Proposed Alternate Lateral Resistance Systems for Whiteland Village: Staggered Truss and Partially Restrained Composite Connections

Senior Thesis
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WHITELAND VILLAGE

Exton, PA

Building Statistics

Occupancy: Retirement Village
Size: 1,320,000 square feet
Cost: $100-150 million
Construction: November 2006-November 2008
Delivery Method: Design-build

Structure

- Precast concrete plank supported by steel beam floor system and CMU bearing wall
- Masonry shear wall, steel moment frame, and steel braced frame lateral resistance systems
- 5" slab on grade with shallow spread footings
- Cold-form steel trusses and overframing at residence roofs

HVAC

- Energy wheel uses recycled energy to condition air before it enters the building
- Chilled water cooling and gas hot water heating provided via a central distribution system

Fire Protection

- Wet system throughout complex
- Standpipes pressurized using 1000 GPM electric fire pump in residence buildings
- No standpipes in commons building

Project Team

Owners: Whiteland Village, Roskamp Management Co., LLC
Architects: Dever Architects, HLM Design
Construction Manager: Paul Risk Associates, Inc.
Structural: Baker, Ingram & Associates
HVAC: Madsen, Inc.
Electrical: Meadow Valley Electric
Fire Protection: Marco, Inc.
Geotechnical: CVM Industries
Civil: Edward B. Walsh
Pool Design: Wade Associates

Architecture

- 3 residential buildings, commons, and healthcare facility
- 6 total stories, 5 above grade
- Mansard roofs clad in fiberglass roof shingles on residences
- Grass roof with water features on commons building
- Manufactured stone and vinyl siding exterior barrier walls on all buildings
- All condominiums have a balcony or porch
- Basement level for underground parking below entire footprint

Electrical

- 208Y/120V service from (1) 5kV transformer
- 1000A distribution panel at each floor of residences
- Individual panels in every condominium
- Co-generation plant onsite, only on power grid during off-peak hours

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http://www.arche.psu.edu/thesis/eportfolio/2007/portfolios/MEL922

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Executive Summary

This report is intended to be a detailed description and preliminary analysis of the structural design of Whiteland Village in Exton, PA. Whiteland Village is a 1,320,000 sq. ft. sprawling retirement community, which is slated for completion by November 2008. The physical components of the first phase of the complex include three 5 story residence buildings, a commons building, and a healthcare facility. The entire footprint has a basement level, which serves as covered parking and utility spaces. The phase one construction will be on the west side of the campus, including U-1 (renamed R-1), U-2 (renamed R-4), and the J building (renamed R-2). The other buildings will go into planning as soon as Whiteland Village becomes profitable, and will be connected with a pedestrian link.

The analysis for this proposal only examines the most current design of the three residence buildings, which were designed by Dever Architects. The current structural system consists of 8" hollow core precast plank, spanning approximately 30' between 10" CMU bearing walls. Lateral loads are resisted by a combination of concrete and masonry shear walls, steel moment frames, and steel braced frames.

In order to reduce building weight and increase the potential for future renovation, the possibility of changing Whiteland Village to steel frame is being investigated. To determine the feasibility of this proposal, two different lateral systems will be investigated: staggered truss and partially restrained composite connections.

As a result of this analysis, it was determined that while staggered truss is a viable alternate structurally, economically it is not a better choice than the current shear wall system. Partially restrained composite connections were not a feasible alternative system with the braced frame plan that was analyzed.

Existing building envelopes were also analyzed as a part of this study. Suggestions were made to improve the building envelope at window openings. This included extended drip edges and the addition of a drainage cavity to the wall section. Detailing at mechanical wells was determined to be sufficient for removal of bulk water and snow. Moisture penetration was also addressed.

Building Overview

Whiteland Village is a 1,320,000 sq. ft. sprawling retirement community slated for completion in November 2008. Located in Exton, Pennsylvania, the site is 30 miles from Philadelphia at the intersection of Route 30 and Business 30. The master plan for the complex is included in Appendix A and a smaller version as Figure 1. Three residence buildings, a commons building, and a healthcare facility will be constructed in the initial phase of construction.

Figure 1: Master Plan for Whiteland Village

Located on the western half of the campus, the residence buildings in the phase one construction are labeled U-1 (renamed R-1), U-2 (renamed R-4), and the J building (renamed R-2). Sketches of the shapes and orientations of the three residences are shown in Figure 2. Locations of expansion joints are indicated with green dashed lines. Floor plans for these buildings are located in Appendix B. As Whiteland Village becomes profitable, the east side of the site will be developed and connected to the original construction with a pedestrian link.

