sizes

The minimum member sizes based on these gravity loads are contained in Table 3.1. (Refer to Figures 1.6, 1.7 and 1.8 for identification of each member label.) LRFD strength criteria were used to determine these member sizes, along with live load deflection criteria of L/240 for the beams (International Building Code 280). (Total load deflection criteria, given in the IBC as L/180, was not considered because the IBC allows dead load for steel structures to be considered as zero when determining deflections. This reduces the total load deflection to live load deflection only, and the L/240 criteria governs over L/180).

Table 3.1 Gravity Member Sizes

Member Label	Member Size	Member Label	Member Size	Member Label	Member Size
RJ1	18K5	C1-1	W8X31	C3-1	W8X31
RB1	W18X35	C1-2	W8X31	C3-2	W8X40
RB2	W12X14	C1-3	W8X31	C3-3	W10X49
RB3	W8X28	C1-4	W10X45	C3-4	W12X65
RB4	W10X12	C1-5	W10X49	C3-5	W12X72
FJ1	20K10	C2-1	W8X31	C4-1	W8X31
FB1	W21X55	C2-2	W10X39	C4-2	W10X54
FB2	W12X19	C2-3	W10X49	C4-3	W12X72
FB3	W18X40	C2-4	W12X58	C4-4	W14X90
FB4	W12X14	C2-5	W12X65	C4-5	W14X109

The total structural material weights for these members are 1,242,018 pounds of structural steel and 1,256,270 pounds of open-web steel joists. The member sizing calculations are located in Appendix B.

Wind Loads

The wind loads for the building model were calculated using the analytical method of Minimum Design Loads for Buildings and Other Structures 2005 (24-30). For the purposes of simplification, Cases 2, 3 and 4 of the procedure were not considered in the lateral system design. These cases, shown graphically in Figure 6-9 of Minimum Design Loads for Buildings and Other Structures 2005 (52), account for wind forces acting at a distance from the building's center of gravity, creating torsional effects. The magnitudes of the wind forces themselves are decreased in these cases, and they are unlikely to be major governing factors in this building's design. In addition, the aim of this design is to determine the most efficient lateral structural system for this theoretical building, which would probably be consistent in configuration, if not in actual member sizes, for different load magnitudes. In a real building design, of course, they would have to be considered. The wind loads based on these criteria are shown in Table 3.2.

Table 3.2 Case 1 Wind Loads: Longitudinal, Transverse Forces Applied Separately

		Wind Direction							
		Longitudinal				Transverse			
		Wall Area (ft^2)	Windward Force (lbs)	Leeward Force (lbs)	W (kips)	Wall Area (ft^2)	Windward Force (lbs)	Leeward Force (lbs)	W (kips)
Floor	Roof	1134	13300	11600	24.9	2268	26700	32300	59.0
	10	2268	25900	23200	49.1	4536	51700	64600	116.3
	9	2268	24700	23200	47.9	4536	49400	64600	113.9
	8	2268	23400	23200	46.6	4536	46800	64600	111.3
	7	2268	22000	23200	45.2	4536	43900	64600	108.5
	6	2268	20400	23200	43.6	4536	40700	64600	105.3
	5	2268	18500	23200	41.7	4536	37000	64600	101.6
	4	2268	16300	23200	39.5	4536	32600	64600	97.1
	3	2268	13400	23200	36.6	4536	26800	64600	91.4
	2	2268	9530	23200	32.7	4536	19100	64600	83.6

Lateral Member Sizes

To design the building's lateral force resisting system, RISA 2D was used to model full-height two-dimensional frames that represented single grid lines of the three-dimensional building model. Four situations were considered:

1. Transverse wind loads with braced frames only in the two side walls.
2. Transverse wind loads with braced frames in the two side walls and along the middle wall.
3. Longitudinal wind loads with braced frames only in the two side walls.
4. Longitudinal wind loads with braced frames in the two side walls and along the middle wall.

Figure 3.2 shows these braced frame arrangements.

All the appropriate loads were applied to the frames, including dead, live, snow and wind, and the appropriate load combinations were applied as well (Minimum Design Loads for Buildings and Other Structures 2005 5). Each two-dimensional frame was then analyzed with braces in every vertical bay, then with braces in fewer and fewer bays all the way down to only one full vertical bay. Naturally, the frames with fewer bays of braced frames required larger brace member sizes in order to handle the lateral loads. The more braces that were added, the smaller the member sizes could become.

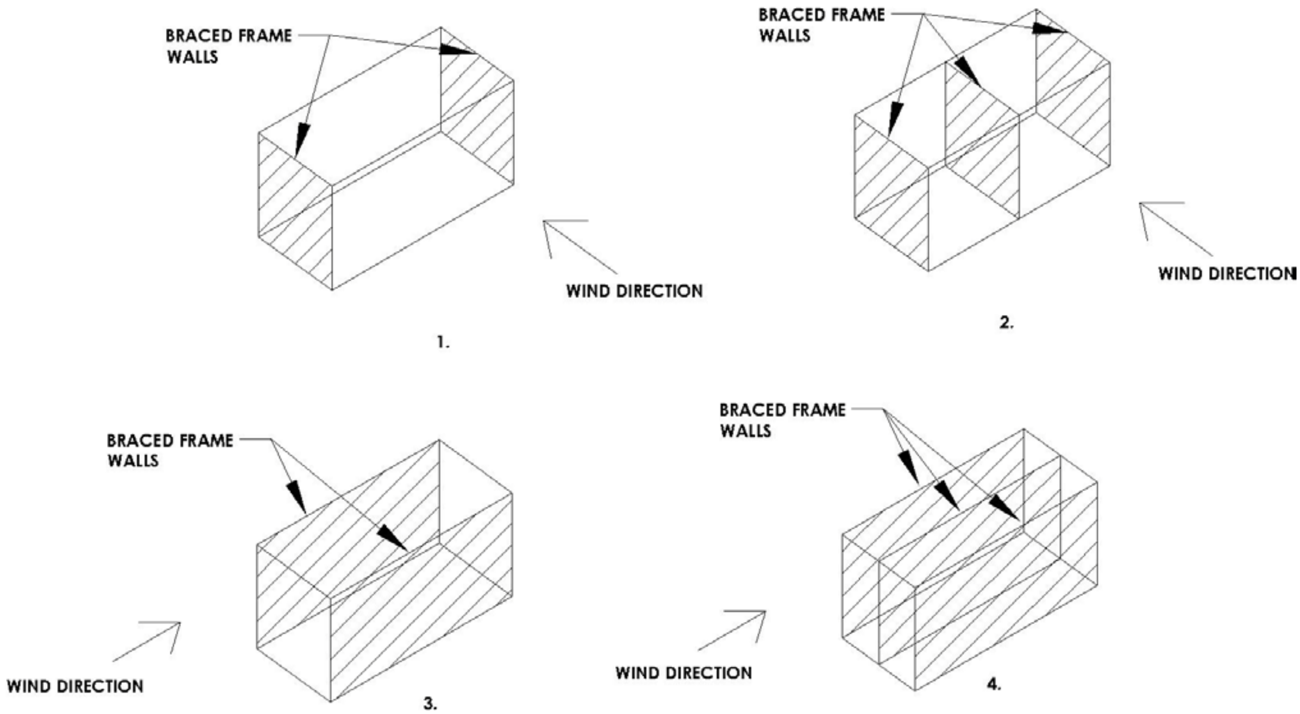

Figure 3.2 Braced Frame Arrangements

However, there was usually a point at which adding extra braced frames didn't allow for an appreciable reduction in braced frame member size, and the total frame material takeoff would start to increase despite the slightly smaller braced frame member sizes. This is due to the slenderness factor becoming a governing factor when the braced frame member sizes are reduced to a certain point. Going above the acceptable slenderness factor in a member forces you to increase the member size regardless of the load put into the member. Adding braced frame bays to reduce the load going into each bay no longer allows you to reduce member size once the slenderness factors for the members are reached. The point at which the material takeoff begins to suffer is represented by minimums in Figures 3.3 and 3.4, which show the amount of steel used in the braced frames for each considered case. The points where the lines in the figures start to move upward after having initially moved downward represent the most efficient braced frame configuration for that building frame, in terms of total steel weight. The situation in which the longitudinal wind forces are applied to only the two side walls of the building was unique in that it had no such minimum point – each braced frame addition increased the overall efficiency of the frame, all the way up to completely filling the side walls with braced frames. The final lateral system was chosen to provide the most efficient possible material usage. Thus, both outer longitudinal walls will be fully braced in every vertical bay, and both outer transverse walls and the middle transverse wall will be braced in 3 of the 4 available vertical bays. (Using the outer walls and middle wall for the transverse braces helps to cut down the distance the floor and roof diaphragms will have to span against transverse direction loads.)

