Courtyard by Marriott

Lancaster, PA

Danielle Shetler – Structural Option

Senior Thesis, April 2005

Department of Architectural Engineering

The Pennsylvania State University

Courtyard by Marriott
Lancaster, PA

Project Team:

- Owner – High Hotels, Ltd.
- Developers – High Associates, Ltd.
- Structural Engineer – Baker, Ingram & Associates
- Mechanical/Electrical Engineer – Barton Associates, Inc.
- Design-Builder – High Construction Company
- Architect – Greenfield Architects, Ltd.

Structural:

- Foundation – spread footings and slab on grade (4" and 6" concrete slabs)
- Masonry bearing walls
- Structural steel beams
- Floor system – 8" pre-cast planks
- Lateral loads resisted by shear walls and moment frames

Architecture:

- Exterior façade is an Exterior Insulated Finish System (E.I.F.S.).
- ½" and 1" E.I.F.S. quartz putz DPR finish - pearl, canvas and prairie clay
- Spandrel glass and curtain wall systems are on the first floor
- Aluminum windows are most common throughout the building
- Roof - fiberglass shingles-dimensional

Construction:

- 83,821 sq ft
- 5 stories high
- Total Job Cost - $9,550,000.00
- Project Delivery Method - Design Build
- Constructions Dates:
  - Start - July 12, 2004
  - Finish - June 30, 2005

Mechanical:

- Split System Heat Pump – 10 units
- Thru wall heat pump, each room – 245 cfm
- Pool Heating-Cooling Unit – 3000 cfm
- Roof Equipment:
  - ♦ 2 Roof Ventilators
  - ♦ 6 Fans – 960 cfm-1940 cfm
  - ♦ Kitchen Ventilation Unit – 2400 cfm

Lighting/Electrical:

- Main Distribution Switchboard
  - ♦ 277/480 volts - 3 phase -4 wires
  - ♦ Provide 3000 amps bus, braced for 65,000 amps sym.
- Distribution Switchboard
  - ♦ 120/208 volts – 3 phase – 4 wires
  - ♦ Provide 1600 amp bus, braced for 35,000 amps sym.
- Fluorescent & incandescent lighting
- Metal Halide lighting outside

Danielle Shetler – Structural Option
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Executive Summary

For this thesis study, it was proposed that the existing structural system of the Courtyard by Marriot in Lancaster, PA be redesigned using the staggered truss system. It was also proposed that a green roof be incorporated in the redesign of the building. A cost analysis of the existing system and the new system was also to be performed.

The existing structural system of the Marriott Hotel consists of load bearing masonry walls, masonry shear walls and moment frames. The exterior walls are load bearing masonry walls, while interior walls are masonry shear walls. The 2nd floor is steel framed with moment connections in both (transverse and longitudinal) directions and used as a transfer the load from the shear walls above. This system is going to be replaced with the staggered truss system which consists of story high steel trusses that span the entire transverse direction of the building. The trusses are placed at on alternating floors and column lines.

A design of the staggered truss system was done by first using the AISC Design Guide 14: Staggered Truss Framing Systems and then a computer analysis on ETABS was used in the design. While designing this new system, extra load from the green roof system was taken into account as well as the addition hollow core planks. The hollow core planks are added to the roof in order to make it flat to incorporate the green roof system.

After the hand and computer analysis of the staggered truss system, it was found that this system was very efficient and cost effective, as well as the green roof with all of its benefits. Even though this was found to be an efficient system, it is not recommended for this building. Due to standards placed by Marriott the Courtyard

brand of hotel and the size of it, the current structural design is the most efficient. Though the staggered truss system is not recommended for this building, the green roof would be worth trying to incorporate to the existing system design because it provides many benefits for the building without incurring to much extra load on the building or extra cost.

Introduction/Background Information

Project Team

Architect:
Greenfield Architects, LTD.
1853 William Penn Way
Lancaster, PA 17605

Mechanical/Lighting Consultant:
Barton Associates, Inc
415 Norway Street
York, PA 17403

Code Enforcement:
East Lampeter Township
2205 Old Philadelphia Pike
Lancaster, PA 17602

Owner:
High Hotels, LTD.
1853 William Penn Way
Lancaster, PA 17605

Design-Builder:
High Construction Company
1853 William Penn Way
Lancaster, PA 17605

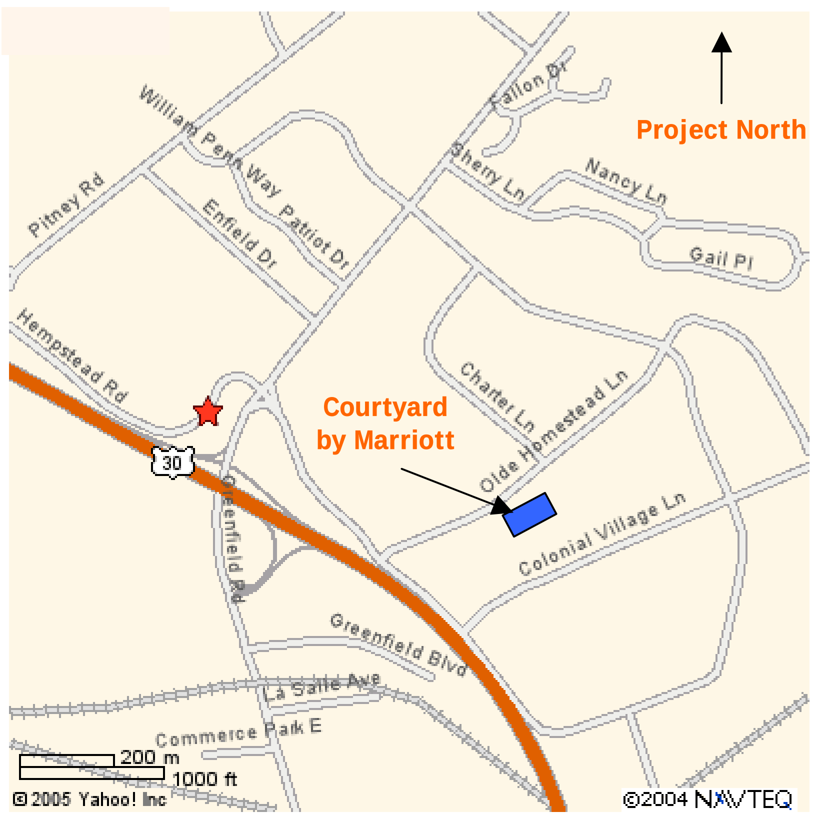
Structural Consultant:
Baker, Ingram & Associates
8 North Queen Street, Suite 212
Lancaster, PA 17603

Building Overview

The Courtyard by Marriott in Lancaster, PA is an 83,821 square foot, 5 story hotel located within the Greenfield Corporate Center. This brand of Marriott hotel is geared more towards business travelers. Large, open spaces are provided on the first floor for a large lobby area, conference rooms and a swimming pool. The first floor height is 12'-9" in order to provide enough open space for these areas. The first floor is approximately 18,000 sq ft. Floors two through five are approximately 16,500 sq ft and include about 32 guestrooms per floor.

Building Location

The building site is located in the Greenfield Corporate Center off of Olde Hempstead Road. This site can be found right off of route US-30. It can be seen behind low landscaping and a classic black iron fence with stone pillars from route US-30. Behind this fence is the Greenfield Corporate Center which is the home of many businesses. The site has small two lane roads that run on both sides and in front (the side facing route US-30) of the site for traffic through the corporate center. The site is surrounded by other business buildings on all sides except the front.



Overall Existing Conditions

Architectural:

The exterior façade is an Exterior Insulated Finish System (E.I.F.S.). The building has ½" and 1" E.I.F.S. which is a quartz putz DPR finish coming in the colors pearl, canvas and prairie clay creating a warm, welcoming façade. Windows includes Spandrel glass, prefinished aluminum curtain wall system and aluminum windows-glazed with aluminum flashing sill. The spandrel glass and curtain wall systems are on the first floor while the aluminum windows are the most common throughout the building. The roof consists of a fiberglass shingles-dimensional. This building gives an overall feeling that is very welcoming and pleasant. The architecture of this building follows the standard Courtyard by Marriott design. It follows a simple looking exterior with warm colors offering an inviting environment.

Construction:

Construction for the Courtyard by Marriott began in July of 2004 and is scheduled to be completed in June of 2005. The project delivery method for this building was a design build method. The total building cost is $9,550,000.

Lighting/Electrical:

The main distribution switchboard is 277/480 volts, 3phase and 4 wires. It provides 300 amps bus, braced for 65,000 amps system. Circuit breaker shall have shunt trip feature. Switchboard LDP is 120/208 volts, 3 phase and 4 wires. This switchboard provides 1600 amps bus, braced for 35,000 amps system. Most lighting in the hotel is fluorescent and incandescent. There is metal halide lighting used as the outside lighting.

Mechanical:

Each room of the hotel contains a thru wall heat pump providing a total of 245 cfm. There are 10 units of split system heat pumps ranging from 460cfm – 5150cfm. The roof equipment includes 2 roof ventilators, 6 fans ranging from 960cfm – 1940cfm and a kitchen ventilation unit – 2400cfm.

Structural:

The lateral system consists of load bearing masonry walls, masonry shear walls and moment frames. The exterior walls are load bearing masonry walls, while interior walls are masonry shear walls. Steel columns run from the foundation to a steel frame on the second floor which contains moment frames in the central location of the building. Above the second floor are masonry shear walls to the top of the building (top of fifth floor). The floor system of this building is an 8" hollow core plank system, and the foundation consists of spread footings and slab-on-grade.

Depth Study - Structural System

Existing Conditions

Courtyard by Marriott is an 83,821 square foot, 5 story hotel located within the Greenfield Corporate Center in Lancaster, PA. Construction for this building began in July 2004 and is scheduled to be complete in June 2005. This building is composed of a variety of masonry, steel and concrete. A combination of structural systems were used in the design of Courtyard by Marriott, such as reinforced masonry bearing walls, reinforced masonry shear walls, and moment frames. The exterior walls of the building are load bearing CMU walls, while the interior consists of masonry shear walls and steel moment frames. Design loads used for this building came from the IBC 2003 Codes.

Overall Building Systems:

- Masonry building
- Load bearing exterior walls
- Interior masonry shear walls second floor to fifth floor for lateral support
- Steel interior columns run from the foundation to second floor. These columns are placed throughout the interior of the first floor in order to create a large open space on the first floor.
- Second floor framing consists of large steel beams with moment frames running E/W and N/S in the central area of the building. The large beams in the moment frame will be used to transfer lateral loads to the exterior bearing walls on the first floor. Since there is a large lobby area on the first floor, this type of system has to be used to leave the area open without walls on the first floor.

- o Foundation: spread footings and slab on grade
- o Floor system: 8" hollow core pre-cast plank

Wall system:

- o All walls at the first floor are 8" ivany block with #5 bars at 8" c/c and #3 bars at 16", each side.
- o All other block used was 8" CMU
- o Wall reinforcement:
  - #5 bars at 32" from second floor to roof between concentrated reinforcement at wall ends or openings
  - #5 bars at 8" from foundation to second floor between concentrated reinforcement at wall ends or openings
- o Grout in walls:
  - Third floor to roof at 32"
  - Foundation to third, solid

Foundations system:

- o Various different sizes of spread footings used
- o On top of these spread footings are piers that will support the steel columns that span from the piers to the second floor framing
- o Reinforced 4" and 6" slab on grade used as well

Floor system:

- o 8" hollow core pre-cast plank with a minimum width of 4'
- o Typically spanning 28'
- o Typically spans in the E/W direction

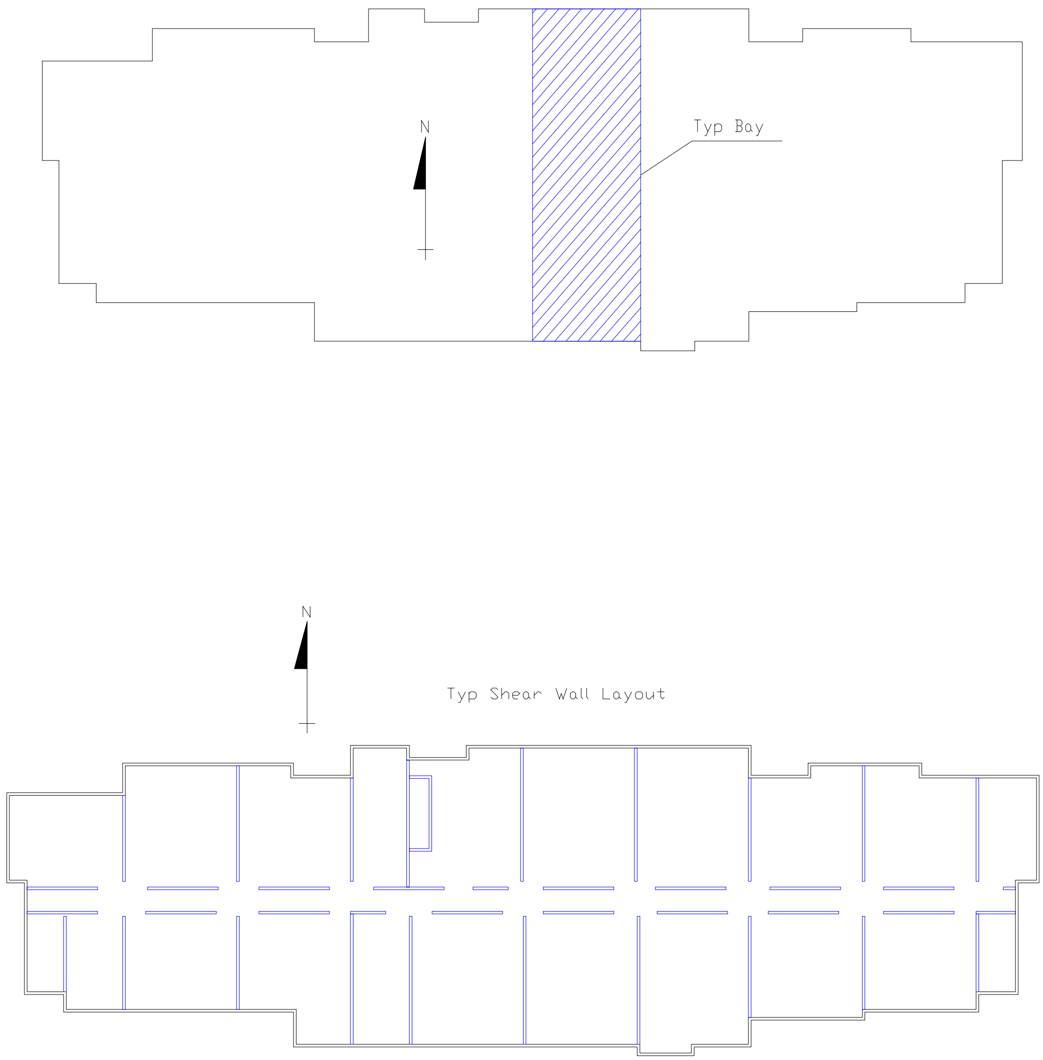


Figure 2. Existing shear wall layout in blue

The following figures show typical bays of the second floor (**Figure 3**) and a typical bay for floors three to five (**Figure 4**).