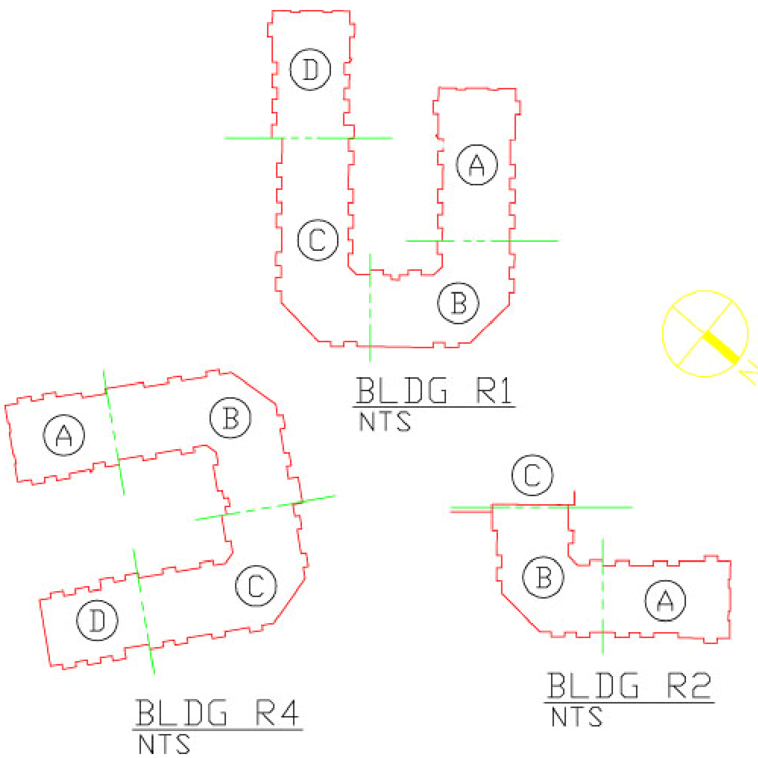


Figure 2: Sketches of Residence Buildings

Currently, phase one of Whiteland Village has a projected total cost of $100-150 million. Key construction dates for the design-build project are summarized in Table 1.

	Contract Award	Pricing for GMP	Actual Construction Start Time	End Time
Site	Nov-06	Nov-06	Nov-06	Nov-08
Residence 1	Dec-06	Nov-06	Feb-07	Jul-08
Residence 2	Dec-06	Dec-06	Jul-07	Jul-08
Commons	Dec-06	Dec-06	Jul-07	Jul-08
Healthcare	Apr-07	Mar-07	Jul-07	Nov-08

Table 1: Key Construction Dates

Designed by Dever Architects, the intent was to have the residence buildings resemble oversized suburban single family homes. This look was achieved through the use of mansard roofs sheathed with fiberglass shingles and vinyl siding. HLM Design was hired to design the commons and healthcare facility. Since the design was not yet completed for these two buildings, the focus of this thesis will be the three residence buildings. The following is a brief synopsis of each of the critical building systems.

Architecture

Whiteland Village is an expansive retirement center consisting of three residential buildings, a commons building, and a healthcare building that includes residential units. Covered parking and utility spaces are provided in a basement level which spans the entire footprint of the complex. Each of the residences and the healthcare building are connected to the commons building with links located on the ground floor, as well as the basement garage level. The residence buildings are five story CMU structures with mansard roofs, in an effort to give the condominiums appearance of traditional housing. Each unit in the complex has its own balcony or porch. The porches on the first floor of the residences are on the same level as the roof of the commons building.

Currently, the commons roof is planned to have a live lawn with water features, bocce courts, putting greens, barbeque pit, and outdoor cocktail bar. Deliberations as to the feasibility of this plan are ongoing; an Astroturf alternate is being considered. In addition to the facilities on the roof, the commons building includes an auditorium, full service restaurant, library, demonstration kitchen, administrative areas, fitness center, spa, woodworking shop, conference rooms, ice cream parlor, and game rooms. The healthcare building, with the planned addition of residential living options on the upper floors, has not yet been designed.

Structure

The residence buildings in Whiteland Village are constructed using untopped 8" hollow core precast plank spanning 30', bearing on 10" CMU walls. At openings in the masonry walls, the plank is supported by structural steel, typically W18x31 beams and (2) L6x4x3/4 angles. Lateral loads are resisted by a combination of concrete and masonry shear walls. Due to a code-related fire protection requirement, the section of the R-2 building adjacent to the commons is built with precast supported by steel moment frames and steel braced frames. A more detailed description of the existing structural system is included in later sections.

HVAC

Air circulation occurs in the residences through a central exhaust system using an energy wheel. This allows for the use of recycled energy to condition air before it enters the building, significantly reducing overall heating and cooling costs. Chilled water cooling and gas hot water heating fulfill the heating and cooling needs of residents via a central distribution system.

Electric

Electrical service is 208Y/120V from a single 5kV transformer. A 1000A distribution panel regulates each floor of the residence buildings, which links to individual panels in each condominium. Power for the entire complex is created by an onsite co-generation plant; Whiteland Village is only on the power grid during off-peak hours and during emergencies requiring backup power.

Fire Protection

Throughout the community, fire protection is provided by a wet sprinkler system. In the residences, a 1000 GPM electric fire pump is used to pressurize the standpipes.

Envelopes

The exterior cladding of the residences consists of manufactured stone and traditional vinyl siding, with fishscale vinyl siding under the roof eaves. Building entrances are detailed using a manufactured store front system. Roofing is comprised of fiberglass shingles on 5/8" plywood supported by cold-form steel over-framing. At the center of the building, sloped insulation is used to direct water into roof drains. Envelopes are discussed in greater detail in later sections.

Code Requirements and Design Theory

Due to the size and the location of Whiteland Village, it is being designed to be acceptable to both the West Whiteland Township Building Code, as well as the East Whiteland Township Building Code. Both codes are based on the 2000 International Building Code (IBC), which is published by the International Code Council and heavily reference ASCE 7-98. In addition, the municipalities have accepted the 1997 Fire Prevention Code, from the National Fire Protection Association (NFPA). Local code requires all buildings to have a mean roof height of 65' or less, which greatly impacted many choices during the design process.

All structural systems for Whiteland Village were designed using the Allowable Stress Design (ASD) Method. In the design of Whiteland Village, the American Institute of Steel Construction's (AISC) Manual of Steel Construction was utilized. This is the accepted industry standard for steel construction. The Building Code for Reinforced Concrete published by the American Concrete Institute (ACI 318), the Precast/Prestressed Concrete Design Handbook (PCI), and the Building Code Requirements and Specifications for Masonry Structures (ACI 530) were referenced during design as industry standards.