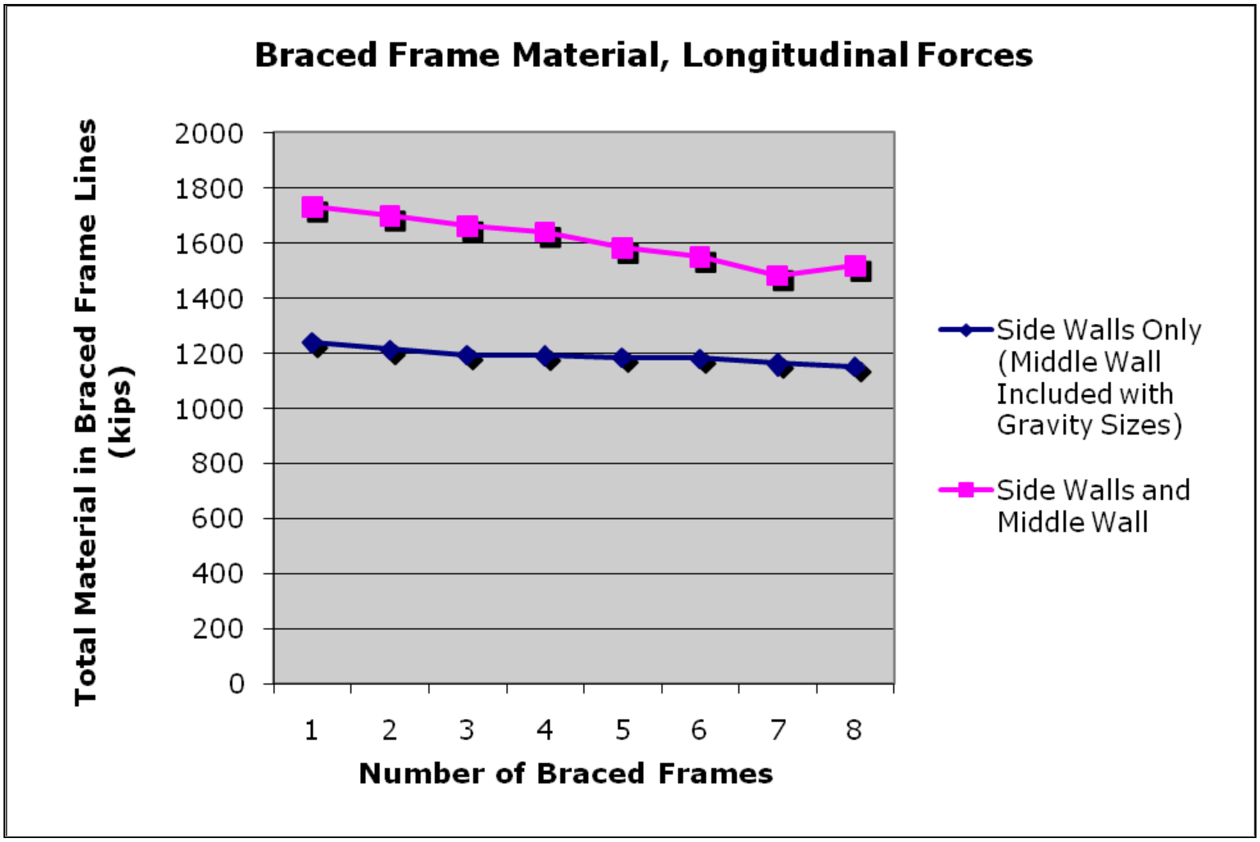

Figure 3.3 Braced Frame Material, Longitudinal Forces

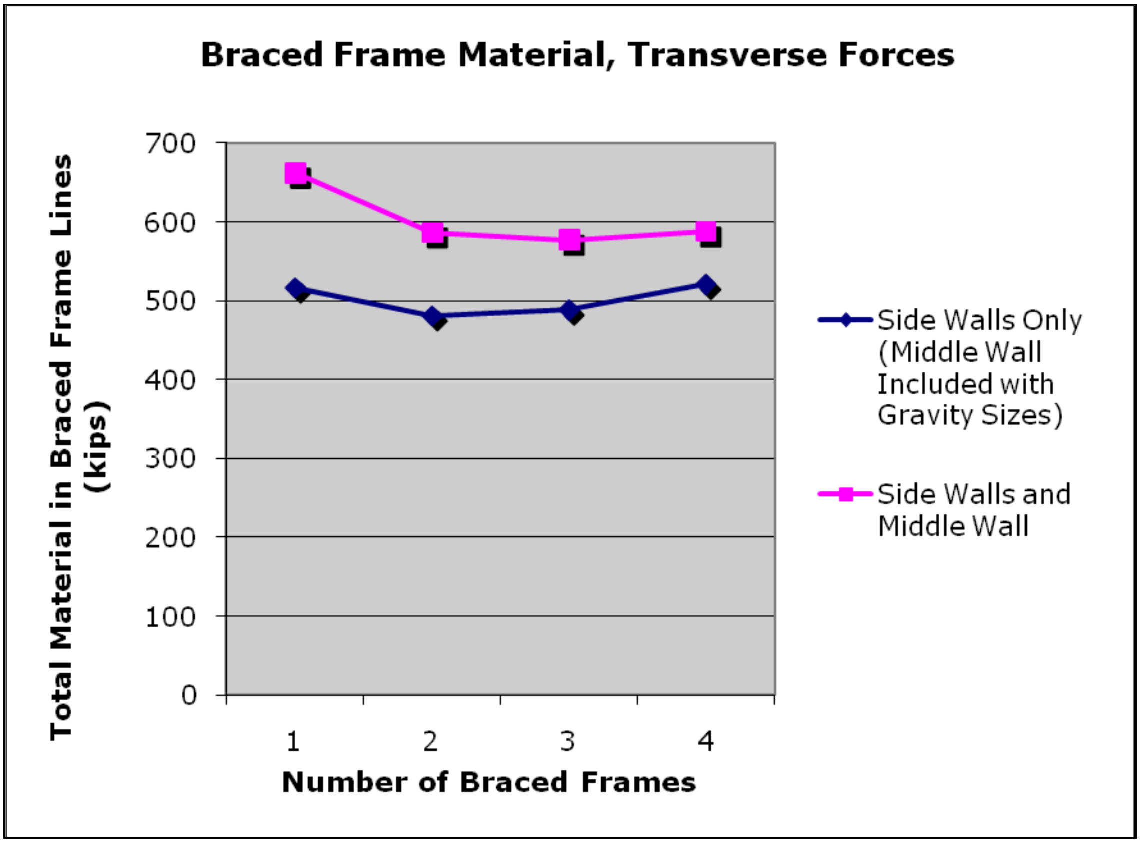

Figure 3.4 Braced Frame Material, Transverse Forces

Cost Analysis

The estimated amount of steel needed for the above system is 3,285,270 pounds. This includes only the steel joists, braces, beams and columns. If the building were designed with no lateral system – that is, if it were only composed of gravity-sized members (and had no braces at all), the amount of steel needed would be 2,498,288 pounds. In this case, the lateral system adds about 30 percent to the steel material cost. (Whether or not to include a lateral force resisting system at all is not an option, however, as any building without one will be unstable when lateral loads are applied.) The difference in labor cost was not calculated, but the relative simplicity of braced frame connections, in comparison to moment frame connections, adds to the assumption that the system chosen truly is the most efficient for this building. Connection cost, both in terms of labor and material, could change the most efficient number of braced frames, as each additional braced frame connection requires an extra amount of labor and material that isn't used for connection that merely transfer gravity loads. Further study of these costs would be required to determine a truly cost-efficient frame system.

CHAPTER 4 - CONCLUSIONS

This report provides an investigation into the details concerning the structural design of a hospital based upon a *universal grid* size determined by architects at Cannon Design. The benefits and drawbacks of moment frames, X-braced frames and chevron braced frames were compared as lateral force resisting elements for frames utilizing the *universal grid*. Upon applying these benefits and drawbacks to a theoretical hospital building model based upon the Cannon Design universal grid, the possible systems were narrowed down to one – the chevron braced frame.

The chevron braced frames were used to model full building frames for the theoretical hospital model, varying the number of braced bays in each frame wall. Designing these different braced frame configurations allowed me to find the most efficient arrangement of braces for wind loads applied in both the longitudinal and the transverse directions. Using steel material takeoffs as the measure of system efficiency, the most economical chevron braced frame arrangement was determined, given the gravity loads, wind loads and building size determined in Chapter 3. This arrangement was to fully brace the outer longitudinal walls of my theoretical building, and to brace 3 out the 4 vertical bays of the outer transverse walls as well as the middle transverse wall. Figure 4.1 illustrates this braced frame arrangement. Fully bracing the exterior walls is not absolutely necessary for the design to work, however, and in a real building, the number of braced frames would likely need to be reduced in order to meet architectural requirements, like desired available wall opening space. The optimum bracing determined by this analysis only takes structural material takeoff into account, and adding additional braced frame bays usually allowed for enough member size reduction to be worthwhile from a pure material takeoff standpoint.

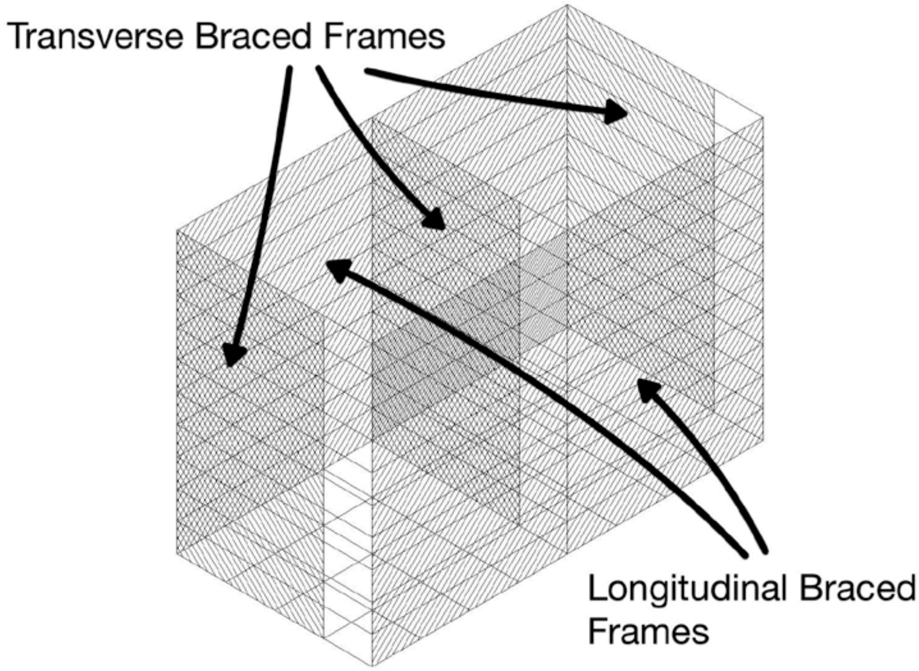

Figure 4.1 Final Braced Frame Arrangement

The main lesson to be learned from this project is that simplifying a building design process by starting with a pre-defined *universal grid* does not eliminate the need to make many important choices. The *universal grid* does not provide a one-size-fits-all structural system – lateral system choices need to be narrowed down by considering a series of other factors, which may or may not hold for buildings in different areas or of different sizes, regardless of whether they use the *universal grid*. The report's building's location in Chicago, IL determined the governing lateral loads, and a similarly sized building in a different location would have to resist different loads, which could drastically alter the braced frame arrangement shown in Figure 4.1. Also, a different sized hospital using the *universal grid* system would have a very different behavior under lateral loads, possibly different enough that a basic lateral element other than the chevron braced frame would be ideal. However, basic trends and patterns in performance between different lateral systems and configurations were identified in this report, and this information could save engineering time by providing useful starting points when designing a building with the *universal grid*.