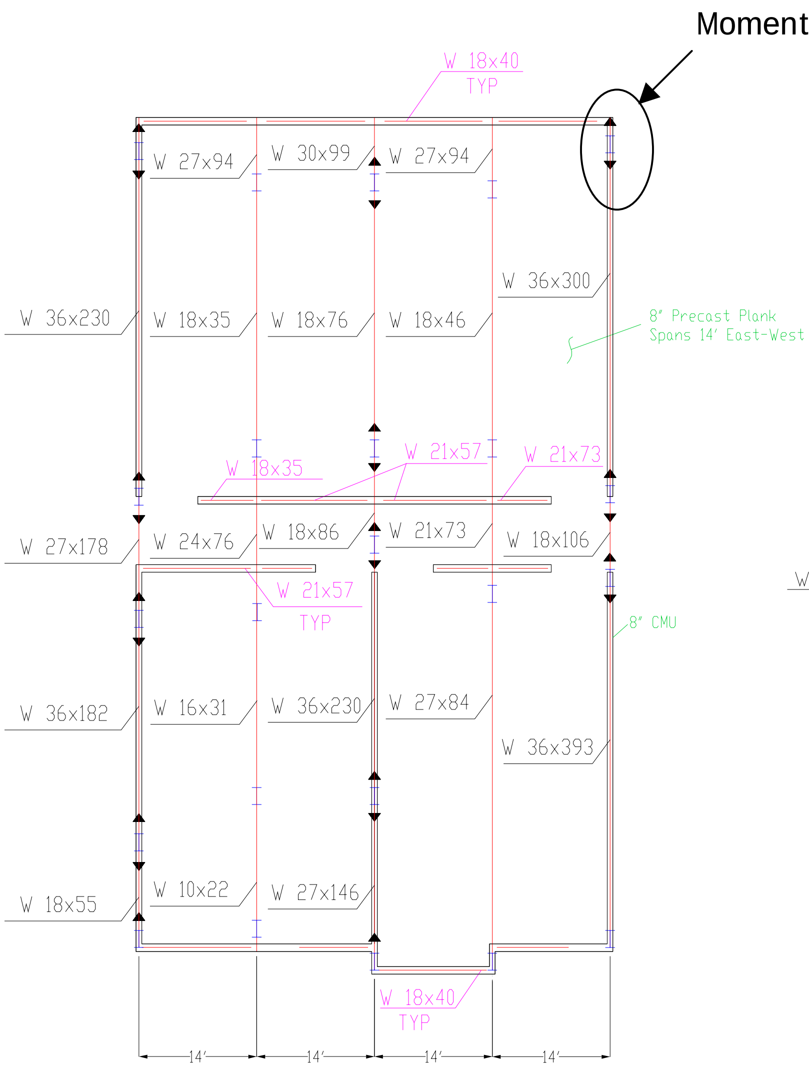


Figure 3. Typical Bays – 2nd Floor

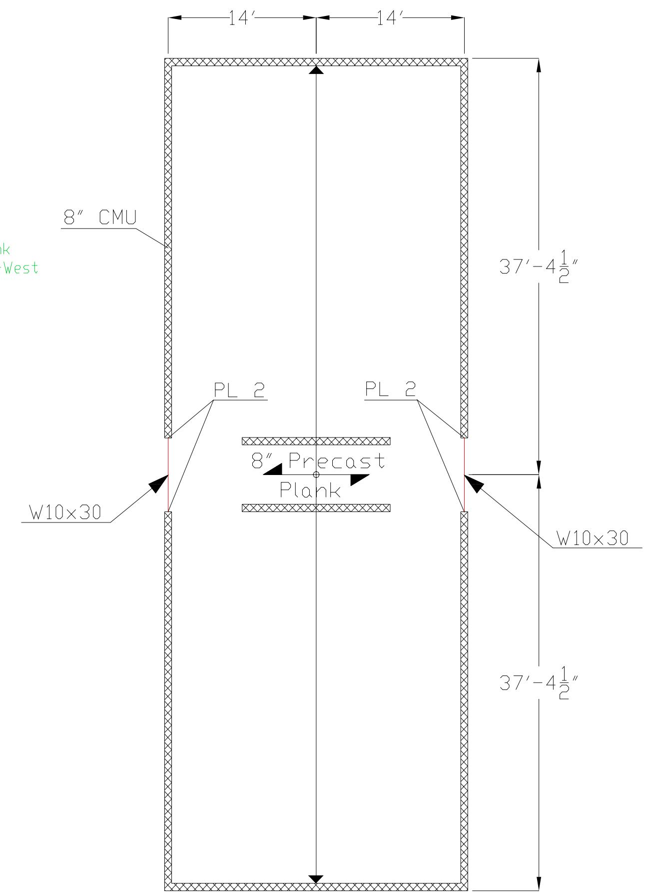


Figure 4. Typical Bay – 3rd to 5th Floors

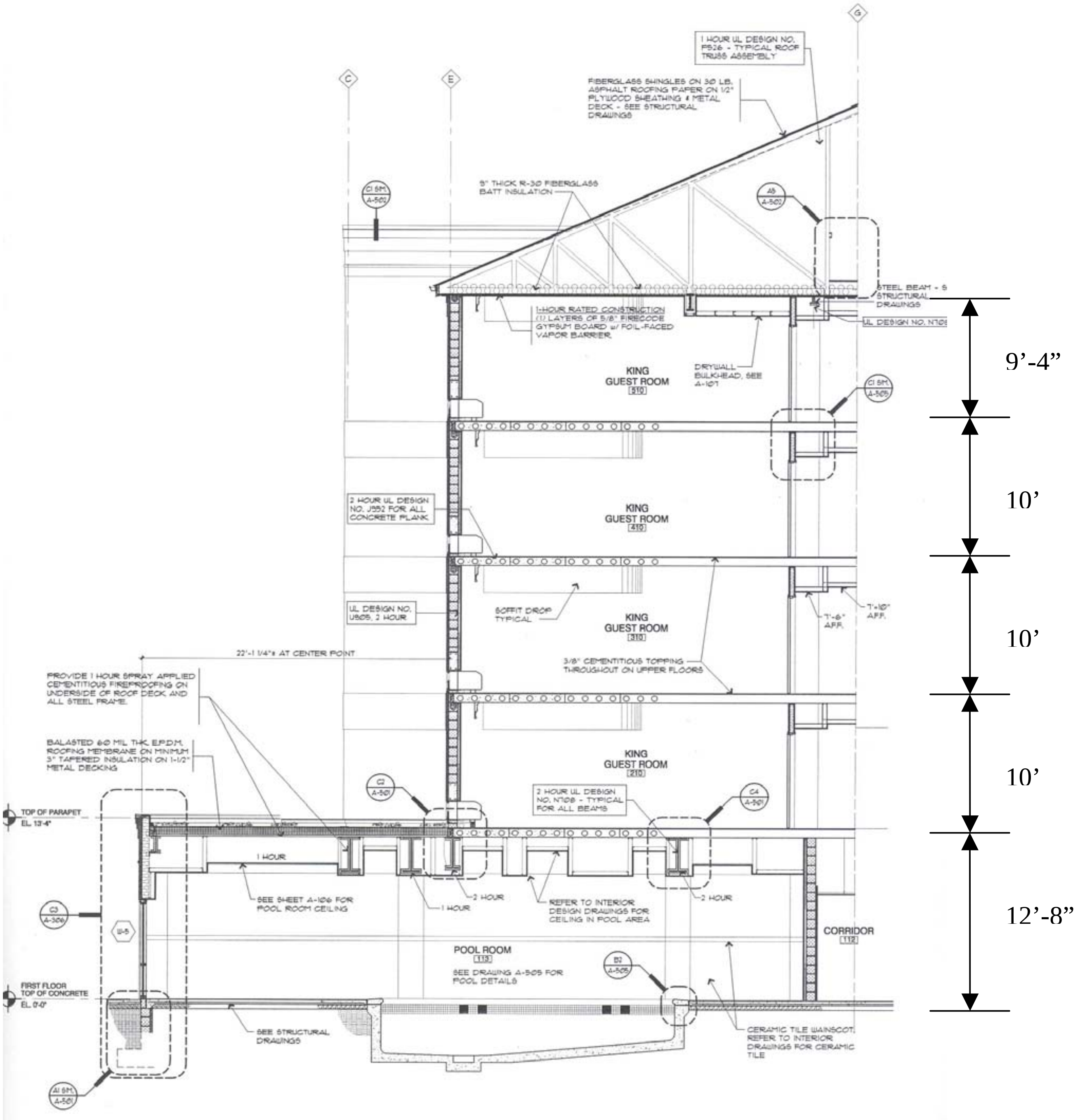


Figure 5. Building section courtesy of Greenfield Architects with existing floor – to - floor heights shown

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Lateral Loads:

Wind Loads

Base Wind Speed V = 90 mph
Exposure B
Importance Factor I = 1.0

Level	H_{trib} (ft)	E/W $Pressure_{Tot}$ (psf)	E/W Story Shear (k)	N/S $Pressure_{Tot}$ (psf)	N/S Story Shear (k)
Roof	9.6	13.25	9.5	16.32	39.8
5	9.7	12.77	9.2	15.84	38.9
4	10	12.17	9.1	15.24	38.7
3	10	10.98	8.2	14.05	35.7
2	11.3	9.89	8.4	12.96	37.2
1	0	0	0	0	0
		Σ=	**44.4**	Σ=	**190.3**

Table 1. Existing Wind Pressures

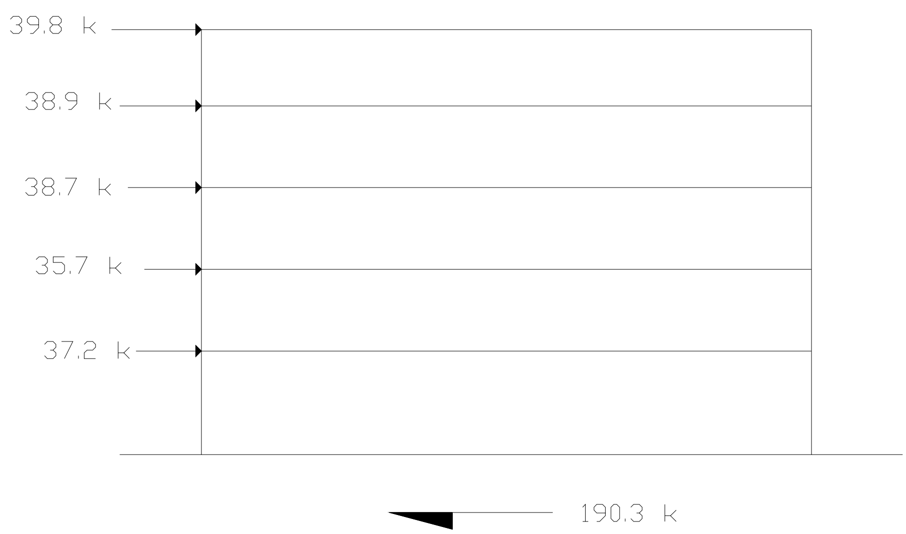

Figure 6. Existing Wind Story Shear – N/S Direction

Seismic Loads

R = 3.5
Seismic Design Category B
Seismic Use Category II
Seismic Factor = 1.0

Level	w_x (k)	h_x (ft)	$w_x h_x^k$	C_{vx}	F_x (k)
Roof	1459.2	52	78,876.8	0.257	155.6
5	2044.9	42.67	87,255.9	0.284	171.9
4	2044.9	32.67	66,806.9	0.218	132.0
3	2044.9	22.67	46,357.9	0.151	91.4
2	2173.13	12.67	27,533.6	0.090	54.5
1	0	0	0	0	0
			Σ = 306,831.1	**Σ = 1**	**Σ = 605.4**

Table 2. Existing Seismic Loads

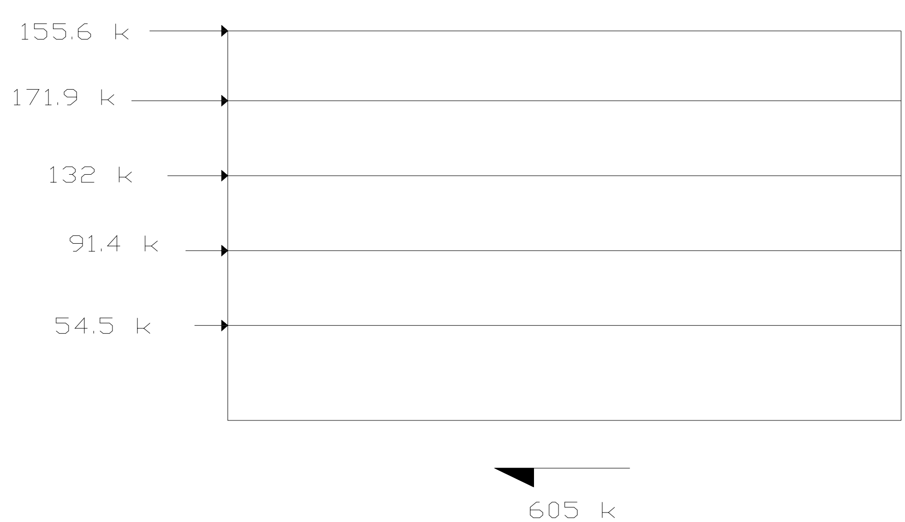

Figure 7. Existing Seismic Story Forces

Through the analysis of wind loads and seismic loads on the building, it was found that seismic (605.4 k) is the controlling loading in this case because it is significantly greater than the wind loading (190.3 k). The seismic is larger than the wind by slightly over a factor of three. Seismic also controls over the following load combinations that could be used for this building: 1.2D+1.6W+L

1.2D+1.0E+L+0.2S

Gravity Loads:

Dead Loads:		
8" Precast Hollow Core Plank	=	65 psf
8" CMU	=	32 psf
Structural Steel	=	5 psf
MEP, misc	=	10 psf
Insulation	=	3 psf
Shingles	=	4 psf
Metal Deck	=	2 psf
	Σ =	121 psf
Live Loads:		
Guest Rooms/Corridors	=	40 psf
Mechanical Rooms	=	150 psf
Stairs	=	100 psf
Wall Loads:		
8" CMU	=	32 psf
E.I.F.S.	=	15 psf
	Σ =	47 psf
Snow Load:	=	30 psf

Table 3. Existing Gravity Loads

Problem Statement

In the design of a building's structural systems there are many different design solutions that can work for any one building. Some solutions are better than others in that they are more cost effective, materials are more readily available, meet certain building standards, provide better performance, ensure a longer lifetime, and easier constructability. These are a few considerations that are taken into account when deciding on the structural system that would best meet the building's needs. For the Courtyard by Marriott in Lancaster, PA a viable solution was used for the structural system. This solution was chosen because it was found to be a good solution for this type of building. There were also requirements and restrictions on the design of the building that had to be fulfilled, such as floor-to-floor height restrictions. The structural system designed for this building met all of these qualifications. In the process of finding this solution, other solutions were analyzed as well. This proposal will research alternative systems that could work with this building. When exploring alternative systems cost, constructability, availability and overall performance will be taken into consideration.