Description of Existing Structural System

Due to the height restrictions imposed by local ordinance, creating Whiteland Village residence buildings from masonry and precast plank would be a logical conclusion. By using this combination, the floor to floor structural sandwich depth could be reduced to simply the thickness of the plank used. Therefore, higher floor to floor heights, as well as an extra floor, could be achieved.

Framing and Lateral Load Resistance

Gravity loads are taken into the overall building structural system by untopped 8" hollow core precast plank spanning approximately 30' at each level. The planks will be designed by the precast contractor to meet the required capacity. Both bearing plank ends frame into a 10" CMU wall that runs from the 5th floor ceiling to the 1st floor. These reinforced masonry walls also act as shear walls for the system, transferring lateral loads from the higher floors to the first floor. PCI 3.6.2 allows for untopped precast plank to be considered as a rigid diaphragm if shear keys are grouted. On this project, all shear keys are to be grouted solid to ensure shear transfer between planks. The following is a sketch of the masonry shear walls and plank spans for a typical intermediate floor (Figure 3).

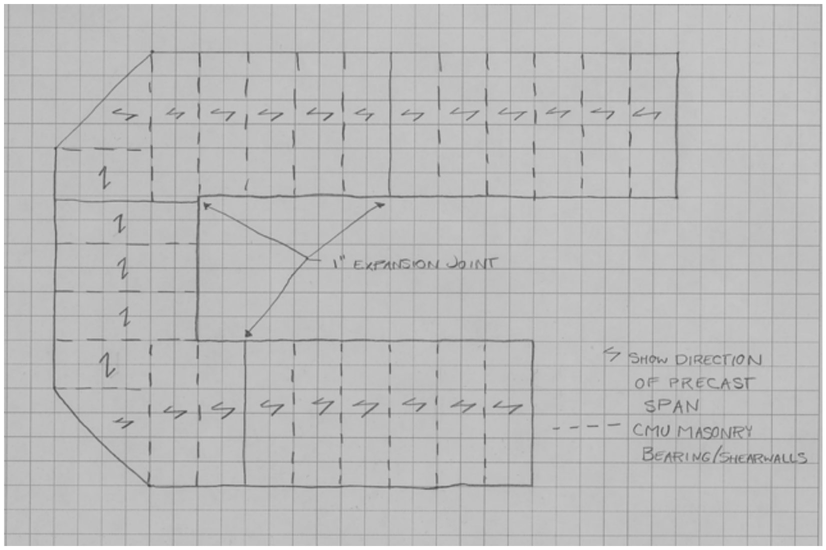


Figure 3: Sketch of Shear Wall Locations and Plank Spans on Typ. Floor

Grade A992 wide flange steel beams are positioned under the 5 story walls to pick up the loads, so the basement can have the open space necessary to allow vehicular traffic. These beams range from a W18x50 to a W36x359, with spans of 7’4” to 30’0”.

The typical masonry shear wall, including basement layout, is seen in the sketch in Figure 4. It consists of two W12x96 columns with a W33x201 spanning between and a W18x119 spanning from the column and bearing on the masonry walls on each side. As seen in Figure 4, most of the masonry shear walls have openings, some which do not stack floor to floor.

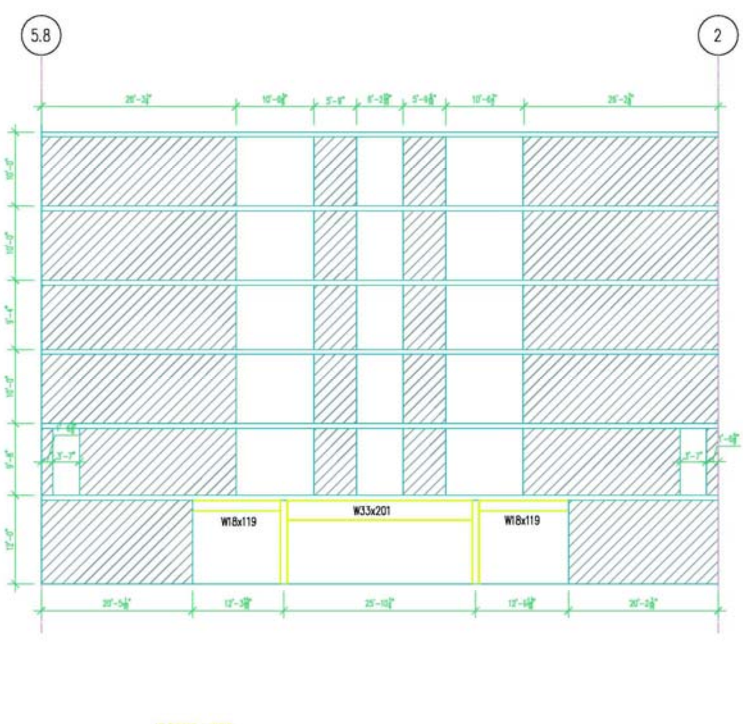


Figure 4: Elevation of Typ. Masonry Shear Wall

At the first floor, lateral loads are redistributed to the building perimeter. By doing so, the basement columns are only required to resist gravity loads and axial loads induced by the overturning of the lateral system above. In design, this means the basement columns can be smaller and be attached with simpler pin connections to the first floor framing. Special checks are required to design the basement steel because of the discontinuity of structural systems.