References

International Building Code, 2nd ed. (Country Club Hills, IL: Internationa Code Council, 2006)

Minimum Design Loads for Buildings and Other Structures, 2nd ed. (Reston, VA: American Society of Civil Engineers, 2002)

Minimum Design Loads for Buildings and Other Structures, 2nd ed. (Reston, VA: American Society of Civil Engineers, 2005)

Natalie Petzoldt, email interview, 8 Aug. 2007.

National Health Care Practice Group Research Initiative: Universal Grid Theory, PDF file, Cannon Design, 2005.

Steel Construction Manual, 13th ed. (Chicago: American Institute of Steel Construction, Inc., 2005)

Vulcraft Steel Floor and Roof Deck (Lawrenceville, GA: Nucor Vulcraft Group, 2001)

Vulcraft Steel Joists and Joist Girders (Lawrenceville, GA: Nucor Vulcraft Group, 2007)

Appendix A - Gravity Load Calculations

The gravity load calculations for the building were carried out according to ASCE 7-05 using MathCAD 13.0.

Dead Load

Floor Dead Load

DL = 44 psf + 4 psf + 10 psf + 2 psf	DL = 60 psf	(5” 3VLI19 deck, mech. ducts, gypsum plaster ceiling, misc.)

Roof Dead Load

DL_R = 2 psf + 1.5 psf + 10 psf + 4 psf + 2.5 psf	DL_R = 20 psf	(B24 roof deck, 1” rigid ins., gypsum plaster ceiling, mech. ducts, misc.)

Live Load

Floor Live Load

$LL := 100psf$ (ASCE 7-05, Table 4-1)

Roof Live Load

$LL_R := 20psf$ (ASCE 7-05, Table 4-1)

Floor Live Load Reduction

N - S Beams (Beams on letter grid lines)

Tributary Area: $A_{T.1} := .75 \cdot 31.5ft \cdot 31.5ft$

$A_{T.1} = 744.188 \cdot ft^2$

$K_{LL.1} := 2$ (ASCE 7-05, Table 4-2)

$K_{LL.1} \cdot A_{T.1} > 400ft^2 = 1$

$$LL_{reduced.1} := \max\left[LL \cdot \left(.25 + \frac{15}{\sqrt{K_{LL.1} \cdot \frac{A_{T.1}}{ft^2}}}\right), .5 \cdot LL\right]$$ (ASCE 7-05, Section 4.8.1)

$LL_{reduced.1} = 63.881 \cdot psf$

Interior Columns

Tributary Area: $A_{T.2} := 31.5ft \cdot 31.5ft$

$A_{T.2} = 992.25 \cdot ft^2$

$K_{LL.2} := 4$ (ASCE 7-05, Table 4-2)

$K_{LL.2} \cdot A_{T.2} > 400ft^2 = 1$

$$LL_{reduced.2} := \max\left[LL\cdot\left(.25 + \frac{15}{\sqrt{K_{LL.2}\cdot\frac{A_{T.2}}{ft^2}}}\right), .4\cdot LL\right]$$ (ASCE 7-05, Section 4.8.1)

$LL_{reduced.2} = 48.81\cdot psf$

Assumption: Columns supporting more than one floor.

Side Columns

Tributary Area: $A_{T.3} := .5\cdot 31.5ft\cdot 31.5ft$

$A_{T.3} = 496.125\cdot ft^2$

$K_{LL.3} := 4$ (ASCE 7-05, Table 4-2)

$K_{LL.3}\cdot A_{T.3} > 400ft^2 = 1$

$$LL_{reduced.3} := \max\left[LL\cdot\left(.25 + \frac{15}{\sqrt{K_{LL.3}\cdot\frac{A_{T.3}}{ft^2}}}\right), .4\cdot LL\right]$$ (ASCE 7-05, Section 4.8.1)

$LL_{reduced.3} = 58.672\cdot psf$

Assumption: Columns supporting more than one floor.

Roof Live Load Reduction

N - S Beams (Beams on letter grid lines)

Tributary Area: $A_{T.1} = 744.188\cdot ft^2$

$R_{1.1} := .6$ (ASCE 7-05, Eq. 4-2)

$R_{2.1} := 1$ (ASCE 7-05, Eq. 4-2)

$LL_{R.reduced.1} := \max\left(LL_R\cdot R_{1.1}\cdot R_{2.1}, 12psf\right)$ (ASCE 7-05, Eq. 4-2)

$LL_{R.reduced.1} = 12\cdot psf$

Interior Columns

Tributary Area: $A_{T.2} = 992.25 \cdot ft^2$

$R_{1.2} := .6$ (ASCE 7-05, Eq. 4-2)

$R_{2.2} := 1$ (ASCE 7-05, Eq. 4-2)

$LL_{R.reduced.2} := \max(LL_R \cdot R_{1.2} \cdot R_{2.2}, 12psf)$ (ASCE 7-05, Eq. 4-2)

$LL_{R.reduced.2} = 12 \cdot psf$

Side Columns

Tributary Area: $A_{T.3} = 496.125 \cdot ft^2$

$R_{1.3} := 1.2 - .001 \cdot \frac{A_{T.3}}{ft^2}$ (ASCE 7-05, Eq. 4-2)

$R_{1.3} = 0.704$

$R_{2.3} := 1$ (ASCE 7-05, Eq. 4-2)

$LL_{R.reduced.3} := \max(LL_R \cdot R_{1.3} \cdot R_{2.3}, 12psf)$ (ASCE 7-05, Eq. 4-2)

$LL_{R.reduced.3} = 14.078 \cdot psf$

Snow Load

$p_g := 25psf$

$C_e := .9$

$C_t := 1$

$I := 1.2$

$p_{f.req} := 0.7 \cdot C_e \cdot C_t \cdot I \cdot p_g$

$p_{f.req} = 18.9 \cdot psf$

$p_{f.min} := 20psf \cdot I$

$p_{f.min} = 24 \cdot psf$

$p_f := p_{f.min}$

$p_f = 24 \cdot psf$

Appendix B - Gravity Member Sizing Calculations

The gravity member sizing calculations for the building were carried out using Microsoft Excel.

Steel Joist Design
Design Per Vulcraft Steel Joists and Joist Girders 2003 Manual
Constant Uniform Loading, Simply Supported

Name: Grant Buell
Project: Master's Research
Member: RJ1

(orange)	User entered
(blue)	Important result

INPUT		
Geometric Properties		
Span (ft)	31.5	See floor plan
Trib. width (ft)	5.25	See floor plan
Superimposed Loading		
Dead load (psf)	20	See load calculations
Live/snow load (psf)	24	See load calculations
Joist Properties		
Designation	18K5	See Vulcraft Steel Joist and Joist Girders Manual
Maximum w_{total} (plf)	242	See Vulcraft Steel Joist and Joist Girders Manual
Maximum w_{live} (for Δ_{LL} < L/360) (plf)	132	See Vulcraft Steel Joist and Joist Girders Manual

OUTPUT	
Total Service Load	
w_{total} (plf)	231
w_{live} (plf)	126
End reactions$_{total}$ (k)	3.63825
End reactions$_{dead}$ (k)	1.65375
End reactions$_{live/snow}$ (k)	1.9845
Design Checks	
Okay for total load?	OK
Okay for live load? (Δ_{LL} < L/360)	OK
Okay for live load? (Δ_{LL} < L/240)	OK

Steel Beam Design
Design Per AISC Steel Construction Manual, 13th Edition
Constant Uniform Loading with Either 5 or 0 Equally Spaced Equal Point Loads

Name: Grant Buell
Project: Master's Research
Member: RB1

(orange)	User entered
(blue)	Important result

INPUT			
Geometric Properties			
L (ft)	31.5		See plan
Trib. width (ft)	0		See plan
Supporting back-to-back joists? (Yes/No)	Yes		
Superimposed Loading			
N	5		N = no. of point loads
	Dead	Live/Snow	
P (kips)	3.3075	3.969	See load calcs
Surface load (ksf)	0	0	See load calcs
Line load (klf)	0	0	See load calcs
LRFD Load Factor	1.2	1.6	IBC 1605.2.1
Steel Beam Properties			
E (ksi)	29000		
Size	W18X35		AISC Table 1-1
b_f (in)	6.00		AISC Table 1-1
$\Phi_b M_{nx}$ (k-ft)	249		AISC Table 3-2
$\Phi_v V_{nx}$ (k)	159		AISC Table 3-2
I_x (in^4)	510		AISC Table 1-1
Limit Δ_L to L/___	240		IBC Table 1604.3
Limit Δ_L to _" for veneer	N/A		

OUTPUT			
Loading			
	Dead	Live/Snow	Ultimate
w (klf)	0	0	0
M (kip-ft)	78.1396875	93.767625	243.795825
V (kips)	8.26875	9.9225	25.7985
Deflection			
Max Δ_L (in)	1.575		
Required I_x (LL) (in^4)	358.5157		
Δ_L (in)	1.1072		
Bending Check			
Okay for bending?	OK		
Shear Check			
Okay for shear?	OK		
Deflection Check			
Okay for Δ_L?	OK		
Other			
Adequate width for joist bearing?	OK		

Steel Beam Design
Design Per AISC Steel Construction Manual, 13th Edition
Constant Uniform Loading with Either 5 or 0 Equally Spaced Equal Point Loads

Name:	Grant Buell
Project:	Master's Research
Member:	RB2

(orange)	User entered
(blue)	Important result

INPUT			
Geometric Properties			
L (ft)	31.5		See plan
Trib. width (ft)	5.25		See plan
Supporting back-to-back joists? (Yes/No)	No		
Superimposed Loading			
N	0		N = no. of point loads
	Dead	Live/Snow	
P (kips)	0	0	See load calcs
Surface load (ksf)	0.02	0.024	See load calcs
Line load (klf)	0	0	See load calcs
LRFD Load Factor	1.2	1.6	IBC 1605.2.1
Steel Beam Properties			
E (ksi)	29000		
Size	W12X14		AISC Table 1-1
b_f (in)	4		AISC Table 1-1
$\Phi_b M_{nx}$ (k-ft)	65.2		AISC Table 3-2
$\Phi_v V_{nx}$ (k)	64.3		AISC Table 3-2
I_x (in^4)	88.6		AISC Table 1-1
Limit Δ_L to L/___	240		IBC Table 1604.3
Limit Δ_L to _" for veneer	N/A		