Proposal

The main area of emphasis of this proposal is on the structural system. The lateral system is a major component of this system and carries the primary gravity loads. Since there are typically several feasible solutions for sufficient lateral systems in a building, this proposal will research several different types of lateral systems that could be used, staggered truss system, light gauged steel stud bearing walls and pre-cast concrete. One system in particular will be analyzed and compared to the existing masonry shear walls that work in conjunction with steel beams, columns and moment frames. In addition to the redesign of the structural system a green roof will be added to the building, which will implement a redesign of the roof system from sloped to flat and add an extra load on the building as well.

The staggered truss system will be researched into further detail than the other options. Reasons to look further into this system would be that it is cost effective, lightweight, fast erection, works well in hotels by providing open space for floor layouts and also works well with the existing hollow core floor plank system of the building. A redesign of the Courtyard by Marriott will be done using this system in conjunction with moment frames or braced frames.

Other alternate systems that may work would be light gauged steel and pre-cast concrete. The light gauged steel stud bearing wall system as well as the staggered truss system is cost effective, lightweight and provides open space for the hotel floor layout. Extra reinforcement such as shear walls may be need for support in this system. The pre-cast concrete alternative would be very similar to the existing masonry system in that pre-cast concrete shear walls would be used in place of masonry shear walls. The combination of steel or concrete columns to shear walls would still be needed. The only difference may be in cost. Neither of these systems will be used in the redesign but are viable alternatives that could be used in the design of this hotel.

Design Criteria

- Meet all code requirements for IBC 2003
- Maintain large open area on first floor
- Maintain low floor to floor height
- Provide feasible and cost effective design

Structural Analysis

Overview:

During the structural analysis of the Courtyard by Marriott in Lancaster, Pa, a redesign of the lateral and gravity system from masonry bearing and shear walls to a staggered truss system was performed. The building's foundation was kept as spread footings and the floor system remained as 8" pre-cast hollow core planks. The basic layout of the building was changed to accommodate the staggered truss systems efficiency. The layout of the building is now a 74' x 252' rectangular shape in order to yield the most efficient economical and structural results from the staggered truss system. The height of the building was also altered slightly. The 52' building was increased in height by 1' to obtain equal floor to floor heights and provide a higher first floor height. Another change made to the structure of Courtyard by Marriott in the structural analysis was the roof. It was transformed to a flat roof with a green roof system incorporated into it.

For the analysis of this building with the staggered truss system, hand calculations were done following the Steel Design Guide 14: Staggered Truss Framing Systems in order to design a typical truss member for this building (which can be found in the appendix) and a computer analysis was also done using ETABS.

Lateral Loads:

In using the IBC 2003 to determine the new lateral, it was found that the seismic load case still governs over the wind load case. The wind forces remained relatively the same as the existing structure, as well as the seismic forces. The seismic force decreased slightly overall. This is because of the extra load presented on the roof of the building from the 8" pre-cast hollow core plank and green roof addition. The hollow core planks add an extra 65 psf, while the green roof will add an extra 25 psf.

The existing system has a much lighter roof system that consists of metal deck with concrete and cold-form trusses. Another reason for the similar seismic loads in the two systems is that the seismic response factor used fot the staggered truss system was 3 and the seismic response factor used for the reinforced masonry shear walls was 3.5. A seismic response factor of 3 for the staggered truss systems is a more conservative value for the overall behavior of the system. Also, if this value is used in steel buildings, then there is no need to design special seismic connections. Since the building is located on the east coast, there is no need to consider special seismic connections. The following tables and figures show the results of the new wind and seismic load cases.

Wind Loads

V = 90 mph Exposure B I = 1.0 h = 53 ft					
Height (ft)	Windward Force (psf)	Leeward Force (psf)		Total Wind Force (psf)	
	N/S & E/W	N/S	E/W	N/S	E/W
0-15	6.83	-6.13	-3.06	12.96	9.89
20	7.43	-6.13	-3.06	13.56	10.49
25	7.92	-6.13	-3.06	14.05	10.98
30	8.39	-6.13	-3.06	14.52	11.45
40	9.11	-6.13	-3.06	15.24	12.17
50	9.71	-6.13	-3.06	15.84	12.77
60	10.19	-6.13	-3.06	16.32	13.25

Table 4. New Wind Loads

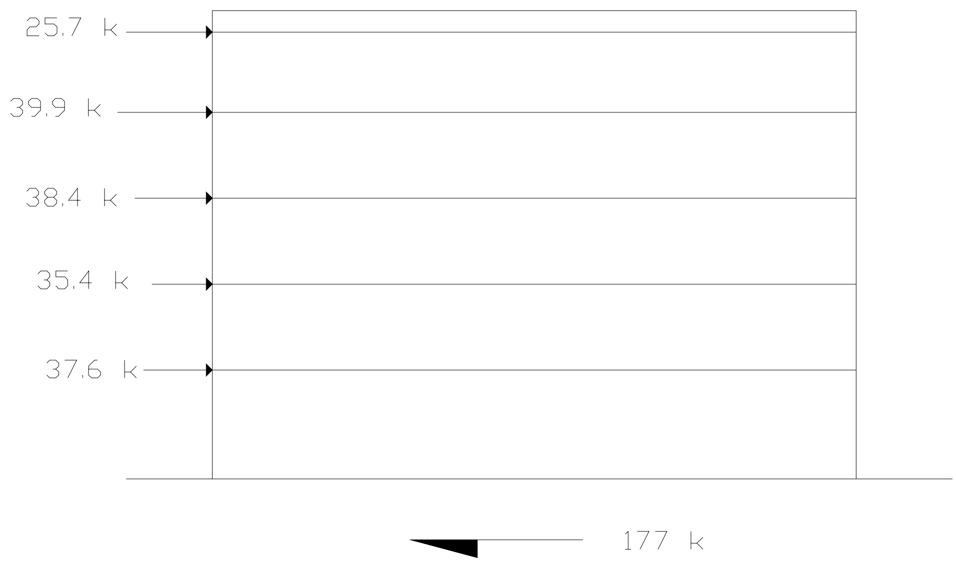

Figure 8. New Wind Story Shear - N/S Direction

Seismic Loads

R = 3 Seismic Design Category B Seismic Use Category II Seismic Factor = 1.0					
Level	w_x (k)	h_x (ft)	$w_x h_x^k$	C_{vx}	F_x
Roof	1870.2	53	99,121	0.344	216.7
5	1689.4	43	72,644	0.252	158.7
4	1689.4	33	55,750	0.193	121.6
3	1689.4	23	38,856	0.135	85
2	1689.4	13	21,962	0.076	47.9
1	0	0	0	0	0
			Σ =	1	629.9

Table 5. New Seismic Loads

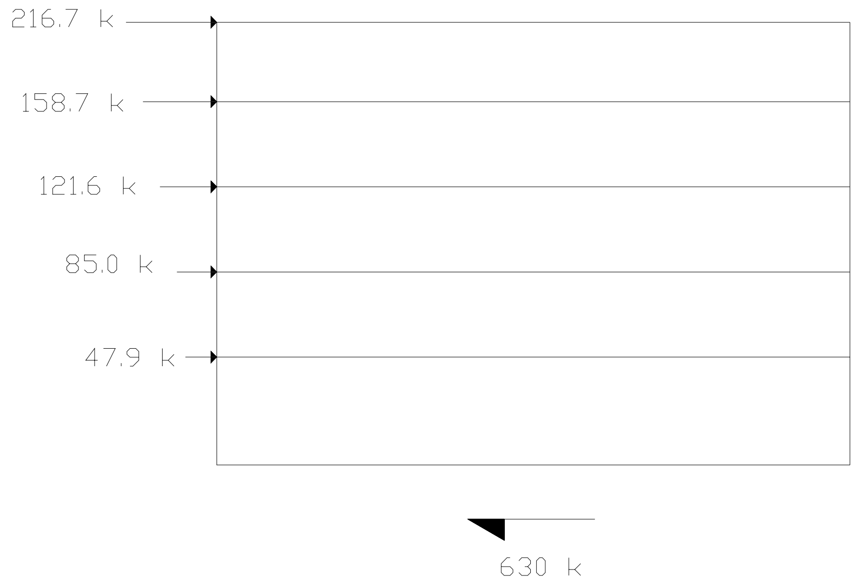

Figure 9. New Seismic Story Forces

Gravity Loads:

With the changes to the roofing system of the building, the gravity loads for the new system remain relatively similar to the existing system. The new total dead load equals 122 psf, while the dead load of the existing system is 121 psf. If the roof changes were not being considered and the roof from the existing system were to remain in the new system, the total dead load case would reduce to 89 psf. The following table provides all loads that contribute to the gravity loads of the building.

Dead Loads:		
8" Precast Hollow Core Plank	=	65 psf
Leveling Compound	=	5 psf
Structural Steel	=	5 psf
Partitions	=	12 psf
MEP, misc	=	10 psf
Green Roof	=	25 psf
	Σ =	122 psf
Live Loads:		
Guest Rooms/Corridors	=	40 psf
Mechanical Rooms	=	150 psf
Stairs	=	100 psf
Wall Loads:		
E.I.F.S.	=	15 psf
Studs	=	3 psf
Aluminum Sheathing	=	2 psf
Gypsum	=	3 psf
	Σ =	23 psf
Snow Load:	=	30 psf

Table 6. New Gravity Loads

Staggered Truss System:

The staggered truss frame designed for the Courtyard by Marriott in Lancaster, PA was designed with steel framing members and 8" pre-cast hollow core planks. Moment frames are used along the longitudinal direction of the building, while staggered trusses are used in the transverse direction. The stairwells and elevator openings will be framed with steel beams.

The hollow core planks will span 28' from truss to truss. The planks will span from the bottom of one truss to the top of the next truss. The trusses will span the total width of the building, 74' and they will be placed at every 28' down the entire 252' length of the building on alternating column lines. These trusses are one-story deep (10' for this design) and are located in the walls between rooms with a vierendeel panel at the corridors. They are only supported at their ends on the longitudinal rows of columns that are placed around the exterior of the building on the north and south sides. These columns are oriented with their strong axis parallel to the transverse direction of the building. There are no interior columns, only spandrel columns. A typical floor plan is shown in **Figure 10**.

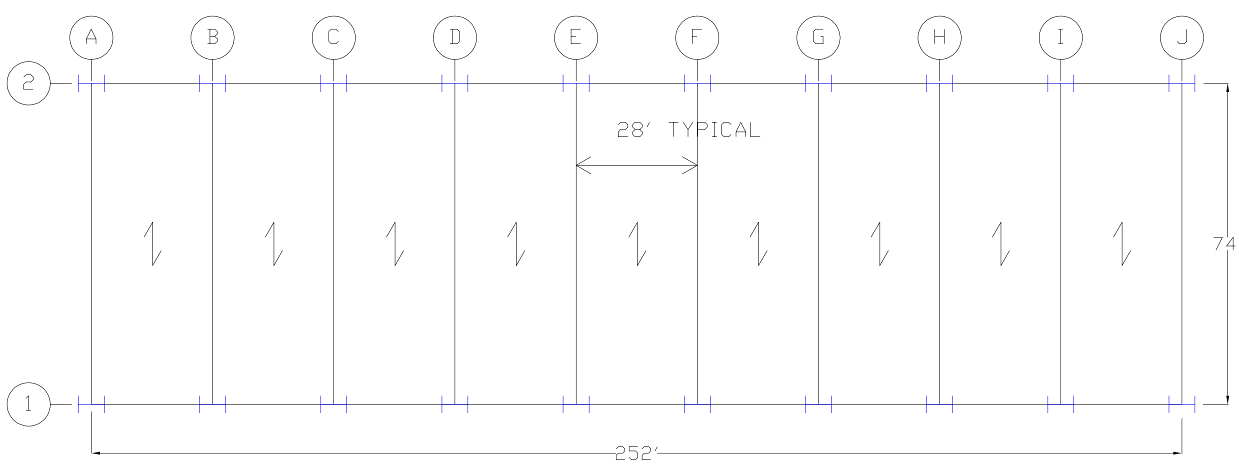

Figure 10. Staggered Truss Floor Layout

The two figures shown below show typical sections where the trusses are placed on odd stories and even stories. In **Figure 11** the second floor is being hung on the third floor truss by the hangers, while the roof is being posted up in **Figure 12**.

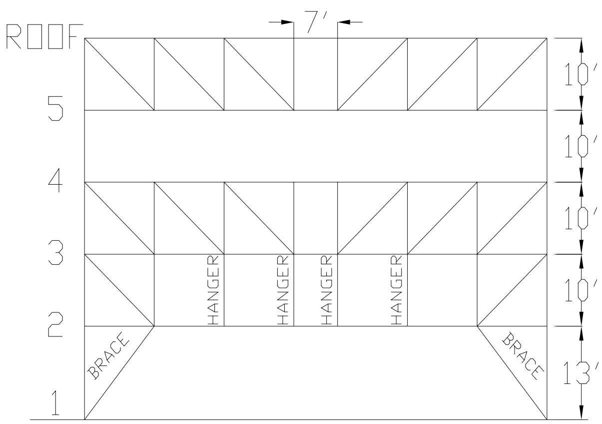

Figure 11. Odd Story Trusses

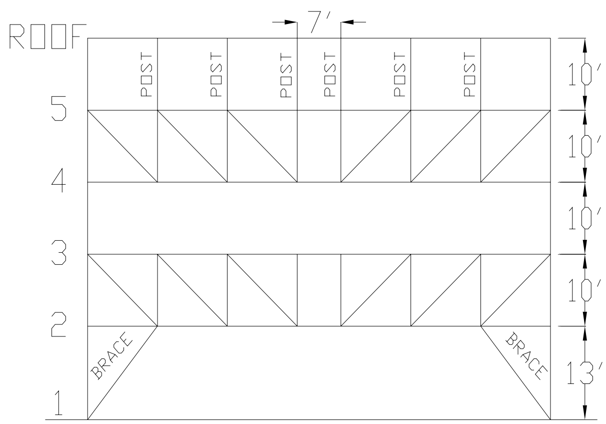

Figure 12. Even Story Trusses

The hollow core plank floor system is a very typical system used in hotel construction as well as staggered truss systems. The plank is connected to the truss chords with welded plates to provide temporary stability during erection. Then shear studs are welded to the chords, reinforcing bars are placed in the joints, and the grout is placed. When the grout cures, a permanent connection is achieved through welded studs. The hollow core planks will act as the diaphragm in the staggered truss system. In order for the plank to distribute lateral forces to the truss, it must act as a deep beam. Calculations including the diaphragm action with hollow core slabs can be found in the appendix.