In the long direction of each section, lateral loads are resisted by 10” concrete shear walls. These shear walls are located in the exterior walls and therefore have large openings for windows. Figure 5 illustrates the elevation of the Type 2 shear wall.

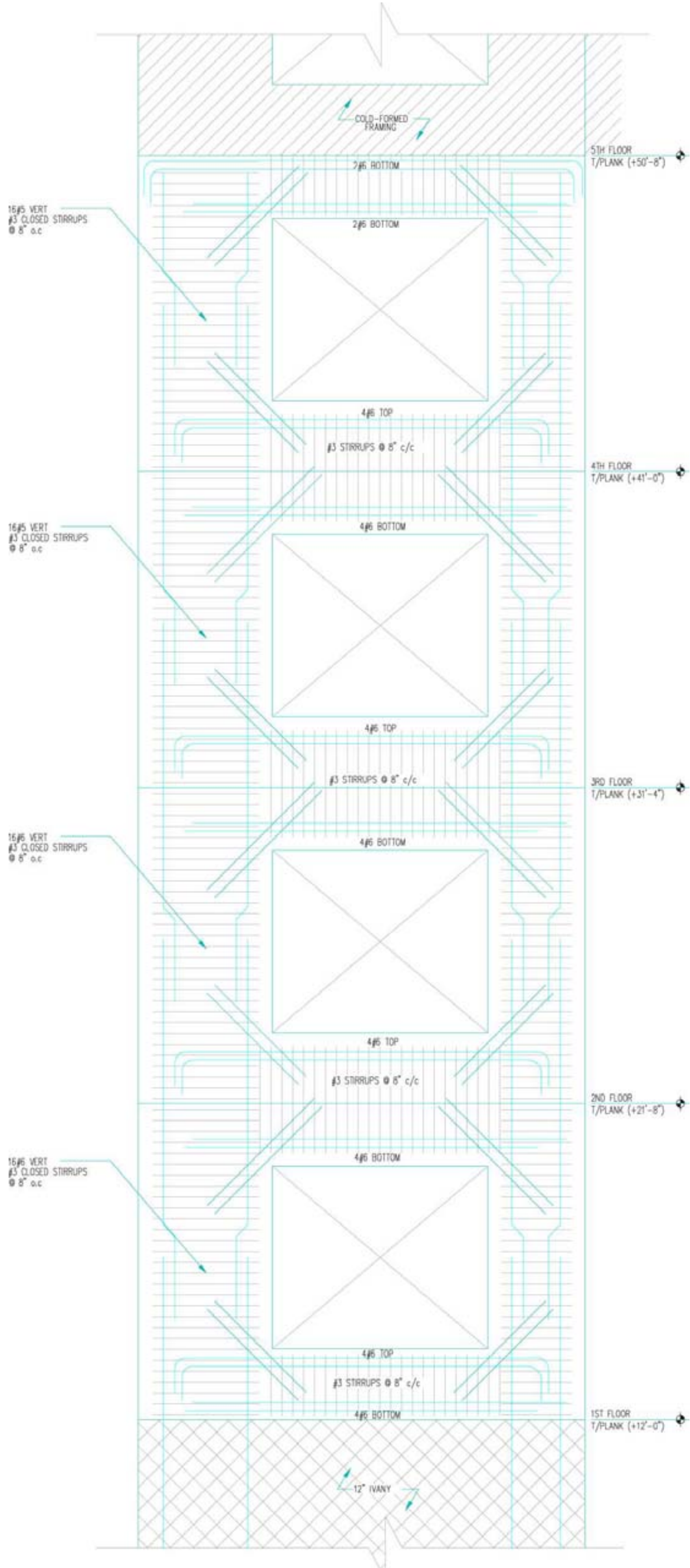


Figure 5: Elevation of 10” Concrete Shear Wall Type 2

There is one section of the residence buildings which differs from this basic plan. In the J Building (R-2), the first section between the commons (separated with a 2” expansion joint) and the 2” expansion joint within the building footprint is 8” hollow core precast plank, spanning 15’ and 30’. On the top 2 floors, the plank bears on a 12” CMU wall. At the third floor, the precast is supported by a wide flange A992 steel frame. Framing members range from a W30x90 to a W36x194 beam size. To resist lateral loading, the third floor framing is braced with W8x31 knee braces. The second floor has no framing because it is part of a 2 story atrium. At the first floor, the steel framing is connected with moment connections to resist lateral load, ranging in size from W24x49 to W24x131. The location of this section is indicated with shading on the following sketch of the J Building in Figure 6. This section of the residence building was not studied in depth due to its extremely different framing and lateral systems.