OUTPUT			
Loading			
	Dead	Live/Snow	Ultimate
w (klf)	0.105	0.126	0.3276
M (kip-ft)	13.02328125	15.6279375	40.6326375
V (kips)	1.65375	1.9845	5.1597
Deflection			
Max Δ_L (in)	1.575		
Required I_x (LL) (in^4)	61.1106		
Δ_L (in)	1.0863		
Bending Check			
Okay for bending?	OK		
Shear Check			
Okay for shear?	OK		
Deflection Check			
Okay for Δ_L?	OK		
Other			
Adequate width for joist bearing?	OK		

Steel Beam Design
Design Per AISC Steel Construction Manual, 13th Edition
Constant Uniform Loading with Either 5 or 0 Equally Spaced Equal Point Loads

Name: Grant Buell
Project: Master's Research
Member: RB3

User entered
Important result

INPUT			
Geometric Properties			
L (ft)	31.5		See plan
Trib. width (ft)	0		See plan
Supporting back-to-back joists? (Yes/No)	No		
Superimposed Loading			
N	5		N = no. of point loads
	Dead	Live/Snow	
P (kips)	1.65375	1.9845	See load calcs
Surface load (ksf)	0	0	See load calcs
Line load (klf)	0	0	See load calcs
LRFD Load Factor	1.2	1.6	IBC 1605.2.1
Steel Beam Properties			
E (ksi)	29000		
Size	W8X28		AISC Table 1-1
b_f (in)	5		AISC Table 1-1
$\Phi_b M_{nx}$ (k-ft)	125		AISC Table 3-2
$\Phi_v V_{nx}$ (k)	94.8		AISC Table 3-2
I_x (in^4)	199		AISC Table 1-1
Limit Δ_L to L/___	240		IBC Table 1604.3
Limit Δ_L to _" for veneer	N/A		

OUTPUT			
Loading			
	Dead	Live/Snow	Ultimate
w (klf)	0	0	0
M (kip-ft)	39.06984375	46.8838125	121.8979125
V (kips)	4.134375	4.96125	12.89925
Deflection			
Max Δ_L (in)	1.575		
Required I_x (LL) (in^4)	179.2578		
Δ_L (in)	1.4187		
Bending Check			
Okay for bending?	OK		
Shear Check			
Okay for shear?	OK		
Deflection Check			
Okay for Δ_L?	OK		
Other			
Adequate width for joist bearing?	OK		

Steel Beam Design
Design Per AISC Steel Construction Manual, 13th Edition
Constant Uniform Loading with Either 5 or 0 Equally Spaced Equal Point Loads

Name: Grant Buell
Project: Master's Research
Member: RB4

User entered
Important result

INPUT			
Geometric Properties			
L (ft)	31.5		See plan
Trib. width (ft)	2.625		See plan
Supporting back-to-back joists? (Yes/No)	No		
Superimposed Loading			
N	0		N = no. of point loads
	Dead	Live/Snow	
P (kips)	0	0	See load calcs
Surface load (ksf)	0.02	0.024	See load calcs
Line load (klf)	0	0	See load calcs
LRFD Load Factor	1.2	1.6	IBC 1605.2.1
Steel Beam Properties			
E (ksi)	29000		
Size	W10X12		AISC Table 1-1
b_f (in)	4		AISC Table 1-1
$\Phi_b M_{nx}$ (k-ft)	46.9		AISC Table 3-2
$\Phi_v V_{nx}$ (k)	56.3		AISC Table 3-2
I_x (in^4)	53.8		AISC Table 1-1
Limit Δ_L to L/___	240		IBC Table 1604.3
Limit Δ_L to _" for veneer	N/A		

OUTPUT			
Loading			
	Dead	Live/Snow	Ultimate
w (klf)	0.0525	0.063	0.1638
M (kip-ft)	6.511640625	7.81396875	20.31631875
V (kips)	0.826875	0.99225	2.57985
Deflection			
Max Δ_L (in)	1.575		
Required I_x (LL) (in^4)	30.5553		
Δ_L (in)	0.8945		
Bending Check			
Okay for bending?	OK		
Shear Check			
Okay for shear?	OK		
Deflection Check			
Okay for Δ_L?	OK		
Other			
Adequate width for joist bearing?	OK		

Steel Joist Design
Design Per Vulcraft Steel Joists and Joist Girders 2003 Manual
Constant Uniform Loading, Simply Supported

Name:	Grant Buell
Project:	Master's Research
Member:	FJ1

(orange)	User entered
(blue)	Important result

INPUT		
Geometric Properties		
Span (ft)	31.5	See floor plan
Trib. width (ft)	2.625	See floor plan
Superimposed Loading		
Dead load (psf)	60	See load calculations
Live/snow load (psf)	100	See load calculations
Joist Properties		
Designation	20K10	See Vulcraft Steel Joist and Joist Girders Manual
Maximum w_{total} (plf)	468	See Vulcraft Steel Joist and Joist Girders Manual
Maximum w_{live} (for Δ_{LL} < L/360) (plf)	276	See Vulcraft Steel Joist and Joist Girders Manual

OUTPUT	
Total Service Load	
w_{total} (plf)	420
w_{live} (plf)	262.5
End reactions$_{total}$ (k)	6.615
End reactions$_{dead}$ (k)	2.480625
End reactions$_{live/snow}$ (k)	4.134375
Design Checks	
Okay for total load?	OK
Okay for live load? (Δ_{LL} < L/360)	OK
Okay for live load? (Δ_{LL} < L/240)	OK

Steel Beam Design
Design Per AISC Steel Construction Manual, 13th Edition
Constant Uniform Loading with Either 5 or 0 Equally Spaced Equal Point Loads

Name:	Grant Buell
Project:	Master's Research
Member:	FB1

(orange)	User entered
(blue)	Important result

INPUT			
Geometric Properties			
L (ft)	31.5		See plan
Trib. width (ft)	0		See plan
Supporting back-to-back joists? (Yes/No)	Yes		
Superimposed Loading			
N	5		N = no. of point loads
	Dead	Live/Snow	
P (kips)	4.96125	8.26875	See load calcs
Surface load (ksf)	0	0	See load calcs
Line load (klf)	0	0	See load calcs
LRFD Load Factor	1.2	1.6	IBC 1605.2.1
Steel Beam Properties			
E (ksi)	29000		
Size	W21X55		AISC Table 1-1
b_f (in)	8.22		AISC Table 1-1
$\Phi_b M_{nx}$ (k-ft)	473		AISC Table 3-2
$\Phi_v V_{nx}$ (k)	234		AISC Table 3-2
I_x (in^4)	1140		AISC Table 1-1
Limit Δ_L to L/___	240		IBC Table 1604.3
Limit Δ_L to _" for veneer	N/A		

OUTPUT			
Loading			
	Dead	Live/Snow	Ultimate
w (klf)	0	0	0
M (kip-ft)	117.2095313	195.3492188	453.2101875
V (kips)	12.403125	20.671875	47.95875
Deflection			
Max Δ_L (in)	1.575		
Required I_x (LL) (in^4)	746.9076		
Δ_L (in)	1.0319		
Bending Check			
Okay for bending?	OK		
Shear Check			
Okay for shear?	OK		
Deflection Check			
Okay for Δ_L?	OK		
Other			
Adequate width for joist bearing?	OK		

Steel Beam Design
Design Per AISC Steel Construction Manual, 13th Edition
Constant Uniform Loading with Either 5 or 0 Equally Spaced Equal Point Loads

Name: Grant Buell
Project: Master's Research
Member: FB2

(orange)	User entered
(blue)	Important result

INPUT			
Geometric Properties			
L (ft)	31.5		See plan
Trib. width (ft)	2.625		See plan
Supporting back-to-back joists? (Yes/No)	No		
Superimposed Loading			
N	0		N = no. of point loads
	Dead	Live/Snow	
P (kips)	0	0	See load calcs
Surface load (ksf)	0.06	0.1	See load calcs
Line load (klf)	0	0	See load calcs
LRFD Load Factor	1.2	1.6	IBC 1605.2.1
Steel Beam Properties			
E (ksi)	29000		
Size	W12X19		AISC Table 1-1
b_f (in)	4.01		AISC Table 1-1
$\Phi_b M_{nx}$ (k-ft)	92.6		AISC Table 3-2
$\Phi_v V_{nx}$ (k)	85.7		AISC Table 3-2
I_x (in^4)	130		AISC Table 1-1
Limit Δ_L to L/___	240		IBC Table 1604.3
Limit Δ_L to _" for veneer	N/A		

OUTPUT			
Loading			
	Dead	Live/Snow	Ultimate
w (klf)	0.1575	0.2625	0.609
M (kip-ft)	19.53492188	32.55820313	75.53503125
V (kips)	2.480625	4.134375	9.59175
Deflection			
Max Δ_L (in)	1.575		
Required I_x (LL) (in^4)	127.3138		
Δ_L (in)	1.5425		
Bending Check			
Okay for bending?	OK		
Shear Check			
Okay for shear?	OK		
Deflection Check			
Okay for Δ_L?	OK		
Other			
Adequate width for joist bearing?	OK		

Steel Beam Design
Design Per AISC Steel Construction Manual, 13th Edition
Constant Uniform Loading with Either 5 or 0 Equally Spaced Equal Point Loads

Name: Grant Buell
Project: Master's Research
Member: FB3

(orange)	User entered
(blue)	Important result

INPUT			
Geometric Properties			
L (ft)	31.5		See plan
Trib. width (ft)	0		See plan
Supporting back-to-back joists? (Yes/No)	No		
Superimposed Loading			
N	5		N = no. of point loads
	Dead	Live/Snow	
P (kips)	2.480625	4.134375	See load calcs
Surface load (ksf)	0	0	See load calcs
Line load (klf)	0.18	0	See load calcs
LRFD Load Factor	1.2	1.6	IBC 1605.2.1
Steel Beam Properties			
E (ksi)	29000		
Size	W18X40		AISC Table 1-1
b_f (in)	6.02		AISC Table 1-1
$\Phi_b M_{nx}$ (k-ft)	294		AISC Table 3-2
$\Phi_v V_{nx}$ (k)	169		AISC Table 3-2
I_x (in^4)	612		AISC Table 1-1
Limit Δ_L to L/___	240		IBC Table 1604.3
Limit Δ_L to _" for veneer	N/A		