Most common trusses are designed with W 10 chords and HSS web members connected with gusset plates. The chords must have a minimum width of 6" to ensure adequate plank bearing during construction. For the design of trusses in the Courtyard by

Marriott, W 10's and HSS 10 x 6's were design goals. The trusses are manufactured with camber to compensate for dead load. They are transported to site, stored and then erected. They can be erected in one piece generally up to 60' in length. In this case they will need to be transported in two pieces and spliced together in the field.

Design of Staggered Truss Members:

As suggested by Design Guide 14 the design of a typical truss member was done and then later analyzed using ETABS. The hand calculations typically ignore secondary effects such as moment transmission through joints, which may produce unconservative results. The typical truss used for analysis in the hand calculations was a truss located on the second floor on grid F (TF1). Truss dimensions can are shown below in **Figure 13**.

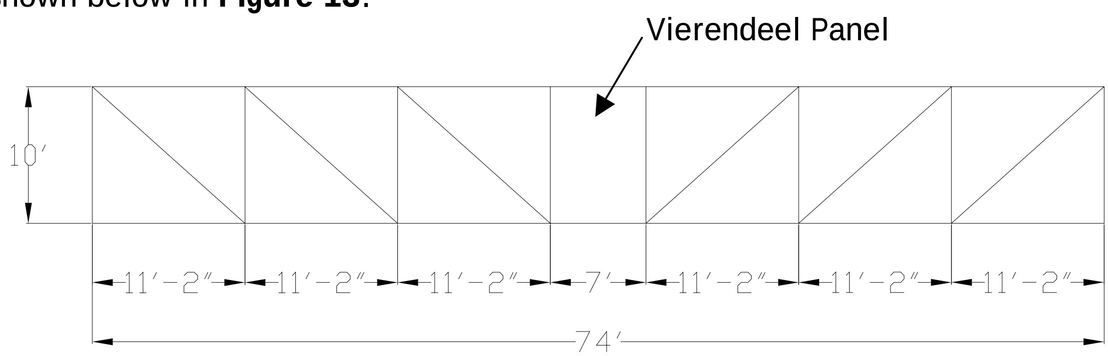

Figure 13. Truss Dimensions

The calculations begin with the design of the web members (vertical and horizontal members). The truss was first analyzed for gravity loads. A distributed load for the gravity load was found: w = (122 psf + 40 psf) x 28' = 4.53 k. This was then used with the tributary area to find the concentrated loads at the top and bottom joints of the truss members. The member forces were then found due to service gravity loads through the method of joints. Detailed calculations can be found in the **Appendix**. **Figure 14**, below, shows the resulting forces.

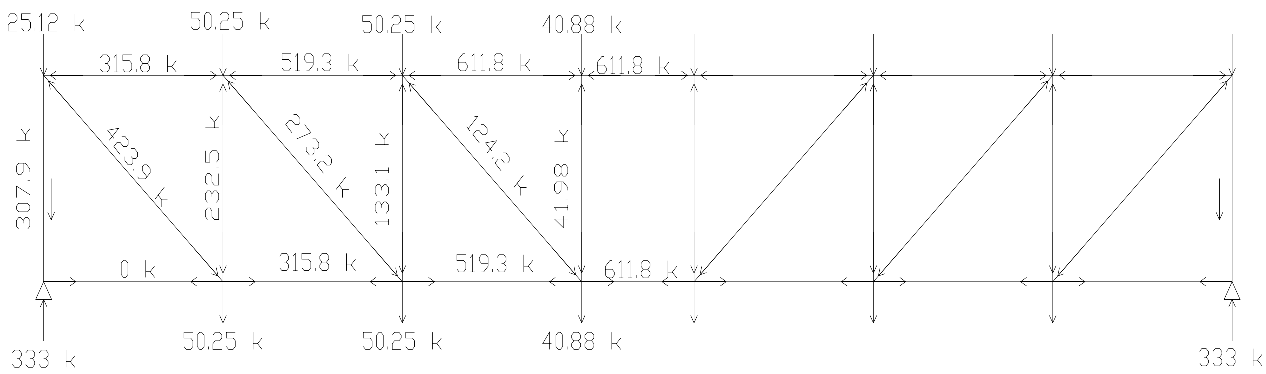

Figure 14. Member Forces Due to Gravity Loads

The truss was then analyzed for lateral loads. From previous calculations in the diaphragm design, the design shear strength was found (165 k) along grid line F. Because of anti-symmetry in the truss about its centerline for this load case, half the load was placed at each end of the top chord (horizontal reactions at supports = 82.5 k). Half the truss was analyzed as a free body and the shear force in the top and bottom chords of the vierendeel panel were assumed the same force in shear: V = ½(82.5 x 10)/37' = 11.15 k. The chord at the end moment at the upper vierendeel joint panel is equal to the shear x ½ panel length: M = (11.15 x 7)/2 = 39.02 ft-k, V = (39.02 + 0)/10 = 3.9k. The member forces were then found due to lateral loads through the method of joints. Detailed calculations can be found in the **Appendix**. **Figure 15**, below, shows the resulting forces.

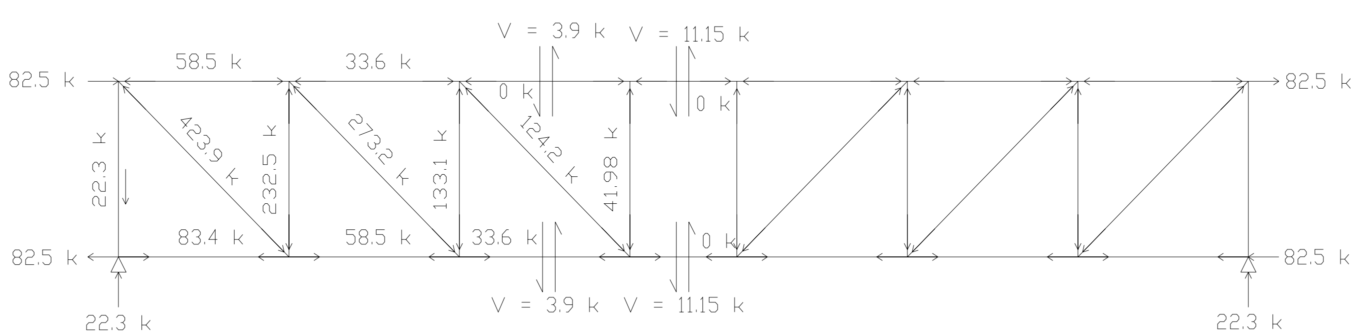

Figure 15. Member Forces Due to Lateral Loads

After the member forces for lateral loads have been calculated, they can be combined with gravity forces in order to find design forces. Once both these forces are calculated, forces for other trusses can then be computed using load coefficients. These load coefficients will be found for different load cases. The load coefficients are calculated in detail in the appendix.

In order to choose the correct size members load combinations must be defined to see which will control the design of the members. The loads used in the load combinations were:

Dead Load = 122 psf
Live Load = 40 psf
Reduced Live Load = 20 psf (50% LL Reduction)

The following load combinations were taken into consideration per code IBC 2003:

1) 1.4 D
2) 1.2D + 1.6L
3) 1.2D + 1.6W + 0.5L
4) 1.2D + 1.0E + 0.5L
5) 0.9D + (1.0E or 1.6W) – no need to calculate, will not govern

The following table shows results of the load combinations and selection of member sizes. The numbers in bold under the load combinations indicate the governing combination for that story. More detailed calculations can be found in the **Appendix**.

Diagonal Members - Typical Truss (TF1)									
	Wind		**Seismic**		**Load Combinations**				
Floor	**ϕ_h**	**$\phi_{ecc}*\phi_h*F_w$**	**ϕ_h**	**$\phi_{ecc}*\phi_h*F_w$**	**1**	**2**	**3**	**4**	**Member Sizes**
Roof	0.15	5.31	0.34	42.85	445	**466.3**	418.1	454.1	HSS 10 x 6 1/2
5	0.37	13.1	0.6	75.62	445	466.3	428.2	**486.8**	HSS 10 x 6 1/2
4	0.59	20.89	0.79	99.57	445	466.3	438.4	**510.8**	HSS 10 x 6 1/2
3	0.79	27.97	0.92	115.95	445	466.3	447.6	**527.2**	HSS 10 x 6 1/2
2	1	35.4	1	126.03	445	466.3	457.2	**537.2**	HSS 10 x 6 1/2
1		□							
F in d1 of Typical Truss TF1		33.4		118.9	423.9				

Table 7. Diagonal Member Design

For the chord design of the truss, two load combinations were considered, 1.2D + 1.6L and 1.2D + 1.0E + 0.5L, since both the cases governed. The steel design for the chords must comply with the AISC Equations H1-1a:

$$P_u/(\phi P_n) + (8/9) \times [M_{ux}/(\phi_b M_{nx})] < 1.0$$

ϕ = 0.9 for Tension

ϕ = 0.85 for Compression

ϕ_b = 0.9 for Bending

The member forces of the chords on the second story due to gravity and seismic forces are computed and then combined. These calculations can be found in the **Appendix**. The following two tables will show the design of the staggered truss chords for the two governing load cases.

Chord Members - Typical Truss (TF1)				
Load Case 1.2D + 1.6L				
Floor	**Mu**	**Pu**	**Section**	**Eq H1-1a**
Roof	50	673	W 10 x68	0.93
5	50	673	W 10 x68	0.93
4	50	673	W 10 x68	0.93
3	50	673	W 10 x68	0.93
2	50	673	W 10 x68	0.93
1				

Table 8. Chord Member Design - Load Case 1.2D + 1.6L

Chord Members - Typical Truss (TF1)						
Load Case 1.2D + 1.0E + 0.5L						
Floor	ϕ_h	**Mu, s**	**Mu**	**Pu**	**Section**	**Eq H1-1a**
Roof	0.34	14	29	593	W 10 x 54	0.98
5	0.6	25	51	593	W 10 x 60	0.95
4	0.79	32	67	593	W 10 x68	0.88
3	0.92	38	78	593	W 10 x68	0.91
2	1	41	85	593	W 10 x68	0.93
1						

Table 9. Chord Member Design - Load Case 1.2D + 1.0E + 0.5L

The floor loads are delivered to the columns through the truss – to – column connections. A typical one-story truss can carry the load from two floors, while a truss with hangers or posts, the load from three floors will be carried. The maximum live load reduction of a 50% (RLL = 20psf) is permitted since the columns support large tributary areas. The table below shows the design values of the columns calculated by hand. Refer to the **Appendix** for more detailed calculations.

Column Design - Typical Column TF1											
	Axial						**Moment**	**Load Combinations**			
	Floor		**Exterior Wall**	**Total**				**1.4D**		**1.2D + 1.6L**	
	DL1	**DL2 + RLL**		**DL1**	**DL2 + RLL**	**Ext. Wall**	**DL1**	**Pu**	**Mu**	**Pu**	**Mu**
Roof	145	294.2	6.44	145	294	6.44	33	203	46.2	377	39.6
5			6.44	145	294	12.9		203		385	
4	145	294.2	6.44	290	588	19.3	51	406	71.4	762	61.2
3			6.44	290	588	25.8		406		769	
2	145	294.2	6.44	435	882	32.2	75	609	105	1146	90
1			6.44	435	882	38.6		609		1154	

Table 10. Column Design – Roof, 5 -> W12 x 65; 4, 3 -> W12 x 96; 2, 1->W12 x 136

ETABS analysis:

In ETABS, a three dimensional model of Courtyard by Marriott with the new staggered truss system was created. Codes used in the computer analysis were IBC 2000 for seismic loading and ASCE 7-98 for wind loads. It was found through this computer analysis that the total building due to seismic loading was 1.41" and the drift due to wind was .0078". Both of these drifts are less then the l/400 building drift that is equal to 1.59". The member size output for the ETABS analysis differed slightly from the hand calculation output. The computer analysis may offer a more accurate solution. Tables with story drifts provided by ETABS as well as other ETABS results can be found in the **Appendix**.