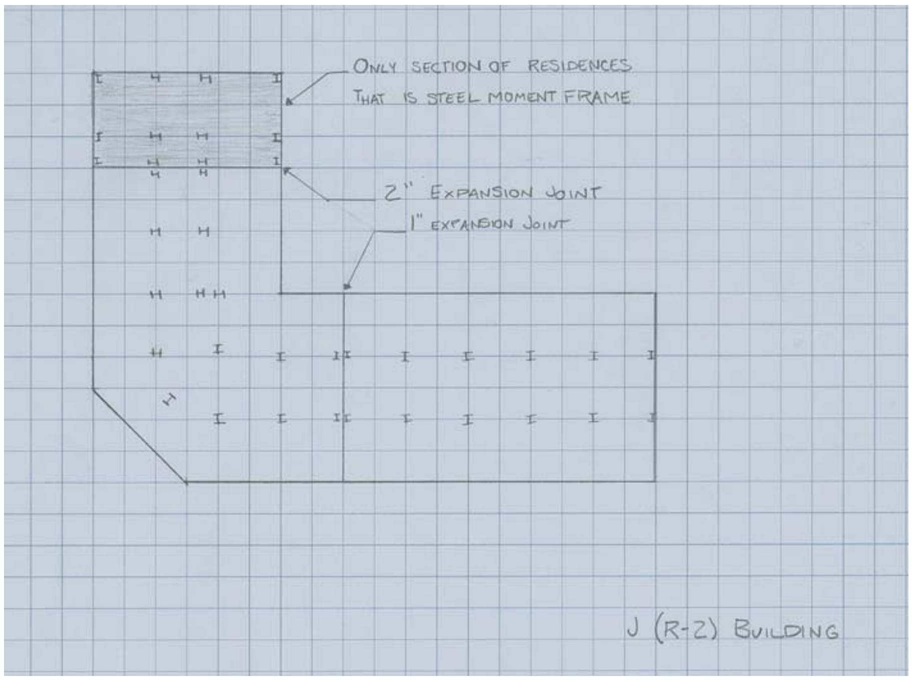


Figure 6: Sketch of J (R-2) Building Floor Plan

Floor System

The current floor system in Whiteland Village is untopped 8” hollow core precast plank, spanning approximately 30’ and bearing on 10” CMU shear walls. Since the buildings are condominiums, the floor system was designed for the following loads, as dictated in the code by ASCE 7-98:

<u>Live Loads</u>

- 40 PSF in dwelling areas
- 100 PSF in corridors
- 100 PSF in stairs
- 125 PSF in storage areas

Dead Loads

Floor
- 8" Hollow Core Plank – 60 PSF
- HVAC – 5 PSF
- Ceiling – 2 PSF
- Partitions – 10 PSF
- Misc. – 3 PSF

Total – 80 PSF

Roof
- Roofing – 2 PSF
- HVAC – 3 PSF
- Ceiling – 2 PSF
- Insulation – 3 PSF
- Precast Plank – 60 PSF
- Misc. – 5 PSF

Total – 75 PSF

Typical sections detailing the connections used in the current system are seen in Figures 7 and 8.

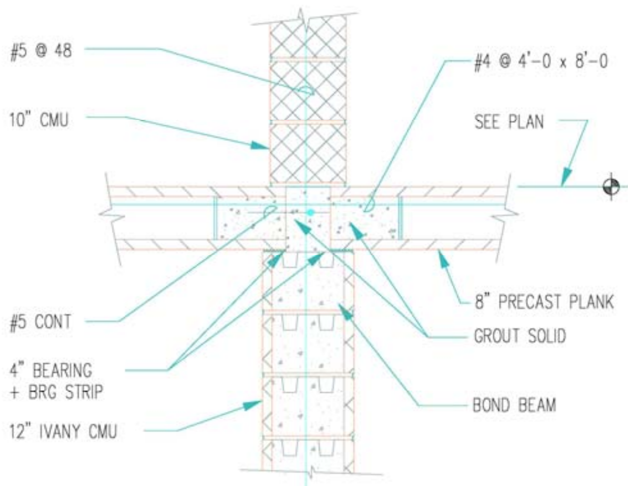


Figure 7: Section of 8" precast plank bearing on 10" CMU wall

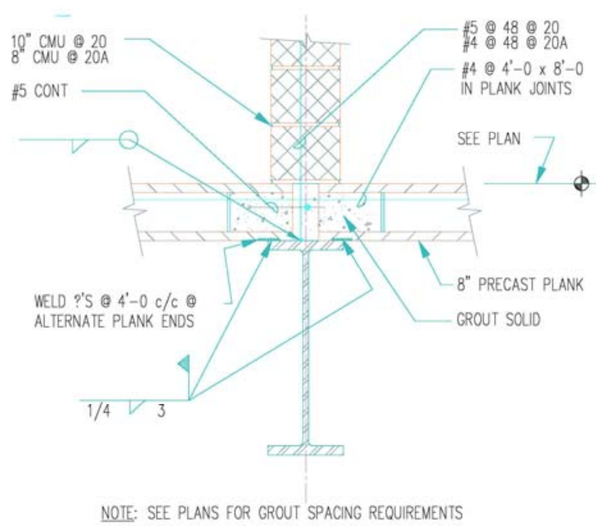


Figure 8: Section of 8" precast plank bearing on W-shape

Precast plank has distinct benefits that are easily utilized in the Whiteland Village project. The floor systems are limited to a maximum depth of 1'-8" due to the zoning restriction on height, restricting mean roof height to 65'. In addition to allowing more plenum space for mechanical systems, a shallower floor system allows for higher ceilings in the condominiums.

The following are other benefits of this shallow floor system:
- Ready to assemble upon delivery
- Easily sequenced
- Installation not impacted by weather
- 1 or 2 hr assembly rating
- Excellent sound attenuation

Foundations

The foundation system of Whiteland Village consists of a 5" slab on grade, reinforced with 6x6 – W2.9xW2.9 welded wire fabric, on top of 4" of drainage fill, with a continuous spread footing around the entire perimeter and under all interior foundation level walls. This spread footing is typically 3' wide when supporting exterior walls, and 6' wide when supporting interior wall sections. Interior columns are supported by spread footings, which range in size from an 8' square to a 12'x19'. The footings are approximately spaced in a 30'x30' grid running through the center of the building. There are thickened slabs below all elevator shafts. The foundation system is very shallow, with the top of the deepest footing only 3'4" below the top of the slab.