OUTPUT			
Loading			
	Dead	Live/Snow	Ultimate
w (klf)	0.18	0	0.216
M (kip-ft)	80.93039063	97.67460938	253.3958438
V (kips)	9.0365625	10.3359375	27.381375
Deflection			
Max Δ_L (in)	1.575		
Required I_x (LL) (in^4)	373.4538		
Δ_L (in)	0.9611		
Bending Check			
Okay for bending?	OK		
Shear Check			
Okay for shear?	OK		
Deflection Check			
Okay for Δ_L?	OK		
Other			
Adequate width for joist bearing?	OK		

Steel Beam Design
Design Per AISC Steel Construction Manual, 13th Edition
Constant Uniform Loading with Either 5 or 0 Equally Spaced Equal Point Loads

Name: Grant Buell
Project: Master's Research
Member: FB4

(orange)	User entered
(blue)	Important result

INPUT			
Geometric Properties			
L (ft)	31.5		See plan
Trib. width (ft)	1.3125		See plan
Supporting back-to-back joists? (Yes/No)	No		
Superimposed Loading			
N	0		N = no. of point loads
	Dead	Live/Snow	
P (kips)	0	0	See load calcs
Surface load (ksf)	0.06	0.1	See load calcs
Line load (klf)	0.18	0	See load calcs
LRFD Load Factor	1.2	1.6	IBC 1605.2.1
Steel Beam Properties			
E (ksi)	29000		
Size	W12X14		AISC Table 1-1
b_f (in)	3.97		AISC Table 1-1
$\Phi_b M_{nx}$ (k-ft)	65.2		AISC Table 3-2
$\Phi_v V_{nx}$ (k)	64.3		AISC Table 3-2
I_x (in^4)	88.6		AISC Table 1-1
Limit Δ_L to L/___	240		IBC Table 1604.3
Limit Δ_L to _" for veneer	N/A		

OUTPUT			
Loading			
	Dead	Live/Snow	Ultimate
w (klf)	0.25875	0.13125	0.5205
M (kip-ft)	32.09308594	16.27910156	64.55826563
V (kips)	4.0753125	2.0671875	8.197875
Deflection			
Max Δ_L (in)	1.575		
Required I_x (LL) (in^4)	63.6569		
Δ_L (in)	1.1316		
Bending Check			
Okay for bending?	OK		
Shear Check			
Okay for shear?	OK		
Deflection Check			
Okay for Δ_L?	OK		
Other			
Adequate width for joist bearing?	OK		

Steel Column Design
Design Per AISC Steel Construction Manual, 13th Edition
Gravity Only

Name: Grant Buell
Project: Master's Research
Member: C1-1

User entered
Important result

INPUT		
Column Properties		
Size	W8X31	
K	0.5	
L (ft)	36	
$\Phi_c P_n$ (kip)	178	AISC Manual Table 4-1
Beam Reactions onto Column		See beam/joist spreadsheets, load calculations
Dead	Live	
4.134375	4.96125	RB3
0.826875	0.99225	RB4
6.2015625	10.3359375	FB3
4.0753125	2.0671875	FB4
Load Factors		
Dead	Live	
1.2	1.6	
OUTPUT		
Loads		
Total Dead	Total Live	
15.238125	18.356625	
Effective Length		
KL (ft)	18	
Compression Check		
Factored Load P_u (k)	47.65635	
$\Phi_c P_n > P_u$?	OK	

Steel Column Design
Design Per AISC Steel Construction Manual, 13th Edition
Gravity Only

Name:	Grant Buell
Project:	Master's Research
Member:	C1-2

(orange)	User entered
(blue)	Important result

INPUT		
Column Properties		
Size	W8X31	
K	0.5	
L (ft)	36	
$\Phi_c P_n$ (kip)	178	AISC Manual Table 4-1
Beam Reactions onto Column		See beam/joist spreadsheets, load calculations
Dead	Live	
6.2015625	10.3359375	FB3
4.0753125	2.0671875	FB4
6.2015625	10.3359375	FB3
4.0753125	2.0671875	FB4
15.238125	18.356625	C1-1
Load Factors		
Dead	Live	
1.2	1.6	

OUTPUT	
Loads	
Total Dead	Total Live
35.791875	43.162875
Effective Length	
KL (ft)	18
Compression Check	
Factored Load P_u (k)	112.01085
$\Phi_c P_n > P_u$?	OK

Steel Column Design
Design Per AISC Steel Construction Manual, 13th Edition
Gravity Only

Name:	Grant Buell
Project:	Master's Research
Member:	C1-3

(orange)	User entered
(blue)	Important result

INPUT		
Column Properties		
Size	W8X31	
K	0.5	
L (ft)	36	
$\Phi_c P_n$ (kip)	178	AISC Manual Table 4-1
Beam Reactions onto Column		See beam/joist spreadsheets, load calculations
Dead	Live	
6.2015625	10.3359375	FB3
4.0753125	2.0671875	FB4
6.2015625	10.3359375	FB3
4.0753125	2.0671875	FB4
35.791875	43.162875	C1-2
Load Factors		
Dead	Live	
1.2	1.6	
OUTPUT		
Loads		
Total Dead	Total Live	
56.345625	67.969125	
Effective Length		
KL (ft)	18	
Compression Check		
Factored Load P_u (k)	176.36535	
$\Phi_c P_n > P_u$?	OK	

Steel Column Design
Design Per AISC Steel Construction Manual, 13th Edition
Gravity Only

Name:	Grant Buell
Project:	Master's Research
Member:	C1-4

(orange)	User entered
(blue)	Important result

INPUT		
Column Properties		
Size	W10X45	
K	0.5	
L (ft)	36	
$\Phi_c P_n$ (kip)	256	AISC Manual Table 4-1
Beam Reactions onto Column		See beam/joist spreadsheets, load calculations
Dead	Live	
6.2015625	10.3359375	FB3
4.0753125	2.0671875	FB4
6.2015625	10.3359375	FB3
4.0753125	2.0671875	FB4
56.345625	67.969125	C1-3
Load Factors		
Dead	Live	
1.2	1.6	
OUTPUT		
Loads		
Total Dead	Total Live	
76.899375	92.775375	
Effective Length		
KL (ft)	18	
Compression Check		
Factored Load P_u (k)	240.71985	
$\Phi_c P_n > P_u$?	OK	

Steel Column Design
Design Per AISC Steel Construction Manual, 13th Edition
Gravity Only

Name: Grant Buell
Project: Master's Research
Member: C1-5

(orange)	User entered
(blue)	Important result

INPUT		
Column Properties		
Size	W10X49	
K	0.5	
L (ft)	36	
$\Phi_c P_n$ (kip)	383	AISC Manual Table 4-1
Beam Reactions onto Column		See beam/joist spreadsheets, load calculations
Dead	Live	
6.2015625	10.3359375	FB3
4.0753125	2.0671875	FB4
6.2015625	10.3359375	FB3
4.0753125	2.0671875	FB4
76.899375	92.775375	C1-4
Load Factors		
Dead	Live	
1.2	1.6	
OUTPUT		
Loads		
Total Dead	Total Live	
97.453125	117.581625	
Effective Length		
KL (ft)	18	
Compression Check		
Factored Load P_u (k)	305.07435	
$\Phi_c P_n > P_u$?	OK	

Steel Column Design
Design Per AISC Steel Construction Manual, 13th Edition
Gravity Only

Name: Grant Buell
Project: Master's Research
Member: C2-1

User entered	(orange)
Important result	(blue)

INPUT		
Column Properties		
Size	W8X31	
K	0.5	
L (ft)	36	
$\Phi_c P_n$ (kip)	178	AISC Manual Table 4-1
Beam Reactions onto Column		See beam/joist spreadsheets, load calculations
Dead	Live	
1.65375	1.9845	RB2
4.134375	4.96125	RB3
4.134375	4.96125	RB3
2.480625	4.134375	FB2
9.0365625	10.3359375	FB3
9.0365625	10.3359375	FB3
Load Factors		
Dead	Live	
1.2	1.6	

OUTPUT	
Loads	
Total Dead	Total Live
30.47625	36.71325
Effective Length	
KL (ft)	18
Compression Check	
Factored Load P_u (k)	95.3127
$\Phi_c P_n > P_u$?	OK

Steel Column Design
Design Per AISC Steel Construction Manual, 13th Edition
Gravity Only

Name: Grant Buell
Project: Master's Research
Member: C2-2

User entered	(orange)
Important result	(blue)

INPUT		
Column Properties		
Size	W10X39	
K	0.5	
L (ft)	36	
$\Phi_c P_n$ (kip)	216	AISC Manual Table 4-1
Beam Reactions onto Column		See beam/joist spreadsheets, load calculations
Dead	Live	
2.480625	4.134375	FB2
6.2015625	10.3359375	FB3
6.2015625	10.3359375	FB3
2.480625	4.134375	FB2
9.0365625	10.3359375	FB3
9.0365625	10.3359375	FB3
24.80625	36.71325	C2-1
Load Factors		
Dead	Live	
1.2	1.6	
OUTPUT		
Loads		
Total Dead	Total Live	
60.24375	86.32575	
Effective Length		
KL (ft)	18	
Compression Check		
Factored Load P_u (k)	210.4137	
$\Phi_c P_n > P_u$?	OK	

Steel Column Design
Design Per AISC Steel Construction Manual, 13th Edition
Gravity Only

Name:	Grant Buell
Project:	Master's Research
Member:	C2-3

(orange)	User entered
(blue)	Important result

INPUT		
Column Properties		
Size	W10X49	
K	0.5	
L (ft)	36	
$\Phi_c P_n$ (kip)	383	AISC Manual Table 4-1
Beam Reactions onto Column		See beam/joist spreadsheets, load calculations
Dead	Live	
2.480625	4.134375	FB2
6.2015625	10.3359375	FB3
6.2015625	10.3359375	FB3
2.480625	4.134375	FB2
9.0365625	10.3359375	FB3
9.0365625	10.3359375	FB3
54.57375	86.32575	C2-2
Load Factors		
Dead	Live	
1.2	1.6	

OUTPUT	
Loads	
Total Dead	Total Live
90.01125	135.93825
Effective Length	
KL (ft)	18
Compression Check	
Factored Load P_u (k)	325.5147
$\Phi_c P_n > P_u$?	OK