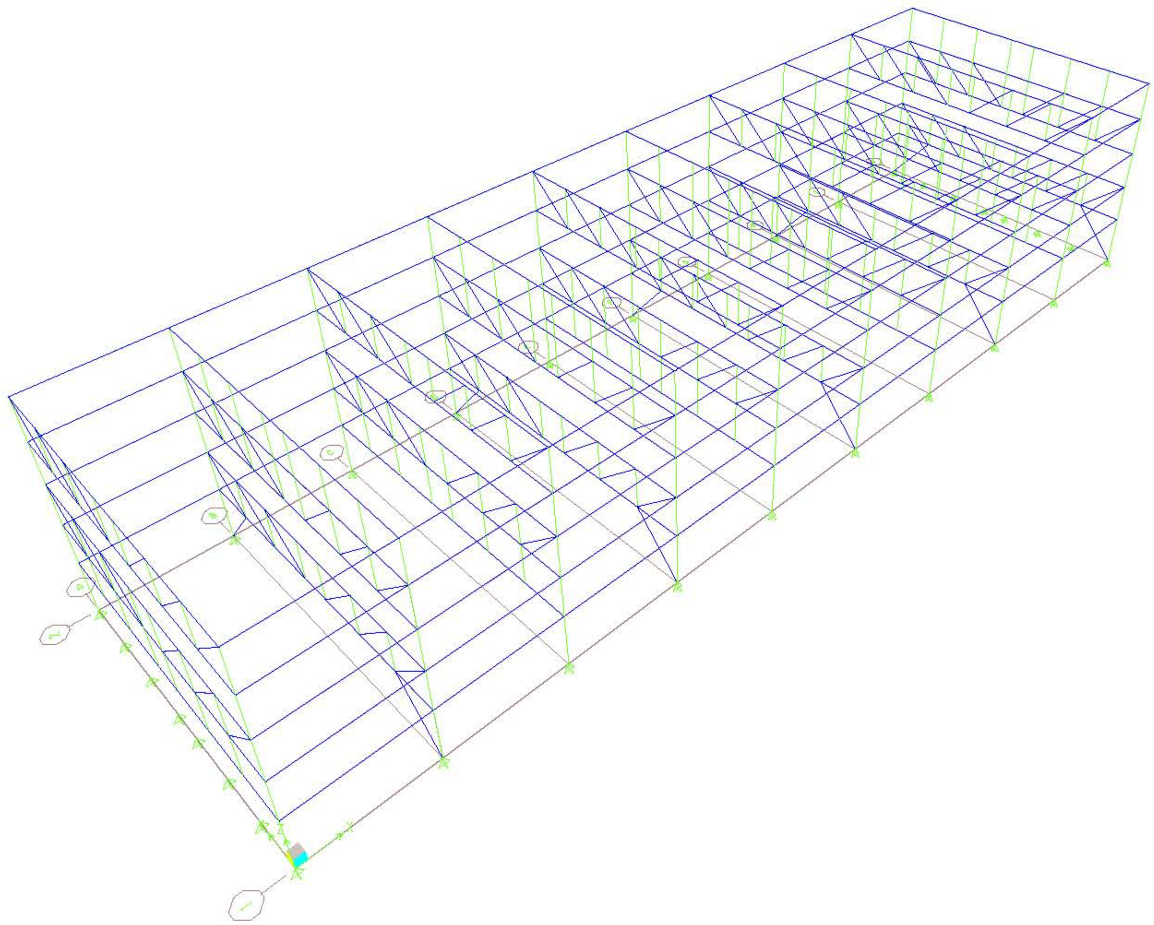

Figure 16. ETABS – 3D building view showing staggered truss alignment

In order to obtain member sizes for the trusses, beams/spandrels in columns general shapes and sizes had to be chosen at the beginning of the analysis. This would also ensure that the desired repetition in the staggered truss system would be satisfied. W 10 sections were chosen for the chords of the truss while HSS10 x 6 x 3/8, HSS10 x 6 x ¼ and HSS10 x 6 x ½ were chosen for the web members. Six-inch HSS members were chosen in order to keep the thickness of the wall to a reasonable size. For the columns, W 12 sections were selected. Though W 12's were specified, some W14 columns were needed in the design. Typical truss designs are shown in **Figure 17** and **Figure 18**, while a section of the front elevation can be found in **Figure19** with spandrel beam and column sizes.

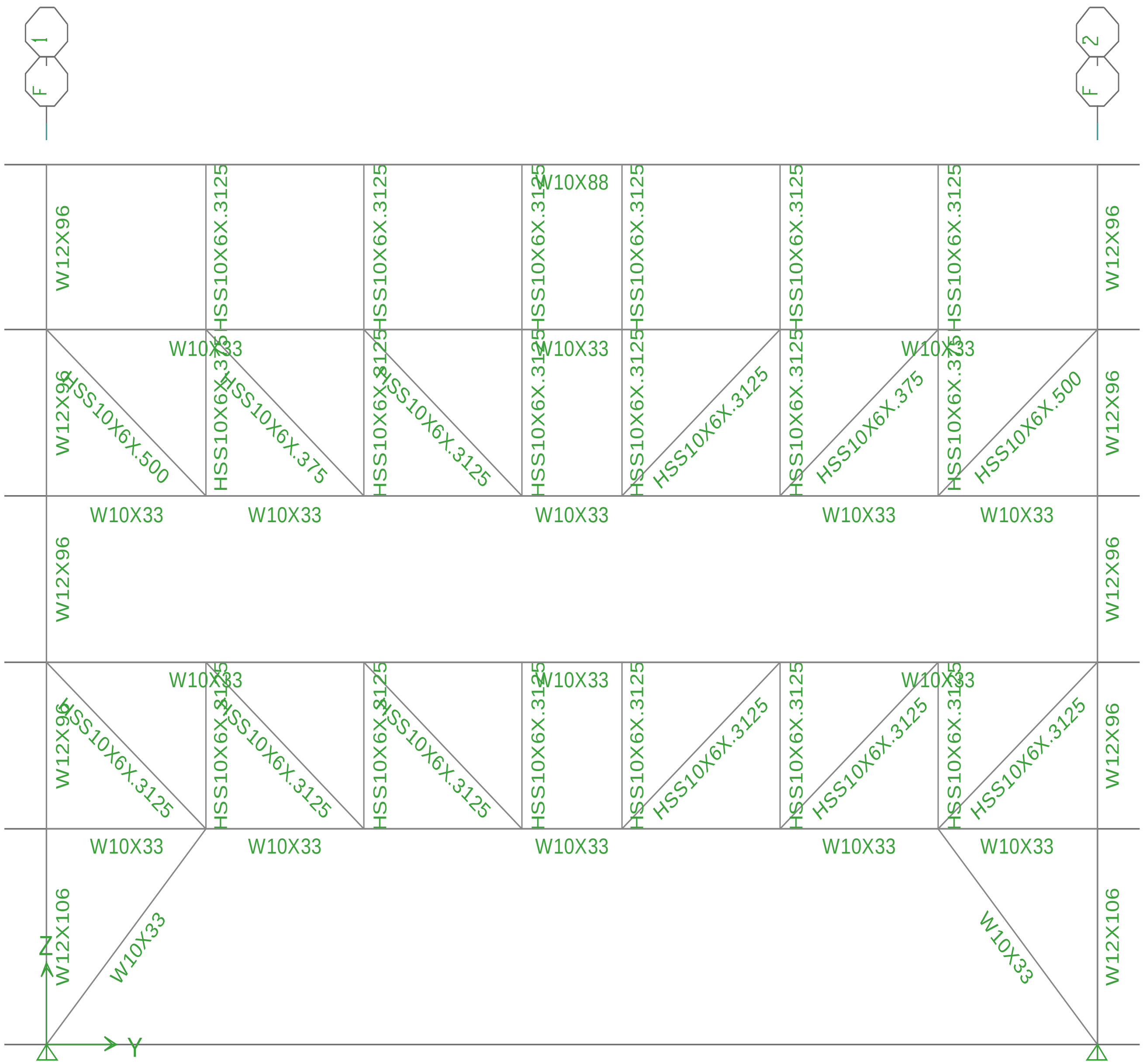

Figure 17. ETABS building section at grid line F – member sizes

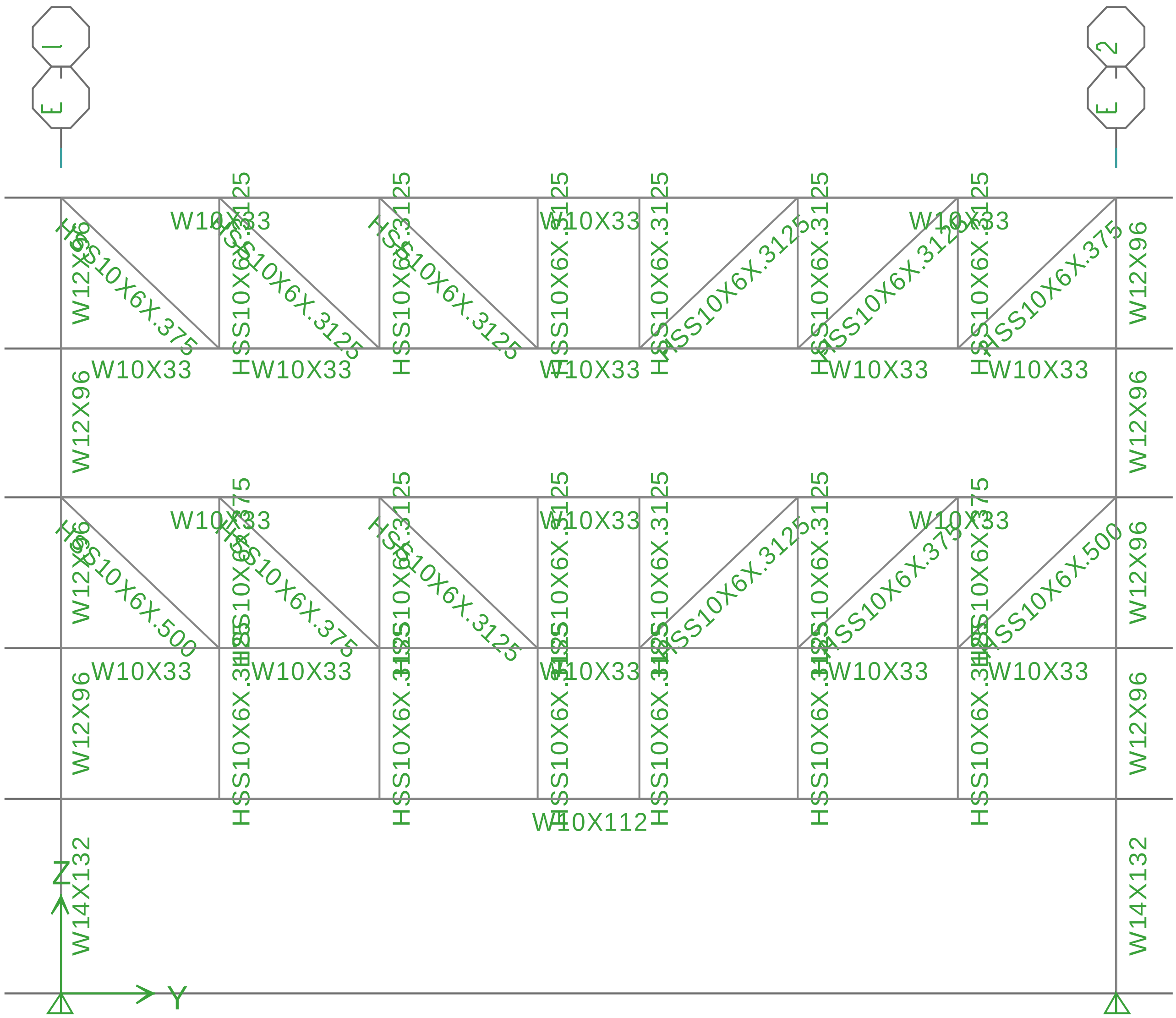

Figure 18. ETABS building section at grid line E – member sizes

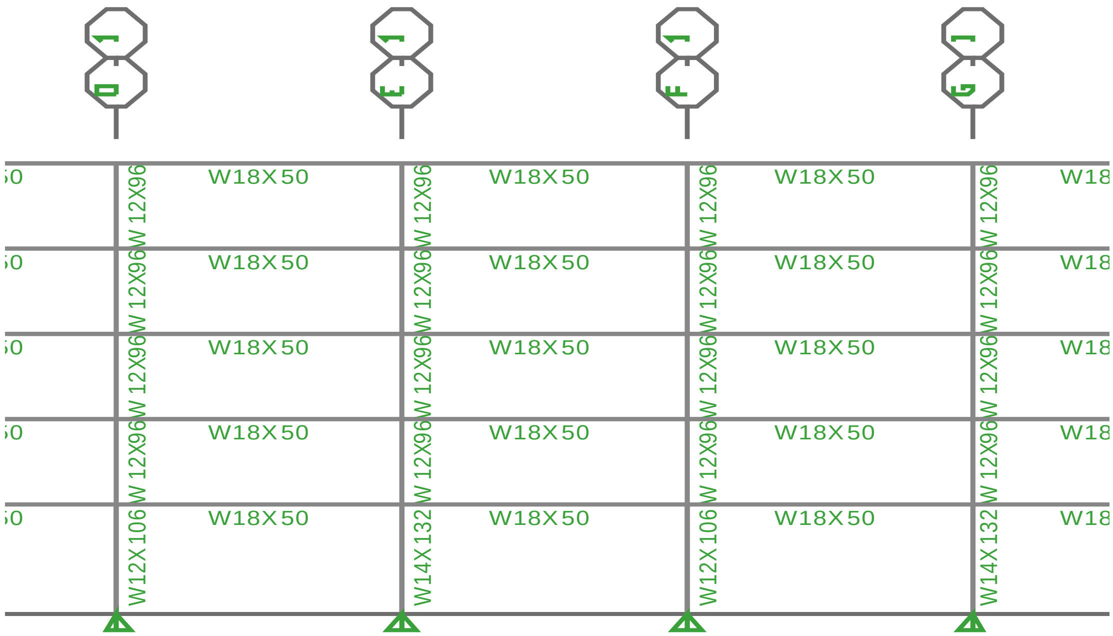

Figure 19. Section from ETABS front elevation at grid line 1

Staggered Truss vs. Masonry Shear Wall:

<u>Staggered Truss System</u>

Advantages:

- Provides large column free open space on first floor
- Provides an open floor layout for hotel rooms on upper floors
- Provide low floor-to-floor height
- Highly efficient for resistance to lateral loads caused by wind and earthquake
- Lighter system
- Quick and easy erection (especially during winter construction)

- Minimum fire protection required - since the trusses are typically placed in demising walls, (3) 1/2" layers of drywall can be placed on each side to achieve the proper fire rating. Also, spray on fireproofing can be used for which will be need at a minimum because of compact sections.

Disadvantages:

- Rectangular geometry of building does not meet the Courtyard brand of Marriott architecture and room layout
- Only efficient with repetition – if the trusses vary in length, height and member size, then the system will not yield any real benefits
- Spans larger than 60' must be erected in pieces and spliced together in the filed, possibly causing some time delays
- Misalignment of trusses during construction can cause problems with the plank alignment, offset interior walls and delay construction. Alignment tolerances are very low and construction has to be monitored closely.

<u>Existing System</u>

Advantages:

- Provides acceptable architectural and room layout for Courtyard brand of Marriott
- Provides a very stiff structural system which allows for minimum building and story drift
- Low floor-to-floor heights are maintained
- Typical type of structural system used in hotel construction
- Material easy to find locally and need not be specially shipped or fabricated

Disadvantages:

- Heavy system
- Columns used on first floor do not follow grid patterns
- Because using a combination of systems, more materials and trades are needed in order to construct the building resulting in higher cost
- Slow construction during the winter

Breadth Studies – LEED & Construction Management

LEED

In order to begin to incorporate some LEED certification criteria into the Marriott Hotel a green roof has been added to the building. This roof will induce extra dead load on the building creating a heavier structure, but it will provide several benefits to the building and progress the building towards LEED certification.

Requirements to earn points towards LEED certification by building a green roof structure under construction can be found in the LEED Green Building Rating System under Heat Island Effect: Roof, credit 7.2. According to this rating system one point will be awarded if the following guideline is met: "A "green" (vegetated) roof is installed for 50% of the roof area. Combinations of high albedo and vegetated roof can be used providing they cover 75% of the roof area." For Courtyard by Marriott, the entire roof will be considered to be covered with vegetation, giving it one point towards the LEED green building rating system.

Specifics about Courtyard by Marriott's green roof:

- Weight = 25 psf
- 80% - 85% roof coverage (coverage entire roof are except where mechanical equipment and roof hatches are located)
- Green Roof Materials:
  - Waterproofing Membrane
  - Drainage Layer
  - Growing Medium
  - Vegetation/Plants

A section of the green roof to be used can be found below in **Figure 20**. This section shows what materials are used and what layer they are located on. This roofing material will cover about 80% - 85% of the total roof area.