The use of a shallow system seems very logical for this site. Due to extremely poor site soils, the first geotechnical report dictated Site Class F for seismic design. In order to obtain a Site Class D, at least 10' of soil between the foundation and hard bedrock had to be provided using compaction grouting. A deep system may disrupt the integrity of the compacted soil.

All reinforced concrete in the foundation is 3000 psi, and is reinforced with 60 ksi rebar. The reinforced CMU exterior foundation walls are designed to withstand 68 PCF of equivalent fluid pressure from the surrounding soil, as dictated by the geotechnical report. Figure 9 is a rough sketch showing the column layout in the U2 building, which is typical for all the residences.

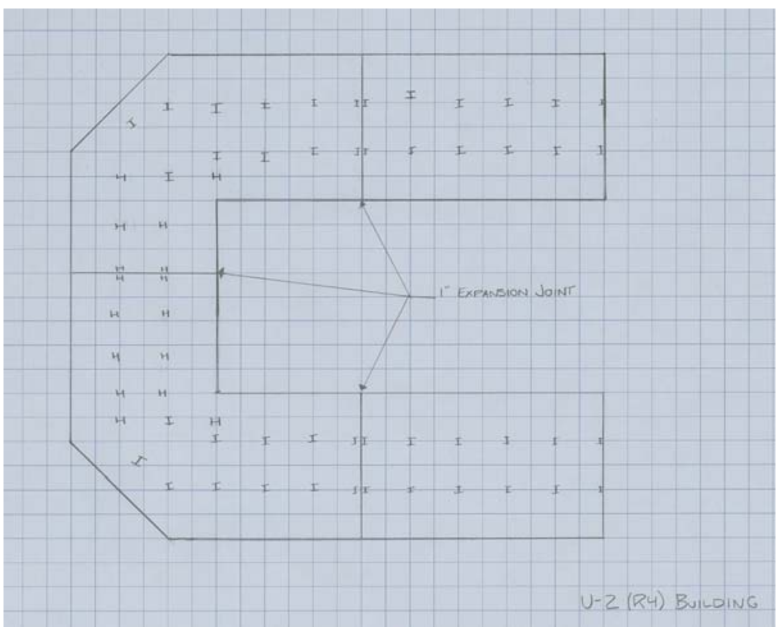


Figure 9: Column layout in U-2 Building

Statement of Problem

One of the major issues with the current structural system is the difficulty of future renovations. The use of shear walls would make changing door or opening locations very challenging, especially considering the numerous openings already detailed. Many condominium owners want to be able to customize their space to make it fit their individual lifestyles. This is nearly impossible with the current structural system. Changing from shear walls to a steel system would significantly increase the flexibility of renovating the condominiums.

The new steel-based lateral system would need to be relatively shallow, less than 1'-8", due to the 65' maximum building height zoning restriction described earlier. However, the new lateral system could potentially be floor to floor where shear walls are currently located. Utilizing a different lateral system could also result in using an alternate floor system. This would also have to meet the floor ceiling envelope requirements previously outlined. Additionally, the floor system would also need to be designed to resist all dead loads, equipment loads, and live loads as outlined in ASCE 7.

Another issue with the current structural system is the weight. Since so many of the walls are 10" CMU and are grouted at less than 48" intervals, the walls and columns in the basement level have to carry significant gravity loads. By switching to an alternative system, the reduced gravity loads would result in a smaller seismic load.

Other considerations relating to changing the nonbearing walls or using an alternate floor system include the following:

- Constructability
- Sound attenuation
- Vibration
- Fire protection
- Depth and size
- Cost
- Weight
- Impact on architecture

In order to determine if either alternate is feasible, the A section of building R4 (see Figure 2) will be analyzed.

Proposed Alternate Lateral Systems

In order to solve some of the issues discussed above, changing the building to steel seems like a logical switch. Steel would provide more of an opportunity for open framing, making future renovations feasible. Changing to steel would allow alternate lateral resistance systems to be used. The two that will be investigated are staggered truss and partially restrained composite connections. In both cases, a maximum overall structure depth of 1'-8" will be used and columns will be limited to 10" flange widths so that room dimensions are not affected by the change in systems.

Staggered Truss

The staggered truss system was developed in the 1960s. The basic elements of the system are full story trusses that span the entire transverse width of the building at alternate floors at every column line. Gravity loads are transferred through the floor diaphragm to the top chord of a truss and the bottom chord of the adjacent truss. It is advantageous in buildings that are long and narrow, because typically the wind loads on the large face of the building are substantial and need to be resisted in the smaller dimension. The following is an example of the framing for a staggered truss system (Figure 10).

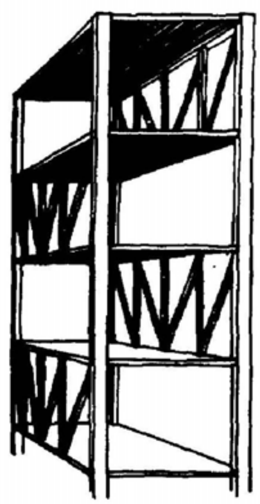


Figure 10: Example of Staggered Truss Framing

The current system shear walls provide an excellent layout for trusses. The floor-to-floor height restrictions will not affect the depth of the beams within the truss; however, allowances for future renovations may dictate revisions to the layout. Additionally, either the existing floor system could be used or the system could change to an alternate, such as composite.