Steel Column Design
Design Per AISC Steel Construction Manual, 13th Edition
Gravity Only

Name:	Grant Buell
Project:	Master's Research
Member:	C2-4

(orange)	User entered
(blue)	Important result

INPUT		
Column Properties		
Size	W12X58	
K	0.5	
L (ft)	36	
$\Phi_c P_n$ (kip)	446	AISC Manual Table 4-1
Beam Reactions onto Column		See beam/joist spreadsheets, load calculations
Dead	Live	
2.480625	4.134375	FB2
6.2015625	10.3359375	FB3
6.2015625	10.3359375	FB3
2.480625	4.134375	FB2
9.0365625	10.3359375	FB3
9.0365625	10.3359375	FB3
84.34125	135.93825	C2-3
Load Factors		
Dead	Live	
1.2	1.6	
OUTPUT		
Loads		
Total Dead	Total Live	
119.77875	185.55075	
Effective Length		
KL (ft)	18	
Compression Check		
Factored Load P_u (k)	440.6157	
$\Phi_c P_n > P_u$?	OK	

Steel Column Design
Design Per AISC Steel Construction Manual, 13th Edition
Gravity Only

Name: Grant Buell
Project: Master's Research
Member: C2-5

User entered	
Important result	

INPUT		
Column Properties		
Size	W12X65	
K	0.5	
L (ft)	36	
$\Phi_c P_n$ (kip)	591	AISC Manual Table 4-1
Beam Reactions onto Column		See beam/joist spreadsheets, load calculations
Dead	Live	
2.480625	4.134375	FB2
6.2015625	10.3359375	FB3
6.2015625	10.3359375	FB3
2.480625	4.134375	FB2
9.0365625	10.3359375	FB3
9.0365625	10.3359375	FB3
114.10875	185.55075	C2-4
Load Factors		
Dead	Live	
1.2	1.6	
OUTPUT		
Loads		
Total Dead	Total Live	
149.54625	235.16325	
Effective Length		
KL (ft)	18	
Compression Check		
Factored Load P_u (k)	555.7167	
$\Phi_c P_n > P_u$?	OK	

Steel Column Design
Design Per AISC Steel Construction Manual, 13th Edition
Gravity Only

Name: Grant Buell
Project: Master's Research
Member: C3-1

User entered
Important result

INPUT		
Column Properties		
Size	W8X31	
K	0.5	
L (ft)	36	
$\Phi_c P_n$ (kip)	178	AISC Manual Table 4-1
Beam Reactions onto Column		See beam/joist spreadsheets, load calculations
Dead	Live	
8.26875	9.9225	RB1
0.826875	0.99225	RB4
0.826875	0.99225	RB4
12.403125	20.671875	FB1
4.0753125	2.0671875	FB4
4.0753125	2.0671875	FB4
Load Factors		
Dead	Live	
1.2	1.6	
OUTPUT		
Loads		
Total Dead	Total Live	
30.47625	36.71325	
Effective Length		
KL (ft)	18	
Compression Check		
Factored Load P_u (k)	95.3127	
$\Phi_c P_n > P_u$?	OK	

Steel Column Design
Design Per AISC Steel Construction Manual, 13th Edition
Gravity Only

Name: Grant Buell
Project: Master's Research
Member: C3-2

(orange)	User entered
(blue)	Important result

INPUT		
Column Properties		
Size	W8X40	
K	0.5	
L (ft)	36	
$\Phi_c P_n$ (kip)	233	AISC Manual Table 4-1
Beam Reactions onto Column		See beam/joist spreadsheets, load calculations
Dead	Live	
12.403125	20.671875	FB1
4.0753125	2.0671875	FB4
4.0753125	2.0671875	FB4
12.403125	20.671875	FB1
4.0753125	2.0671875	FB4
4.0753125	2.0671875	FB4
30.47625	36.71325	C3-1
Load Factors		
Dead	Live	
1.2	1.6	

OUTPUT	
Loads	
Total Dead	Total Live
71.58375	86.32575
Effective Length	
KL (ft)	18
Compression Check	
Factored Load P_u (k)	224.0217
$\Phi_c P_n > P_u$?	OK

Steel Column Design
Design Per AISC Steel Construction Manual, 13th Edition
Gravity Only

Name:	Grant Buell
Project:	Master's Research
Member:	C3-3

(orange)	User entered
(blue)	Important result

INPUT		
Column Properties		
Size	W10X49	
K	0.5	
L (ft)	36	
$\Phi_c P_n$ (kip)	383	AISC Manual Table 4-1
Beam Reactions onto Column		See beam/joist spreadsheets, load calculations
Dead	Live	
12.403125	20.671875	FB1
4.0753125	2.0671875	FB4
4.0753125	2.0671875	FB4
12.403125	20.671875	FB1
4.0753125	2.0671875	FB4
4.0753125	2.0671875	FB4
71.58375	86.32575	C3-2
Load Factors		
Dead	Live	
1.2	1.6	
OUTPUT		
Loads		
Total Dead	Total Live	
112.69125	135.93825	
Effective Length		
KL (ft)	18	
Compression Check		
Factored Load P_u (k)	352.7307	
$\Phi_c P_n > P_u$?	OK	

Steel Column Design
Design Per AISC Steel Construction Manual, 13th Edition
Gravity Only

Name: Grant Buell
Project: Master's Research
Member: C3-4

	User entered
	Important result

INPUT		
Column Properties		
Size	W12X65	
K	0.5	
L (ft)	36	
$\Phi_c P_n$ (kip)	591	AISC Manual Table 4-1
Beam Reactions onto Column		See beam/joist spreadsheets, load calculations
Dead	Live	
12.403125	20.671875	FB1
4.0753125	2.0671875	FB4
4.0753125	2.0671875	FB4
12.403125	20.671875	FB1
4.0753125	2.0671875	FB4
4.0753125	2.0671875	FB4
112.69125	135.93825	C3-3
Load Factors		
Dead	Live	
1.2	1.6	

OUTPUT	
Loads	
Total Dead	Total Live
153.79875	185.55075
Effective Length	
KL (ft)	18
Compression Check	
Factored Load P_u (k)	481.4397
$\Phi_c P_n > P_u$?	OK

Steel Column Design
Design Per AISC Steel Construction Manual, 13th Edition
Gravity Only

Name: Grant Buell
Project: Master's Research
Member: C3-5

User entered
Important result

INPUT		
Column Properties		
Size	W12X72	
K	0.5	
L (ft)	36	
$\Phi_c P_n$ (kip)	657	AISC Manual Table 4-1
Beam Reactions onto Column		See beam/joist spreadsheets, load calculations
Dead	Live	
12.403125	20.671875	FB1
4.0753125	2.0671875	FB4
4.0753125	2.0671875	FB4
12.403125	20.671875	FB1
4.0753125	2.0671875	FB4
4.0753125	2.0671875	FB4
153.79875	185.55075	C3-4
Load Factors		
Dead	Live	
1.2	1.6	
OUTPUT		
Loads		
Total Dead	Total Live	
194.90625	235.16325	
Effective Length		
KL (ft)	18	
Compression Check		
Factored Load P_u (k)	610.1487	
$\Phi_c P_n > P_u$?	OK	

Steel Column Design
Design Per AISC Steel Construction Manual, 13th Edition
Gravity Only

Name: Grant Buell
Project: Master's Research
Member: C4-1

User entered	(orange)
Important result	(blue)

INPUT		
Column Properties		
Size	W8X31	
K	0.5	
L (ft)	36	
$\Phi_c P_n$ (kip)	178	AISC Manual Table 4-1
Beam Reactions onto Column		See beam/joist spreadsheets, load calculations
Dead	Live	
8.26875	9.9225	RB1
8.26875	9.9225	RB1
1.65375	1.9845	RB2
1.65375	1.9845	FB2
12.403125	20.671875	FB1
12.403125	20.671875	FB1
2.480625	4.134375	FB2
2.480625	4.134375	FB2
Load Factors		
Dead	Live	
1.2	1.6	

OUTPUT	
Loads	
Total Dead	Total Live
49.6125	73.4265
Effective Length	
KL (ft)	18
Compression Check	
Factored Load P_u (k)	177.0174
$\Phi_c P_n > P_u$?	OK

Steel Column Design
Design Per AISC Steel Construction Manual, 13th Edition
Gravity Only

Name:	Grant Buell
Project:	Master's Research
Member:	C4-2

(orange)	User entered
(blue)	Important result

INPUT		
Column Properties		
Size	W10X54	
K	0.5	
L (ft)	36	
$\Phi_c P_n$ (kip)	423	AISC Manual Table 4-1
Beam Reactions onto Column		See beam/joist spreadsheets, load calculations
Dead	Live	
12.403125	20.671875	FB1
12.403125	20.671875	FB1
2.480625	4.134375	FB2
2.480625	4.134375	FB2
12.403125	20.671875	FB1
12.403125	20.671875	FB1
2.480625	4.134375	FB2
2.480625	4.134375	FB2
49.6125	73.4265	C4-1
Load Factors		
Dead	Live	
1.2	1.6	

OUTPUT	
Loads	
Total Dead	Total Live
109.1475	172.6515
Effective Length	
KL (ft)	18
Compression Check	
Factored Load P_u (k)	407.2194
$\Phi_c P_n > P_u$?	OK

Steel Column Design
Design Per AISC Steel Construction Manual, 13th Edition
Gravity Only

Name: Grant Buell
Project: Master's Research
Member: C4-3

User entered	(orange)
Important result	(blue)

INPUT		
Column Properties		
Size	W12X72	
K	0.5	
L (ft)	36	
$\Phi_c P_n$ (kip)	657	AISC Manual Table 4-1
Beam Reactions onto Column		See beam/joist spreadsheets, load calculations
Dead	Live	
12.403125	20.671875	FB1
12.403125	20.671875	FB1
2.480625	4.134375	FB2
2.480625	4.134375	FB2
12.403125	20.671875	FB1
12.403125	20.671875	FB1
2.480625	4.134375	FB2
2.480625	4.134375	FB2
109.1475	172.6515	C4-2
Load Factors		
Dead	Live	
1.2	1.6	

OUTPUT	
Loads	
Total Dead	Total Live
168.6825	271.8765
Effective Length	
KL (ft)	18
Compression Check	
Factored Load P_u (k)	637.4214
$\Phi_c P_n > P_u$?	OK