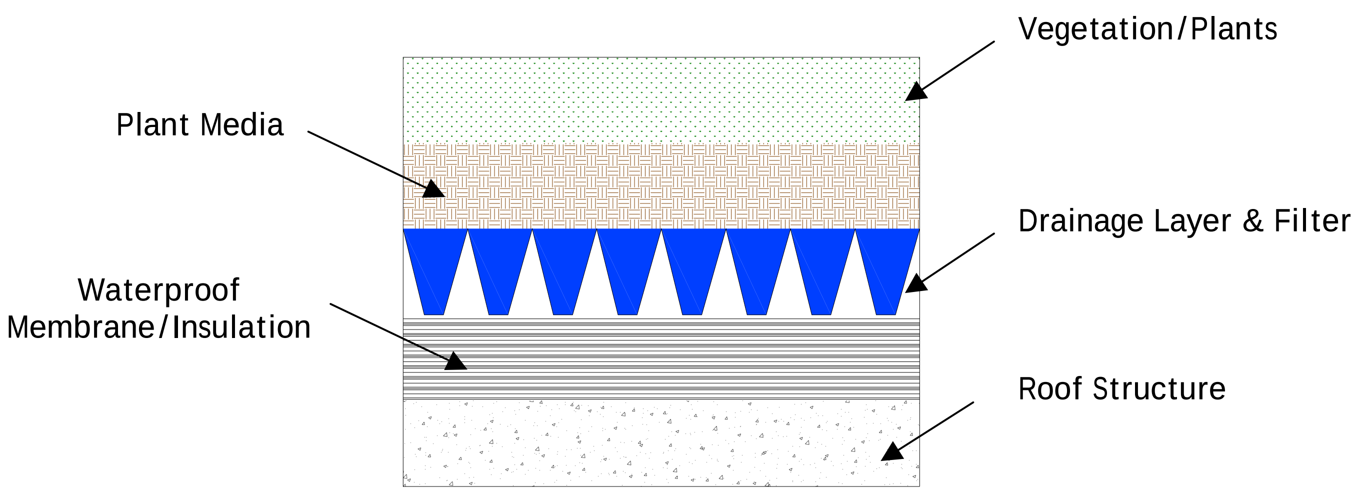

Figure 20. Green Roof Section

Costs:

Costs for extensive green roofs, such as this one, are estimated to average $15 to $20 per square foot. These costs include all aspects of the green roof material from the waterproofing membrane to the soil creation to planting of the vegetation on the roof. If 85% of the total roof area were covered with the green roof material (15, 850 sq ft) at $20 per square foot, then the cost of for the green roofing material would be $317,000. Installation costs, ongoing maintenance costs and lifecycle cost will also be applied to this base material cost. Replacement for this green roof may incur more costs than a typical roof. Often a vegetated roof will have a longer life than a conventional roof, saving on replacement costs as often. Green roofs can also reduce heating and cooling costs due to their increased insulation value.

Benefits:

There are many added benefits to using a green roofing system. Some of these advantages are:

- Reduces heat island effect
- Lowers noise levels through absorption
- Reduces immediate storm water run-off
- Reduces building heating and cooling cost due to increased insulation
- Lengthens roof life by two or three times by preventing Ultra Violet rays to get through to the roof surface
- Absorbs carbon dioxide and produces oxygen
- Aesthetically pleasing
- Provides green space in an area where there is minimum green space
- LEED credit

Feasibility:

The addition of a green roofing system to the new Courtyard by Marriott structure seems to be feasible in that it provides many benefits for the cost. This hotel is located in a corporate center surrounded by businesses and a major roadway. The addition of a green roof could be very beneficial to this hotel considering the benefits listed above and the location of Courtyard by Marriott in Lancaster.

Construction Management

The two different systems, masonry and steel, being compared will be dependent on different things. With steel there are three cost factors that must be considered for steel. These the factors are mill cost of the steel, the cost to fabricate the steel and the cost to erect the steel. While, with masonry the cost of the blocks and mortar, cost of the reinforcing and the cost of labor to lay the block are the factors that must be considered.

Costs of new system vs. existing system:

Costs to be compared for the existing structural system include masonry, masonry reinforcing steel, structural steel, cold-formed metal framing and metal deck. The costs from this system will be compared to the following costs of the new system W shapes for beams/spandrels/truss chords and columns, HSS shapes for truss web, truss fabrication, and additional hollow core plank. The cost of the new structural system is estimated to be $1,003,043, which is about $400,000 (30%) less than the cost of the existing structural system of $1,423,584. Existing systems cost come from the actual budget from the construction manager on the job. Costs from the Materials, Labor and equipment are all factored into both estimates. Tables with the breakdown of the costs can be found in the **Appendix**. There will be a decrease in the number of spread footings needed in the new system as well as the size of these footings with the lighter structure. The spread footings will decrease from 31 footings in the existing system to 20 footings in the new system. The cost difference of this was not calculated in the estimate. There is also the possibility of using two long strip footings on the north and the south sides of the building because that is where all columns are located. This may reduce cost in the foundation system as well.

Time line of both systems:

In staggered truss construction, the steel framing, including spandrel beams and pre-cast floors, are projected to be erected at the rate of one floor every five days. Once two floors are erected, window installation can start and stay right behind the steel and floor erection. There is no waiting for other trades, such as bricklayers or carpenters, to start or finish work with the staggered truss system except for foundations and grouting. Some time may be lost in erecting the trusses. The trusses will need to be spliced in the filed once erected because of their long span of 74'. Generally, if the truss is up to 60' long, it can be erected as one piece. Otherwise it will need to be erected in pieces and spliced.

In the erection of the existing system, time could have been lost in waiting for trades to begin and complete work since the structure composed of different materials and systems. The masons would be able to start laying their block for the piers and first floor exterior bearing walls as the first (ground) floor interior steel columns and 2nd floor steel framing is being erected. After the steel columns, 2nd floor framing and hollow core planks are erected, the masons can then continue their work and begin constructing the interior shear walls that start on the 2nd floor. Then the planks can be places once the second floor walls are completed and so on. Then finally the roof trusses can be placed when the masons have finished the fifth floor walls.

With these differences in the erection of the building, it seems that the staggered truss system may be a less timely process.

Conclusions/Final Recommendations

With the use of Design Guide 14 and ETABS an evaluation of the staggered truss system could be done. After evaluating and analyzing the possibility of replacing the current structural system of Courtyard by Marriott with a staggered truss system and the addition of the green roof system, several conclusions have been made.

The staggered truss system is a very beneficial system for this type of building in that it provides the large totally open first floor and column free floor layout in the upper floors. This open space creates freedom in the placement of rooms and wall partitions. This system also presents the low floor-to-floor height desired by Marriott with each floor spanning 10' high. In addition to the open floor plan and floor-to-floor height, the staggered truss framing system is highly efficient for resistance to lateral loading caused by wind and earthquake. This is especially good for Courtyard by Marriott because seismic loading is the governing load case. The staggered truss system provides an excellent solution to this type of building, but may not be the best for this building.

Since Marriott has certain criteria that must be met for each brand of hotel, the staggered truss system will not work to its full advantage with the requirements of the Courtyard brand. The shape of the building had to be changed from a non-symmetrical long and slender building to a rectangular geometry in order to efficiently incorporate the staggered truss system. This change in architecture of the Courtyard brand would be unacceptable to Marriott due to the fact that they have specific plans and room/floor layouts for this brand. Since a certain percentage of each room type must go into this brand of Marriott, the different sizes provided in the original design would not comply with that of a rectangular building. Also, due to the size of this building the staggered truss system may not be as cost efficient as it would be in a larger building with more trusses and repetition.

The implementation of the green roof system however may be something that Marriott would be willing to try on their Courtyard brand hotels. They can provide many benefits to the building as well as its surrounds.

In conclusion, the staggered truss system is a very efficient system that is especially good for hotel construction. It has been used in several Marriott's, but not the Courtyard brand. The redesign to a staggered truss system was beneficial to the building, but not feasible per Marriot standards and the building size. The current system of block and plank with a steel transfer system seems to be the most logical design decision for the Courtyard by Marriott in Lancaster, PA.

Tables & Figures

Tables

Figures

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Aaron Meier, best friend

My Family, Mom and Dad

Appendix

Appendix A

Seismic Loads:

SDS = 0.219g

SD1 = 0.084g

IE = 1.0

R = 3

Seismic Design Category, B

Seismic Use Group, II

$V = C_s*W$

$C_S = (S_{DS})/(R/I_E)$ < or = : $Cs = (S_{D1})/[(R/I_E)*T)$
> or = : $Cs = 0.044*S_{DS}*I_E$

Cs = (0.219/3) = 0.073

T = Ct*hn3/4
Ct = 0.02 (other building category)
T = (0.02)*(53')3/4 = 0.39 T< 0.5, k=1

< : Cs = (0.084)/[(30)*(0.39) = 0.071, OK

>: CS = (0.044)*(0.219)*(1) = 0.0096. OK
THEREFORE, Cs = 0.073 CONTROLS

w_2 = 1689.4 k

w_3 = 1689.4 k

w_4 = 1689.4 k

w_5 = 1689.4 k

w_{roof} = 1870.2 k

W_{Tot} = 8627.8 k

V = (0.073)*(8627.8 k) = 629.8 k

Appendix B

Wind Load Calculations:

Windward P = qz*G*Cp
Leeward P = qh*G*Cp

Exposure B, case 2

Kzt = 1.0 (no hill)

V = 90 MPH

I = 1.0

G = 0.85 (rigid frame)

Cp (windward) = 0.8

Cp (leeward): N/S, L/B = 74'/252 = 0.2945, Cp = -0.5
E/W, L/B = 252'/74' = 3.396, Cp = -0.25

z (ft)	Kz	qz (psf)
0-15	0.57	10.05
20	0.62	10.93
25	0.66	11.64
30	0.70	12.34
40	0.76	13.40
50	0.81	14.28
60	0.85	14.98

q_h = 14.42 psf

$P_{windward}$: **P_{0-15} = 6.83 psf**
P_{20} = 7.43 psf
P_{25} = 7.92 psf
P_{30} = 8.39 psf
P_{40} = 9.11 psf
P_{50} = 9.71 psf
P_{60} = 10.19 psf

$P_{leeward}$: **$P_{N/S}$ = -6.13 psf**
$P_{E/W}$ = -3.06 psf

Appendix C

ETABS Results:

Drift due to seismic loads:

D I S P L A C E M E N T S A N D D R I F T S AT P O I N T O B J E C T 19
Kip-in

STORY	DISP-X	DISP-Y	DRIFT-X	DRIFT-Y
STORY5	1.411217	-0.000438	0.001147	0.000017
STORY4	1.273626	-0.002446	0.001608	0.000003
STORY3	1.080685	-0.002842	0.001947	0.000002
STORY2	0.846996	-0.002592	0.002326	0.000026
STORY1	0.567907	0.000506	0.003640	0.000003

Drift due to wind loads:

D I S P L A C E M E N T S A N D D R I F T S AT P O I N T O B J E C T 9
Kip-in

STORY	DISP-X	DISP-Y	DRIFT-X	DRIFT-Y
STORY5	0.007817	-0.175785	0.000000	0.000017
STORY4	0.007853	-0.173724	0.000001	0.000028
STORY3	0.007931	-0.170346	0.000004	0.000037
STORY2	0.007454	-0.165871	0.000019	0.000357
STORY1	0.005219	-0.123016	0.000033	0.000789

Story Data

Story	Height	Elevation	SimilarTo
STORY5	120	636	None
STORY4	120	516	STORY5
STORY3	120	396	STORY5
STORY2	120	276	STORY5
STORY1	156	156	STORY5
BASE	0	0	None

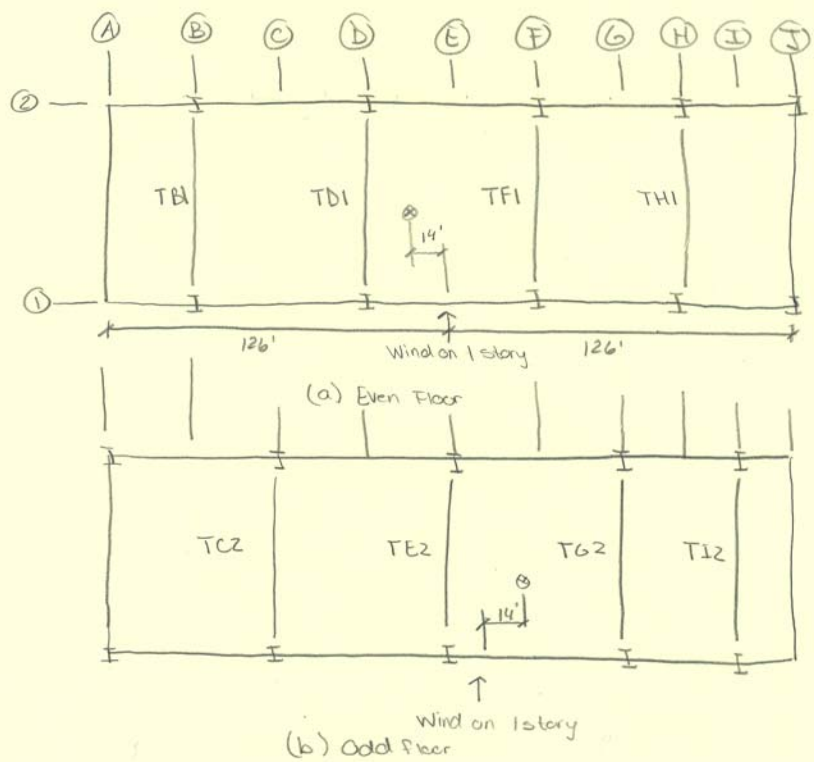
Material List By Story							
Story	ElementType	Material	TotalWeight	FloorArea	UnitWeight	NumPieces	NumStuds
STORY5	Column	STEEL	37.00282	2685312	1.377971E-05	80	
STORY5	Beam	STEEL	71.74342	2685312	2.671697E-05	38	0
STORY5	Brace	STEEL	14.04582	2685312	5.230609E-06	30	
STORY5	Floor	CONC	1864.681	2685312	0.0006944		
STORY4	Column	STEEL	28.52368	2685312	1.062211E-05	50	
STORY4	Beam	STEEL	49.43724	2685312	1.841024E-05	58	0
STORY4	Brace	STEEL	15.97615	2685312	5.949458E-06	30	
STORY4	Floor	CONC	1864.681	2685312	0.0006944		
STORY3	Column	STEEL	28.72744	2685312	1.069799E-05	50	
STORY3	Beam	STEEL	49.51769	2685312	1.84402E-05	58	0
STORY3	Brace	STEEL	15.97615	2685312	5.949458E-06	30	
STORY3	Floor	CONC	1864.681	2685312	0.0006944		
STORY2	Column	STEEL	37.48505	2685312	1.395929E-05	80	
STORY2	Beam	STEEL	48.30809	2685312	1.798975E-05	58	0
STORY2	Brace	STEEL	13.37794	2685312	4.981894E-06	30	
STORY2	Floor	CONC	1864.681	2685312	0.0006944		
STORY1	Column	STEEL	29.70277	2685312	1.10612E-05	20	
STORY1	Beam	STEEL	76.8477	2685312	2.861779E-05	48	0
STORY1	Brace	STEEL	5.651123	2685312	2.104457E-06	10	
STORY1	Floor	CONC	1864.681	2685312	0.0006944		
SUM	Column	STEEL	161.4418	1.342656E+07	1.202406E-05	280	
SUM	Beam	STEEL	295.8541	1.342656E+07	2.203499E-05	260	0
SUM	Brace	STEEL	65.02718	1.342656E+07	4.843175E-06	130	
SUM	Floor	CONC	9323.403	1.342656E+07	0.0006944		
TOTAL	All	All	9845.727	1.342656E+07	7.333022E-04	670	0