Partially Restrained Composite Connections

Another alternate lateral system is to use a steel frame with a composite slab using Type 3, or partially restrained connections. Partially restrained composite connections (PR-CCs) are flexible moment connections in which the reinforcing in the composite floor slab is used to create the top portion of the moment resistant mechanism. In traditional moment connections, this top portion would be an angle or plate on top of the slab. Shear resistance is provided at the bottom of the connection by a steel seat angle. A section of a typical PR-CC is seen in Figure 11.

Advantages to PR-CCs include:

- Eliminate top angle or top plate
- Capacity controlled by amount of steel
- Composite action reduces beam size, deflection, and vibration
- Eliminate need to cut and shore decking around supports

This connection type suits the building well since it is most applicable to buildings less than 10 stories and in an area with low to moderate seismic risk.

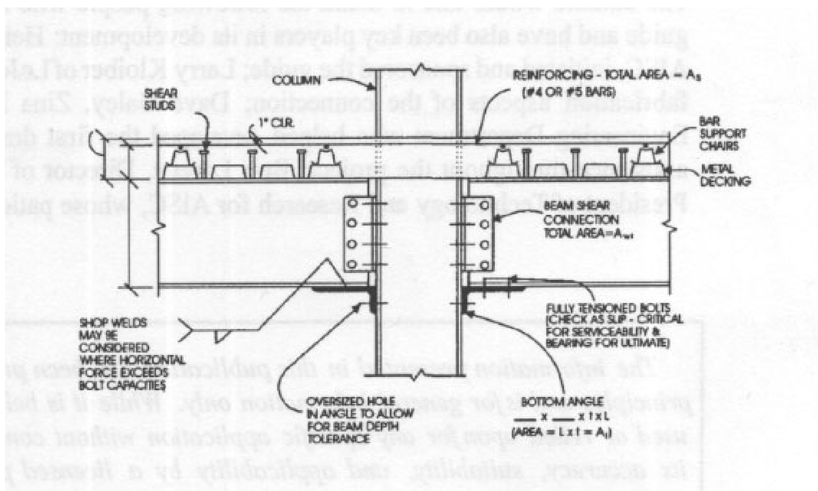


Figure 11: Section of 8” precast plank bearing on W-shape

Staggered Truss Lateral System Alternate

The staggered truss lateral system was designed using the Load and Resistance Factor Design (LRFD) Method and the procedures outlined in *AISC Steel Design Guide 14: Staggered Truss Framing Systems*.

Framing Layout

Figure 12 shows the typical floor plan for the 5-story staggered truss building. Columns are oriented with the strong axis parallel to the short direction of the structure. In the long direction, two moment frames, spanning the entire building length, are used to resist lateral loads. Six staggered trusses span the short direction. Gravity-only framing was added as necessary to carry the loads from the bump-out sections. It was assumed that these sections, extending a maximum distance of 12’ from the end of the truss, would not impact the design of the lateral system.

The geometry chosen for the bracing was that of a Pratt truss, where diagonal members are in tension when gravity loads are applied. Two different truss types, T1 and T2, are indicated on the floor plan. Truss T1 has bracing on the odd floors and T2 on the even floors. The truss elevations are included in Figures 13 and 14. Vierendeel panels are 10’-8” wide to allow for the corridor running through the center of the building. This layout also allows for the garage to remain open to allow for parking and driving lanes. Trusses span 78’ from column line 3 to column line 6.4.