Steel Column Design
Design Per AISC Steel Construction Manual, 13th Edition
Gravity Only

Name:	Grant Buell
Project:	Master's Research
Member:	C4-4

(orange)	User entered
(blue)	Important result

INPUT		
Column Properties		
Size	W14X90	
K	0.5	
L (ft)	36	
$\Phi_c P_n$ (kip)	928	AISC Manual Table 4-1
Beam Reactions onto Column		See beam/joist spreadsheets, load calculations
Dead	Live	
12.403125	20.671875	FB1
12.403125	20.671875	FB1
2.480625	4.134375	FB2
2.480625	4.134375	FB2
12.403125	20.671875	FB1
12.403125	20.671875	FB1
2.480625	4.134375	FB2
2.480625	4.134375	FB2
168.6825	271.8765	C4-3
Load Factors		
Dead	Live	
1.2	1.6	
OUTPUT		
Loads		
Total Dead	Total Live	
228.2175	371.1015	
Effective Length		
KL (ft)	18	
Compression Check		
Factored Load P_u (k)	867.6234	
$\Phi_c P_n > P_u$?	OK	

Steel Column Design
Design Per AISC Steel Construction Manual, 13th Edition
Gravity Only

Name: Grant Buell
Project: Master's Research
Member: C4-5

(orange)	User entered
(blue)	Important result

INPUT		
Column Properties		
Size	W14X109	
K	0.5	
L (ft)	36	
$\Phi_c P_n$ (kip)	1130	AISC Manual Table 4-1
Beam Reactions onto Column		See beam/joist spreadsheets, load calculations
Dead	Live	
12.403125	20.671875	FB1
12.403125	20.671875	FB1
2.480625	4.134375	FB2
2.480625	4.134375	FB2
12.403125	20.671875	FB1
12.403125	20.671875	FB1
2.480625	4.134375	FB2
2.480625	4.134375	FB2
228.2175	371.1015	C4-4
Load Factors		
Dead	Live	
1.2	1.6	
OUTPUT		
Loads		
Total Dead	Total Live	
287.7525	470.3265	
Effective Length		
KL (ft)	18	
Compression Check		
Factored Load P_u (k)	1097.8254	
$\Phi_c P_n > P_u$?	OK	

Appendix C - Wind Load Calculations

The wind load calculations were carried out according to ASCE 7-05 using MathCAD 13.0 and Microsoft Excel.

Method 2: Analytical Procedure

Location: Chicago IL

$V_w := 90$ (ASCE 7-05, Figure 6-1)

$I_w := 1.15$ (ASCE 7-05, Table 6-1)

Surface Roughness Assumed: B (ASCE 7-05, Sec. 6.5.6.2)

Exposure Category Assumed: B (ASCE 7-05, Sec. 6.5.6.3)

$z_g := 1200 \qquad \alpha := 7$ (ASCE 7-05, Table 6-2)

For 0 ft < z < 15 ft:

$$K_{z.15} := 2.01 \cdot \left(\frac{15}{z_g}\right)^{\frac{2}{\alpha}}$$ (ASCE 7-05, Table 6-3)

$$K_{z.15} = 0.575$$

For 15 ft < z < zg:

$$K_z(z) := 2.01 \left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}}$$

Floor 2: $$K_{z.f2} := \frac{6K_{z.15} + \int_{15}^{27} K_z(z)\,dz}{18} \qquad K_{z.f2} = 0.612$$

Floor 3: $$K_{z.f3} := \frac{\int_{27}^{45} K_z(z)\,dz}{18} \qquad K_{z.f3} = 0.736$$

Floor 4: $$K_{z.f4} := \frac{\int_{45}^{63} K_z(z)\,dz}{18} \qquad K_{z.f4} = 0.828$$

Floor 5: $$K_{z.f5} := \frac{\int_{63}^{81} K_z(z)\,dz}{18} \qquad K_{z.f5} = 0.899$$

Floor 6:

$$K_{z.f6} := \frac{\int_{81}^{99} K_z(z)\,dz}{18}$$ $K_{z.f6} = 0.959$

Floor 7:

$$K_{z.f7} := \frac{\int_{99}^{117} K_z(z)\,dz}{18}$$ $K_{z.f7} = 1.01$

Floor 8:

$$K_{z.f8} := \frac{\int_{117}^{135} K_z(z)\,dz}{18}$$ $K_{z.f8} = 1.056$

Floor 9:

$$K_{z.f9} := \frac{\int_{135}^{153} K_z(z)\,dz}{18}$$ $K_{z.f9} = 1.097$

Floor 10:

$$K_{z.f10} := \frac{\int_{153}^{171} K_z(z)\,dz}{18}$$ $K_{z.f10} = 1.134$

Roof:

$$K_{z.fr} := \frac{\int_{171}^{180} K_z(z)\,dz}{9}$$ $K_{z.fr} = 1.16$

$K_d := .85$ (ASCE 7-05, Table 6-4)

$K_{zt} := 1.0$ (ASCE 7-05, Sec. 6.5.7)

$q_z\left(K_z, K_{zt}, K_d, V_w, I_w\right) := .00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V_w^{\,2} \cdot I_w \cdot psf$ (ASCE 7-05, Eq. 6-15)

Floor 2: $q_{z.f2} := q_z\left(K_{z.f2}, K_{zt}, K_d, V_w, I_w\right)$

$q_{z.f2} = 12.409\ psf$

Floor 3: $q_{z.f3} := q_z(K_{z.f3}, K_{zt}, K_d, V_w, I_w)$

$q_{z.f3} = 14.928\ \text{psf}$

Floor 4: $q_{z.f4} := q_z(K_{z.f4}, K_{zt}, K_d, V_w, I_w)$

$q_{z.f4} = 16.781\ \text{psf}$

Floor 5: $q_{z.f5} := q_z(K_{z.f5}, K_{zt}, K_d, V_w, I_w)$

$q_{z.f5} = 18.227\ \text{psf}$

Floor 6: $q_{z.f6} := q_z(K_{z.f6}, K_{zt}, K_d, V_w, I_w)$

$q_{z.f6} = 19.43\ \text{psf}$

Floor 7: $q_{z.f7} := q_z(K_{z.f7}, K_{zt}, K_d, V_w, I_w)$

$q_{z.f7} = 20.471\ \text{psf}$

Floor 8: $q_{z.f8} := q_z(K_{z.f8}, K_{zt}, K_d, V_w, I_w)$

$q_{z.f8} = 21.395\ \text{psf}$

Floor 9: $q_{z.f9} := q_z(K_{z.f9}, K_{zt}, K_d, V_w, I_w)$

$q_{z.f9} = 22.227\ \text{psf}$

Floor 10: $q_{z.f10} := q_z(K_{z.f10}, K_{zt}, K_d, V_w, I_w)$

$q_{z.f10} = 22.989\ \text{psf}$

Roof: $q_{z.fr} := q_z(K_{z.fr}, K_{zt}, K_d, V_w, I_w)$

$q_{z.fr} = 23.523\ \text{psf}$

$q_h := q_{z.fr}$

$q_h = 23.523\ \text{psf}$

Gust Effect Factor: Assume rigid structure (ASCE 7-05, Sec. 6.5.8)

$G := .85$ (ASCE 7-05, Sec. 6.5.8.1)

Assume building is enclosed (ASCE 7-05, Sec. 6.5.9)

$GC_{pi.plus} := .18$ $GC_{pi.minus} := -.18$ (ASCE 7-05, Fig. 6-5)

Cp for wind in longitudinal direction

$L_l := 8 \cdot 31.5ft$ $\quad B_l := 4 \cdot 31.5ft$ $\quad h := 10 \cdot 18ft$

$L_l = 252\ ft$ $\quad B_l = 126\ ft$ $\quad h = 180\ ft$

$$\frac{L_l}{B_l} = 2$$

$C_{p.l.windward} := .8$ (ASCE 7-05, Fig. 6-6)

$C_{p.l.leeward} := -0.3$

$C_{p.l.side} := -.7$

$$\frac{h}{L_l} = 0.714$$

$$C_{p.l..roof.0.to.h.over.2} := \frac{\frac{h}{L_l} - .5}{1 - .5} \cdot (-1.3 - -.9) + -.9$$

$C_{p.l..roof.0.to.h.over.2} = -1.071$

$$C_{p.l.roof.h.over.2.to.h} := \frac{\frac{h}{L_l} - .5}{1 - .5} \cdot (-.7 - -.9) + -.9$$

$C_{p.l.roof.h.over.2.to.h} = -0.814$

$$C_{p.l.roof.over.h} := \frac{\frac{h}{L_l} - .5}{1 - .5} \cdot (-.7 - -.5) + -.5$$

$C_{p.l.roof.over.h} = -0.586$

$C_{p.l.alt} := -.18$

Cp for wind in transverse direction

$L_t := 4 \cdot 31.5ft$ $\quad B_t := 8 \cdot 31.5ft$

$L_t = 126\ ft$ $\quad B_t = 252\ ft$ $\quad h = 180\ ft$

$$\frac{L_t}{B_t} = 0.5$$

$C_{p.t.windward} := .8$ (ASCE 7-05, Fig. 6-6)

$C_{p.t.leeward} := -0.5$

$C_{p.t.side} := -.7$

$\frac{h}{L_t} = 1.429$

$C_{p.t.roof.0.to.h.over.2.original} := -1.3$

$A_{applicable} := \frac{h}{2} \cdot B_t$

$A_{applicable} = 22680\ ft^2$

Reduction := .8

$C_{p.t.roof.0.to.h.over.2} := C_{p.t.roof.0.to.h.over.2.original} \cdot Reduction$

$C_{p.t.roof.0.to.h.over.2} = -1.04$

$C_{p.t.roof.over.h.over.2} := -.7$

$C_{p.t.alt} := -.18$

MWFRS pressures based on these loads:

h (ft)	180
h/2 (ft)	90
G	0.85
GC_{pi}	0.18
	-0.18
$q_i = q_h$ (psf)	23.523

C_p			
		Wind Direction	
		Longitudinal	Transverse
Walls	Windward	0.8	0.8
	Leeward	-0.3	-0.5
	Side	-0.7	-0.7
Roof	0 < x < h/2	-1.071	-1.04
	h/2 < x < h	-0.814	-0.7
	h < x	-0.586	-0.7
	Alternative	-0.18	-0.18

q_z (psf)		
Floor	Roof	23.523
	10	22.989
	9	22.227
	8	21.395
	7	20.471
	6	19.43
	5	18.227
	4	16.781
	3	14.928
	2	12.409