Material List By Section					
Section	**ElementType**	**NumPieces**	**TotalLength**	**TotalWeight**	**NumStuds**
W10X33	Beam	158	34816	93.34142	0
W10X33	Brace	10	2056.502	5.651123	
W10X39	Beam	2	704	2.291168	0
W10X77	Beam	2	1776	11.20288	0
W10X112	Beam	8	7104	65.14139	0
W12X96	Column	80	9744	77.76297	
W12X106	Column	10	1488	13.13844	
W12X120	Column	2	240	2.397576	
W14X132	Column	8	1248	13.70354	
W18X50	Beam	84	28224	112.8993	0
W18X60	Beam	2	672	3.220585	0
W21X68	Beam	2	672	3.659756	0
W24X76	Beam	2	672	4.097659	0
HSS10X6X.3125	Column	164	19680	48.7883	
HSS10X6X.3125	Brace	80	14390.22	35.6745	
HSS10X6X.375	Column	16	1920	5.650944	
HSS10X6X.375	Brace	24	4317.065	12.70599	
HSS10X6X.500	Brace	16	2878.044	10.99557	
PLANK1	Floor			9323.403	

Material List By Element Type				
ElementType	**Material**	**TotalWeight**	**NumPieces**	**NumStuds**
Column	STEEL	161.4418	280	
Beam	STEEL	295.8541	260	0
Brace	STEEL	65.02718	130	

Appendix D

Diaphragm action w/ hollow core planks 1



Even

Truss	x_i (ft)
TB1	28
TD1	84
TF1	140
TH1	+ 196
	Σ = 448

$x_e = \frac{448}{4} = 112'$

$e_e = 126 - 112 = 14'$

+5% ecc:

$e_e = 14 \pm (5\% \cdot 252)$

$= 26.6' ;\ 1.4'$

Odd

Truss	x_i (ft)
TC2	56
TE2	112
TG2	168
TI2	+ 224
	Σ = 560

$x_o = \frac{560}{4} = 140'$

$e_o = 126 - 140 = -14'$

± 5% ecc:

$e_o = -14 \pm (5\% \cdot 252)$

$= -1.4' ;\ -26.6$

(cont.) 2

$T = 630 \times 26.6 = 16,758$ ft-k
$T = 630 \times 1.4 = 882$ ft-k

$V_s = \frac{630}{4} = 157.5$ k

$V_i = V_s + V_{TORS}$ (2-1)

$V_s = \frac{V_w \times GA_i}{\Sigma GA_i}$ (2-2)

$V_{TORS} = \frac{V_w \times e \bar{x}_i GA_i}{GJ}$ (2-3)

GJ
Torsional Rigidity, Even

Truss	$\bar{x}_i$	$\bar{x}_i^2$
T1B	-84	7056
T1D	-28	784
T1F	28	784
T1H	84	7056
		Σ = 15,680

GJ
Torsional Rigidity, Odd

Truss	$\bar{x}_i$	$\bar{x}_i^2$
T2C	-84	7056
T2E	-28	784
T2G	28	784
T2I	84	7056
		Σ = 15,680

Vs

	Truss	$\bar{x}_i$	Vs	T = 16,758 (ft-k) V_{TORS}	V_i	T = 882 (ft-k) V_{TORS}	V_i	V (k) Design Shear V_i	ϕecc
	T1B	-84	157.5	-22.4	134.6	-1.2	156.3	156.3	1.00
e = 26.6	T1D	-28		-7.5	150	-0.4	157.1	157.1	1.01
1.4	T1F	28		+7.5	165	0.4	157.9	165	1.06
	T1H	84		+22.4	179.4	1.2	158.7	179.4	1.15
	T2C	-84		+22.4	179.4	1.2	158.7	179.4	1.15
e = -1.4	T2E	-28		+7.5	165	0.4	157.9	165	1.06
-26.6	T2G	28		-7.5	150	-0.4	157.1	157.1	1.01
	T2I	84		-22.4	134.6	-1.2	156.3	156.3	1.00

Vw:

	Wind	ϕ_h	Seismic	ϕ_h
Roof	25.7	0.15	216.7	0.34
5	65.6	0.37	375.4	0.60
4	104	0.59	497	0.79
3	139.4	0.79	582	0.92
2	177	1.0	629.9	1.0
1	0	0	0	0

(cont.) 3

$V_u = 1.7 \times \phi_n \times V \times 0.75$
$= 1.7 \times 1.0 \times 156.3 \times 0.75 = 199.3^k$

$\phi V_c = \phi \times 2\sqrt{f'_c}\, bd$
$= 0.85 \times 2\sqrt{4000}\,(6)(0.8 \times 74 \times 12) = 458^k$

$\phi V_s = \phi A_{VF} f_y \mu$

$\mu = 1.4$

#5 bar $A_s = 0.31 in^2$

$A_{VF} = 0.31 \times 9 = 2.79 in^2$

$\phi V_s = 0.85 \times 1.4 \times 60 \times 2.79 = 199.2$

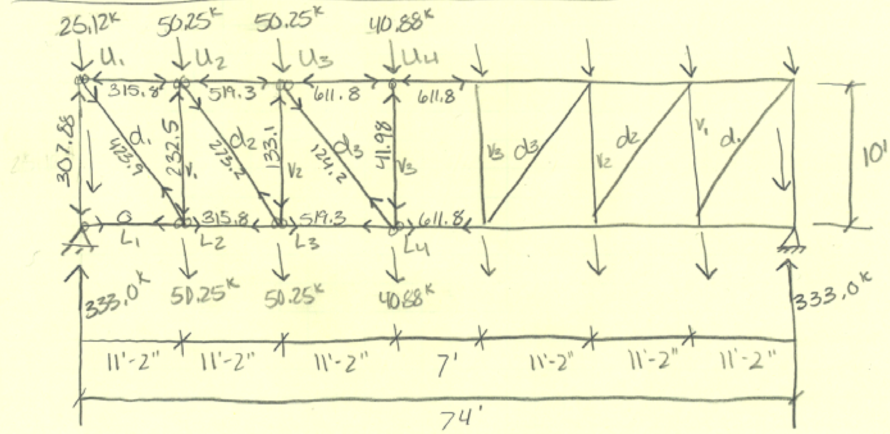
$\phi V_n = 458 + 199.2 = 657.2 > 199.3 \therefore OK$

$\#planks = \frac{74'}{8'} = 9.25 \approx 10$ planks

$10 - 1 = 9$ jts

Design of Truss Member 4

Truss TF1 - 2nd floor line F (74' span)



d_1: $TA = (7/2 + 11.167 \times 2 + 11.167/2)\, 28 \times 2 = 1759.38\,ft^2$

d_2: $TA = (7/2 + 11.167 + 11.167/2)\, 28 \times 2 = 1134.03\,ft^2$

d_3: $TA = (7/2 + 11.167/2)\, 28 \times 2 = 508.67\,ft^2$

TF1: $TA = 74 \times 28 \times 2 = 4144\,ft^2$

$W_{DL} = 4144 \times 97 = 402^k$

member d1:

Axial Force $T = \frac{165^k \times 97}{(97 + 40)} = 117^k$

Vertical component of $T = \frac{117}{\sqrt{2}} = 82.7$

$TA = \frac{82.7}{402} \times 4144 = 852.5\,ft^2$

$w = (122 + 40) \times 28 = 4.53^k$

$P_1 = 4.53^k \times 11.167/2 = 25.12^k$

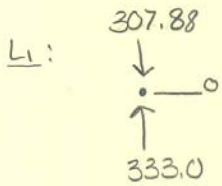
$P_2 = 4.53^k \times 11.167 = 50.25^k$

$P_3 = 4.53^k \times (11.167 + 7)/2 = 40.88^k$

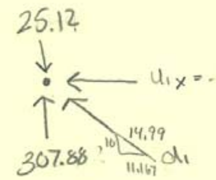
$R = (25.12 + (50.25)(2) + 40.88) \times 2 = 333.0^k$

(cont.) Gravity Loads 5

L1:



U1:



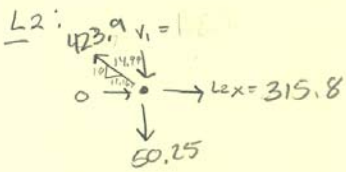
$\Sigma F_y = -25.12 + 307.88 + d_1(10/14.99) = 0$

$d_1 = -423.9$

$\Sigma F_x = 0 = -u_{1x} - (-423.9)(11.167/14.99)$

$u_{1x} = 315.8^k$

L2:



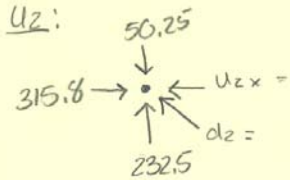
$\Sigma F_y = -v_1 - 50.25 + 423.9(10/14.99) = 0$

$v_1 = 232.5$

$\Sigma F_x = 0 = L_{2x} - 423.9(11.167/14.99)$

$L_{2x} = 315.8$

U2:



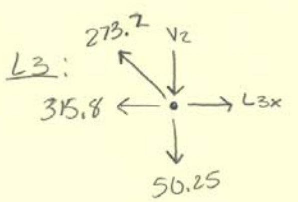
$\Sigma F_y = -50.25 + 232.5 + d_2(10/14.99) = 0$

$d_2 = -273.2$

$\Sigma F_x = 315.8 - u_{2x} - (-273.2(11.167/14.99)) = 0$

$u_{2x} = 519.3$

L3:



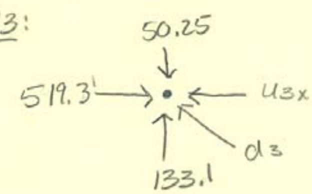
$\Sigma F_y = -v_2 - 50.25 + 273.2(10/14.99) = 0$

$v_2 = 133.1$

$\Sigma F_x = -315.8 + L_{3x} - 273.2(11.167/14.99) = 0$

$L_{3x} = 519.3$

U3:



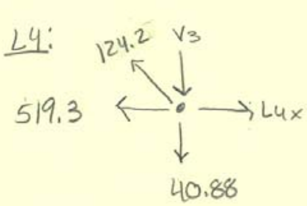
$\Sigma F_y = -50.25 + 133.1 + d_3(10/14.99) = 0$

$d_3 = -124.2$

$\Sigma F_x = 519.3 - u_{3x} - (-124.2(11.167/14.99)) = 0$

$u_{3x} = 611.8$

L4:



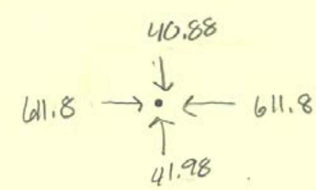
$\Sigma F_y = -40.88 - v_3 + 124.2(10/14.99) = 0$

$v_3 = 41.98$

$\Sigma F_x = -519.3 + L_{4x} - 124.2(11.167/14.99) = 0$

$L_{4x} = 611.8$

U4:



(cont.) Lateral Loads 6

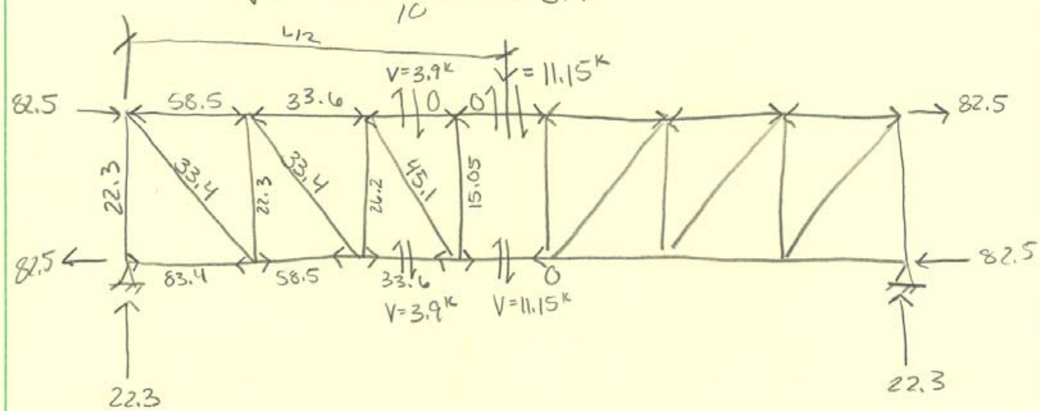
$R = ((0.5)(165) \times 2 \times 10)/74 = 22.3^k \Rightarrow$ Vertical Reaction @ each support

$V = \frac{\frac{1}{2}(82.5 \times 10)}{37} = 11.15^k$

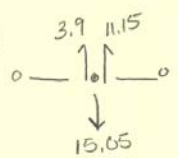
Chord end moment @ jt U4:

$M = \frac{11.15 \times 7}{2} = 39.02$ ft-k

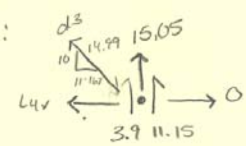
$V = \frac{(39.02 + 0)}{10} = 3.9^k$



U4:



L4:



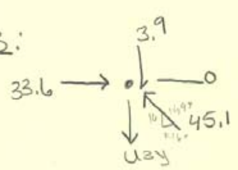
$\Sigma F_y = 3.9 + 11.15 + 15.05 + d3(10/14.99) = 0$

$d3 = -45.1$

$\Sigma F_x = -L4x - (-45.1(11.167/14.99)) = 0$

$L4x = 33.6$

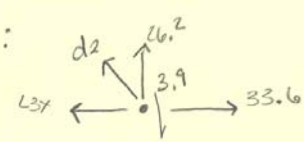
U3:



$\Sigma F_y = -3.9 - U3y + 45.1(10/14.99) = 0$

$U3y = 26.2$

L3:



$\Sigma F_y = -3.9 + 26.2 + d2(10/14.99) = 0$

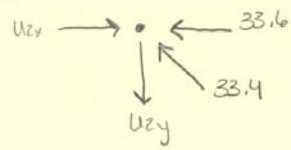
$d2 = -33.4$

$\Sigma F_y = -L3x + 33.6 - (-33.4(11.167/14.99)) = 0$

$L3x = 58.5$

(cont.) Lateral Loads 7

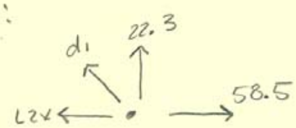
U2:



$\Sigma F_y = -U_{2y} + 33.4(10/14.99) = 0$
$U_{2y} = 22.3$

$\Sigma F_x = -33.6 + U_{2x} - 33.4(11.167/14.99) = 0$
$U_{2x} = 58.5$

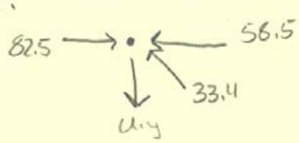
L2:



$\Sigma F_y = 22.3 + d_1(10/14.99) = 0$
$d_1 = -33.4$

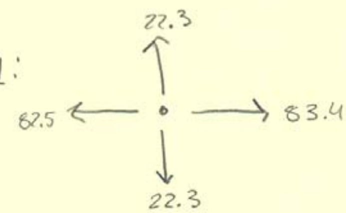
$\Sigma F_x = 58.5 - L_{2x} - (-33.4(11.167/14.99)) = 0$
$L_{2x} = 83.4$

U1:



$\Sigma F_y = -U_{1y} + 33.4(10/14.99) = 0$
$U_{1y} = 22.3$

L1:



Load Coefficients 8

Load combinations:

$1.4D$

$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$

$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (f_1 L \text{ or } 0.8W)$

$1.2D + 1.6W + f_1 L + 0.5(L_r \text{ or } S \text{ or } R)$

$1.2D + 1.0E + f_1 L + f_2 S$

$0.9D + (1.0E \text{ or } 1.6W)$

DL = 122 psf, LL = 40 psf, RLL = 20psf (50% LL Reduction)

$\phi_w = 1.0$
$\phi_{ecc} = 1.06$
ϕ_h wind = 1.0, ϕ_h seismic = 1.0

① $1.4D = \frac{1.4(122)}{(122+40)} = 1.05 = \phi_{L1}$

② $1.2D + 1.6L_r = \frac{(1.2)(122) + 1.6(20)}{(122+40)} = 1.10 = \phi_{L2}$

③ $1.2D + 0.5L_r = \frac{(1.2)(122) + 0.5(20)}{(122+40)} = 0.97 = \phi_{L3}$

Load comb. 1: $1.4D = \phi_{L1} F_G = 1.05(423.9) = 445.1$

Load comb. 2: $1.2D + 1.6L_r = \phi_{L2} F_G = 1.10(423.9) = 466.3$

Load comb. 3: $1.2D + 1.6W + 0.5L_r = \phi_{L3} F_G + 1.3\phi_{ecc}\phi_h F_W$
$= (0.97)(423.9) + 1.3(1.06)(1)(33.4)$
$= 457.2$

Load comb 4: $1.2D + 1.0E + 0.5L_r = \phi_{L3} F_G + \phi_{ecc}\phi_h F_E$
$= (0.97)(423.9) + (1.06)(1)\left(\frac{33.4 \times 630}{177}\right)$
$= 537.2$

Load comb 5: Will not controll

Diagonal Members - TF1 9

	Wind		Seismic		Load comb			
Floor	ϕ_h	$\phi_{ecc}\,\phi_h\,F_w$	ϕ_h	$\phi_{ecc}\,\phi_h\,F_E$	1	2	3	4
Roof	0.15	5.31	0.34	42.85	445	466.3	418.1	454.95
5	0.37	13.10	0.60	75.62	445	466.3	428.2	486.82
4	0.59	20.89	0.79	99.57	445	466.3	438.4	510.77
3	0.79	27.97	0.92	115.95	445	466.3	447.6	527.15
2	1.00	35.40	1.00	126.03	445	466.3	457.2	537.2
ground								
F in diag typ truss TF		33.4		118.9	423.9			

$\phi_{ecc} = 1.06$

$F_W = 33.4$

$F_E = \frac{33.4 \times 630}{177} = 118.9$

$F_G = 423.9$

466 — 537

$P_n = F_y A_g$

$A = \frac{P_n}{F_y} = \frac{537.2^k}{46\ ksi} = 11.68 in^2$

HSS 10×6×1/2 ⇒ $A = 13.5 in^2$

Truss Chords

Load comb 4 : $1.2D + 1.0E + 0.5L_r$ governs

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left[\frac{M_{ux}}{(\phi_b M_{nx})}\right]$$

$\phi = 0.9$ for tension
$\phi = 0.85$ for compression
$\phi_b = 0.9$ for bending

a.) Gravity

$\phi_w = 1.0$

$M = \frac{4.53 \times 10^2}{10} = 45.3'^K$

$P = 611.8^K$

① $1.2D + 1.6L_r$: $P_u = \phi_{L2} P = 1.10(611.8) = 673^K$

$M_u = \phi_{L2} M = 1.10(45.3) = 49.8'^K$

② $1.2D + 1.0E + 0.5L_r$: $P_u = \phi_{L3} P = 0.97(611.8) = 593.45^K$

$M_u = \phi_{L3} M = 0.97(45.3) = 43.9'^K$

b.) Seismic

$M = 39.02'^K$

$\phi_{ecc} M = 1.06(39.02) = 41.36'^K$

$M_u = 41.36'^K$

c.) Comb.

Load case 1 : $M_u = 49.8'^K$
$P_u = 673^K$ *

Load case 2 : $M_u = 43.9 + 41.36 = 85.26'^K$ *
$P_u = 593.45^K$

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$M_u = M_{u,G} + M_{u,S}$

$M_{u,G}$ = Gravity Load Moment = 43.9'k every floor

$M_{u,S}$ = Seismic Load Moment

Chords
Load case 1.2D + 1.6Lr

Floor	Mu	Pu	Section	Eq H1-1a
Roof	50	673	W 10×68	0.931
5	50	673	W 10×68	0.931
4	50	673	W 10×68	0.931
3	50	673	W 10×68	0.931
2	50	673	W 10×68	0.931
1 (ground)				

Chords
Load Case 1.2D + 1.0E + 0.5Lr

Floor	Φ_n	$M_{u,S}$	Mu	Pu	Section	Eq H1-1a
Roof	.34	14	29	593	W 10×54	0.98
5	.60	25	51	593	W 10×60	0.95
4	.79	32	67	593	W 10×68	0.88
3	.92	38	78	593	W 10×68	0.91
2	1.00	41	85	593	W 10×68	0.934
1 (ground)						

H1-1a: $\frac{P_u}{\phi P_n} + \frac{8}{9}\left[\frac{M_{ux}}{(\phi_b M_{nx}}\right]$

$\phi = 0.9$ tension
$\phi = 0.85$ comp
$\phi = 0.9$ bending

12

	$\phi_c P_n$	$\phi_b M_{nx}$
W10 x 112	1400	551
W10 x 100	1250	488
W10 x 88	1100	424
W10 x 77	961	366
W10 x 68	850	320
W10 x 60	748	280
W10 x 54	672	250
W10 x 49	612	227
W10 x 45	565	206
W10 x 39	489	176
W10 x 33	414	146

Mu = 50
Pu = 673 } H1-1a :
0.561 ⇒ W10 x 112
1.41 ⇒ W10 x 45
0.717 ⇒ W10 x 88
[0.931 ⇒ W10 x 68]
1.05 ⇒ W10 x 50

Pu = 593
Mu = 85 } H1-1a ;
try 88 ⇒ 0.717
try 77 ⇒ 0.824
[try 68 ⇒ 0.934]
try 60 ⇒ 1.06

Column Design 13

50% LL reduction assumed – b/c support large area

Col. 1F
- a typical 1 story truss will carry load from 2 floors
- a truss w/ hangers or post, the weight from three floors is carried.

$TA = \frac{56}{2} \times \frac{74}{2} = 1036\ ft^2$

DL_1 (plate loads) = 65psf (HC plank) + 5psf (struct. steel) = 70psf × 1036 = 72.5^K

DL_2 (all walls - exterior) = 122psf × 1036 = 126.4^K

RLL = 20psf × 1036 = 20.7psf

$DL_2 + RLL = 126.4 + 20.7 = 147.1^K$

2 floors

$DL_1 = 72.5 \times 2 = 145^K$

$DL_2 + RLL = 147.1 \times 2 = 294.2^K$

Exterior Wall = 23psf × 28 × 10 = 6.44^K/story

$M_{col} = M_{Trans} + M_{Rot}$

$M_{Trans} = \frac{6EI(\Delta_t + \Delta_b)}{l_c^2}$

$M_{Rot} = \frac{3EI\theta}{l_c}$

where

$\theta = \frac{2\Delta_{TS}}{L}$

$\therefore\ M_{col} = \frac{6EI}{l_c}\left(\frac{\Delta_t + \Delta_b}{l_c} - \frac{\Delta_{TS}}{L}\right)$

(cont.) 14

$\Delta_{TS} = 3/4''$ (assumed)
$L = 74'$
$l_c = 10'$

Top + Bottom Chords W10×68

$$\Delta_t = \frac{\Sigma P_i L_i}{(E A_i)}$$
$$= \left[\frac{(11.167)(12)}{(29{,}000)(20\ in^2)}\right] \times \left(611.8/2 + 611.8 + 519.3 + 315.8\right) \times \left[\frac{70}{122+40}\right]$$
$$= 0.175\ in$$

$$\Delta_b = \left[\frac{(11.167)(12)}{(29{,}000)(20)}\right] \times \left(611.8/2 + 519.3 + 315.8 + 0\right) \times \left[\frac{70}{122+40}\right]$$
$$= 0.114\ in$$

Try W12×65 column

$$M = \left[\frac{6(29{,}000)(174)}{(10\times12)}\right] \times \left[\frac{(0.175+0.114)}{(10\times12)} - \frac{0.75}{(74\times12)}\right]$$
$$= 394.5\ in\text{-}k$$
$$= 32.9'k$$

Col 1F:

		Axial						Moment	Load Comb.			
		Floor		Exterior	Total			DL	1.4 D		1.2D+1.6L	
		DL	DL2+RLL	Wall	DL	DL2+RLL	Ext. Wall		Pu	Mu	Pu	Mu
W12×65	Roof	145	294.2	6.44	145	294	6.44	33	203	46.2	377	39.6
	5			6.44	145	294	12.9		203		385	
W12×96	4	145	294.2	6.44	290	588	19.3	51	406	71.4	762	61.2
	3			6.44	290	588	25.8		406		769	
W12×136	2	145	294.2	6.44	435	882	32.2	75	609	105	1146	90
	ground (1)			6.44k	435k	882k	38.6k		609		1154	

Appendix E

Structural Steel Material Costs							
Section	**Element**	**#**	**Total Length**		**Weight**		**Cost**
			Inches	LF	Kip	Ton	(W - $570/ton, HSS-$700/ton)
W10X33	Beam	158	34816	2901	93.3	46.7	$26,602.30
W10X39	Beam	2	704	59	2.3	1.2	$652.98
W10X77	Beam	2	1776	148	11.2	5.6	$3,192.82
W10X112	Beam	8	7104	592	65.1	32.6	$18,565.30
W18X50	Beam	84	28224	2352	112.9	56.5	$32,176.30
W18X60	Beam	2	672	56	3.2	1.6	$917.87
W21X68	Beam	2	672	56	3.7	1.8	$1,043.03
W24X76	Beam	2	672	56	4.1	2.1	$1,167.83
				6220		147.9	$84,318.44
HSS10X6X.3125	Brace	80	14390	1199	35.7	17.8	$12,486.08
HSS10X6X.375	Brace	24	4317	360	12.7	6.4	$4,447.10
HSS10X6X.500	Brace	16	2878	240	11.0	5.5	$3,848.45
W10X33	Brace	10	2057	171	5.7	2.8	$1,610.57
				1970		32.5	$22,392.19
W12X96	Column	80	9744	812	77.8	38.9	$22,162.45
W12X106	Column	10	1488	124	13.1	6.6	$3,744.46
W12X120	Column	2	240	20	2.4	1.2	$683.31
W14X132	Column	8	1248	104	13.7	6.9	$3,905.51
HSS10X6X.3125	Column	164	19680	1640	48.8	24.4	$17,075.91
HSS10X6X.375	Column	16	1920	160	5.7	2.8	$1,977.83
				2860		80.7	$49,549.46
		Total of all Members:		11050		261.2	$156,260.08

Flooring & Roofing System Material Costs			
Material	**$/SF**	**SF**	**Cost**
8" Plank	$7.95	15,850	$126,007.50
Green Roof	$20.00	15,850	$317,000.00
			$443,007.50

Truss Fabrication Costs				
Members	**# of Members**	**Ton**		
Diagonal	120	30		
Vertical	164	28		
Chord	160	48		
		106		
Fabrication = $1600/ton:		$1600 x 106 =		$169,600.00

Labor Costs			
8" Plank	$0.84	15,850	$13,314.00
Trusses	$360.00	106	$38,160.00
Spandrel Beams	$360.00	100	$36,000.00
Columns	$360.00	54	$19,440.00
Bracing/Kickers	$360.00	3	$1,080.00
			$107,994.00

Equipment Costs			
8" Plank	$0.45	15,850	$7,132.50
Trusses	$169.00	106	$17,914.00
Spandrel Beams	$169.00	100	$16,900.00
Columns	$169.00	54	$9,126.00
Bracing/Kickers	$169.00	3	$507.00
150 ton crane	$1,445.00	30	$43,350.00
			$94,929.50

Overall Total Cost		
Steel		$156,260.08
Flooring & Roofing		$443,007.50
Steel Fabrication		$169,600.00
Labor/Erection		$107,994.00
Equipment		$94,929.50
Connections		$31,252.00
		$1,003,043.08

Existing System Costs			
Materials	**Quantity (Lump Sum)**	**U/Price (Lump Sum)**	**Budget (Cost)**
Masonry	1.00	$627,849.00	$627,849.00
Masonry Reinforcement	1.00	$70,935.00	$70,935.00
Structural Steel	1.00	$548,600.00	$548,600.00
Cold Formed Metal Framing	1.00	$176,200.00	$176,200.00
	Total Cost (including labor) =		$1,423,584.00