$p = qGC_p - q_i(GC_{pi})$

$q = q_z$ for windward walls, q_h for roofs and leeward and side walls

Wind Pressure (Considering Positive GC_{pi}) (Longitudinal Direction) (psf)								
		Walls			Roof			
		Windward	Leeward	Side	0 ft < x < 90 ft	90 ft < x < 180 ft	h < 180 ft	Alternative
Floor	Roof	11.7615	-10.2325	-18.2303	-25.648303	-20.50970	-15.95094	-7.833159
	10	11.39838	-10.2325	-18.2303	-25.648303	-20.50970	-15.95094	-7.833159
	9	10.88022	-10.2325	-18.2303	-25.648303	-20.50970	-15.95094	-7.833159
	8	10.31446	-10.2325	-18.2303	-25.648303	-20.50970	-15.95094	-7.833159
	7	9.68614	-10.2325	-18.2303	-25.648303	-20.50970	-15.95094	-7.833159
	6	8.97826	-10.2325	-18.2303	-25.648303	-20.50970	-15.95094	-7.833159
	5	8.16022	-10.2325	-18.2303	-25.648303	-20.50970	-15.95094	-7.833159
	4	7.17694	-10.2325	-18.2303	-25.648303	-20.50970	-15.95094	-7.833159
	3	5.9169	-10.2325	-18.2303	-25.648303	-20.50970	-15.95094	-7.833159
	2	4.20398	-10.2325	-18.23032	-25.648303	-20.50970	-15.95094	-7.833159

Wind Pressure (Considering Negative GC_{pi}) (Longitudinal Direction) (psf)								
		Walls			Roof			
		Windward	Leeward	Side	0 ft < x < 90 ft	90 ft < x < 180 ft	h < 180 ft	Alternative
Floor	Roof	20.22978	-1.7642	-9.7620	-17.1800	-12.04142	-7.4826	0.635121
	10	19.86666	-1.7642	-9.7620	-17.1800	-12.04142	-7.4826	0.635121
	9	19.3485	-1.7642	-9.7620	-17.1800	-12.04142	-7.4826	0.635121
	8	18.78274	-1.7642	-9.7620	-17.1800	-12.04142	-7.4826	0.635121
	7	18.15442	-1.7642	-9.7620	-17.1800	-12.04142	-7.4826	0.635121
	6	17.44654	-1.7642	-9.7620	-17.1800	-12.04142	-7.4826	0.635121
	5	16.6285	-1.7642	-9.7620	-17.1800	-12.041423	-7.4826	0.635121
	4	15.64522	-1.7642	-9.7620	-17.1800	-12.04142	-7.4826	0.635121
	3	14.38518	-1.7642	-9.7620	-17.1800	-12.04142	-7.4826	0.635121
	2	12.67226	-1.7642	-9.7620	-17.1800	-12.04142	-7.4826	0.635121

Wind Pressure (Considering Positive GC_{pi}) (Transverse Direction) (psf)								
		Walls			Roof			
		Windward	Leeward	Side	0 ft < x < 90 ft	90 ft < x < 180 ft	h < 180 ft	Alternative
Floor	Roof	11.7615	-14.2314	-18.2303	-25.0284	-18.2303	-18.2303	-7.833159
	10	11.39838	-14.2314	-18.2303	-25.0284	-18.2303	-18.2303	-7.833159
	9	10.88022	-14.2314	-18.2303	-25.0284	-18.2303	-18.2303	-7.833159
	8	10.31446	-14.2314	-18.2303	-25.0284	-18.2303	-18.2303	-7.833159
	7	9.68614	-14.2314	-18.2303	-25.0284	-18.2303	-18.2303	-7.833159
	6	8.97826	-14.2314	-18.2303	-25.0284	-18.2303	-18.2303	-7.833159
	5	8.16022	-14.2314	-18.2303	-25.0284	-18.2303	-18.2303	-7.833159
	4	7.17694	-14.2314	-18.2303	-25.0284	-18.2303	-18.2303	-7.833159
	3	5.9169	-14.2314	-18.2303	-25.0284	-18.2303	-18.2303	-7.833159
	2	4.20398	-14.2314	-18.2303	-25.0284	-18.2303	-18.2303	-7.833159

Wind Pressure (Considering Negative GC_{pi}) (Transverse Direction) (psf)								
		Walls			Roof			
		Windward	Leeward	Side	0 ft < x < 90 ft	90 ft < x < 180 ft	h < 180 ft	Alternative
Floor	Roof	20.22978	-5.7631	-9.7620	-16.560192	-9.762045	-9.7620	0.635121
	10	19.86666	-5.7631	-9.7620	-16.560192	-9.762045	-9.7620	0.635121
	9	19.3485	-5.7631	-9.7620	-16.560192	-9.762045	-9.7620	0.635121
	8	18.78274	-5.7631	-9.7620	-16.560192	-9.762045	-9.7620	0.635121
	7	18.15442	-5.7631	-9.7620	-16.560192	-9.762045	-9.7620	0.635121
	6	17.44654	-5.7631	-9.7620	-16.560192	-9.762045	-9.7620	0.635121
	5	16.6285	-5.7631	-9.7620	-16.560192	-9.762045	-9.7620	0.635121
	4	15.64522	-5.7631	-9.7620	-16.560192	-9.762045	-9.7620	0.635121
	3	14.38518	-5.7631	-9.7620	-16.560192	-9.762045	-9.7620	0.635121
	2	12.67226	-5.7631	-9.7620	-16.560192	-9.762045	-9.7620	0.635121

Note: GC_{pi} direction does not affect total horizontal forces, only roof pressures.

CASE 1 - Apply Longitudinal and Transverse Forces Separately									
		Wind Direction							
		Longitudinal				Transverse			
		Wall Area (ft^2)	Windward Force (lbs)	Leeward Force (lbs)	W (kips)	Wall Area (ft^2)	Windward Force (lbs)	Leeward Force (lbs)	W (kips)
Floor	Roof	1134	13337.541	-11603.6	24.94	2268	26675.082	-32276.84	58.951
	10	2268	25851.525	-23207.3	49.05	4536	51703.051	-64553.69	116.25
	9	2268	24676.338	-23207.3	47.88	4536	49352.677	-64553.69	113.90
	8	2268	23393.195	-23207.3	46.60	4536	46786.390	-64553.69	111.34
	7	2268	21968.165	-23207.3	45.17	4536	43936.331	-64553.69	108.49
	6	2268	20362.693	-23207.3	43.57	4536	40725.387	-64553.69	105.27
	5	2268	18507.378	-23207.3	41.71	4536	37014.757	-64553.69	101.56
	4	2268	16277.299	-23207.3	39.48	4536	32554.599	-64553.69	97.108
	3	2268	13419.529	-23207.3	36.62	4536	26839.058	-64553.69	91.392
	2	2268	9534.626	-23207.3	32.74	4536	19069.253	-64553.69	83.622

CASE 2 - Apply Longitudinal and Transverse Forces and Moments Separately CASE 3 - Apply Longitudinal and Transverse Forces Together, Do Not Apply Moments							
		Wind Direction					
		Longitudinal			Transverse		
		.75W (kips)	e (ft)	M_T (k-ft)	.75W (kips)	e (ft)	M_T (k-ft)
Floor	Roof	18.70590125	18.9	353.5415337	44.21394842	37.8	1671.28725
	10	36.79413539	18.9	695.4091588	87.19256259	37.8	3295.878866
	9	35.91274523	18.9	678.7508848	85.42978227	37.8	3229.24577
	8	34.95038747	18.9	660.5623231	83.50506675	37.8	3156.491523
	7	33.88161515	18.9	640.3625262	81.36752211	37.8	3075.692336
	6	32.67751127	18.9	617.6049629	78.95931435	37.8	2984.662082
	5	31.28602523	18.9	591.3058768	76.17634227	37.8	2879.465738
	4	29.61346595	18.9	559.6945064	72.83122371	37.8	2753.020256
	3	27.47013791	18.9	519.1856064	68.54456763	37.8	2590.984656
	2	24.55646099	18.9	464.1171126	62.71721379	37.8	2370.710681

CASE 4 - Apply Longitudinal and Transverse Forces and Moments Together							
		Wind Direction					
		Longitudinal			Transverse		
		.563W (kips)	e (ft)	M_T (k-ft)	.563W (kips)	e (ft)	M_T (k-ft)
Floor	Roof	14.04189654	18.9	265.3918446	33.18993728	37.8	1254.579629
	10	27.62013096	18.9	522.0204752	65.45255032	37.8	2474.106402
	9	26.95850075	18.9	509.5156642	64.12928989	37.8	2424.087158
	8	26.23609086	18.9	495.8621172	62.68447011	37.8	2369.47297
	7	25.4337991	18.9	480.698803	61.0798866	37.8	2308.819713
	6	24.52991846	18.9	463.6154588	59.27212531	37.8	2240.486337
	5	23.48537627	18.9	443.8736115	57.18304093	37.8	2161.518947
	4	22.22984177	18.9	420.1440094	54.67197193	37.8	2066.600539
	3	20.62091685	18.9	389.7353285	51.4541221	37.8	1944.965815
	2	18.43371671	18.9	348.3972459	47.07972182	37.8	1779.613485

Appendix D - Permissions

From: "Petzoldt, Natalie" <npetzoldt@CANNONDESIGN.COM>

Date: April 6, 2009 9:52:42 PM CDT

To: Grant Buell <gbuell@gmail.com>

Cc: "Sun, Ruofei" <rsun@CANNONDESIGN.COM>

Subject: RE: Universal Grid Project Master's Report

Grant - good to hear from you and hope all is well in Manhattan. Glad to know that you are still working on this project involving our study of the universal grid concept. We have been implementing this and exploring it further on our latest projects as well. Yes, you can use select images from the Cannon Design presentation for your project as long as you provide appropriate credit to us. When you are finished with the project, we would love to see what you've developed.

Best of luck as you finish the year.

Natalie Petzoldt, AIA, LEED™AP

Vice President

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