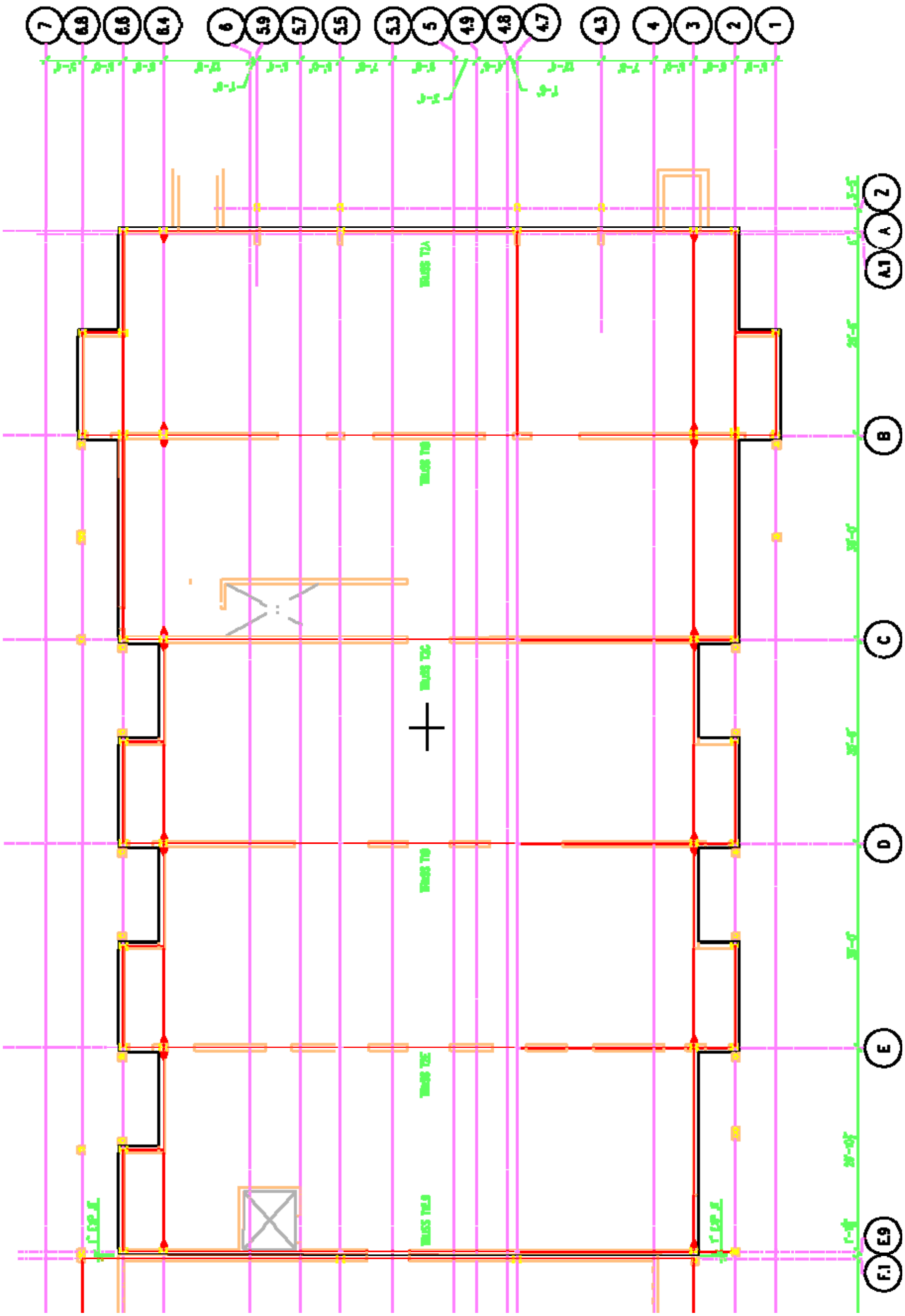


Figure 12: Structural Plan for Proposed Staggered Truss System

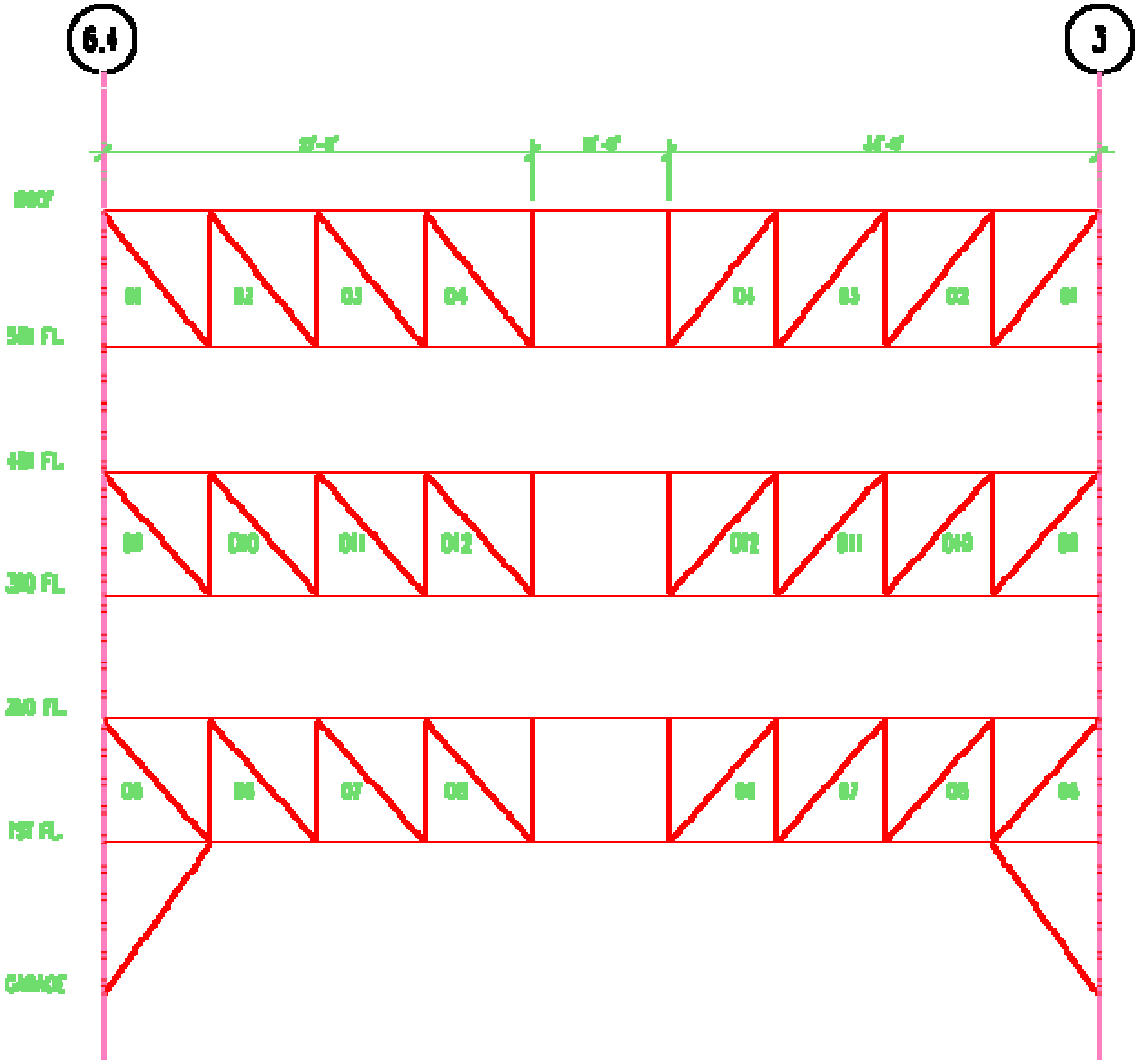


Figure 13: Staggered Truss Type T1

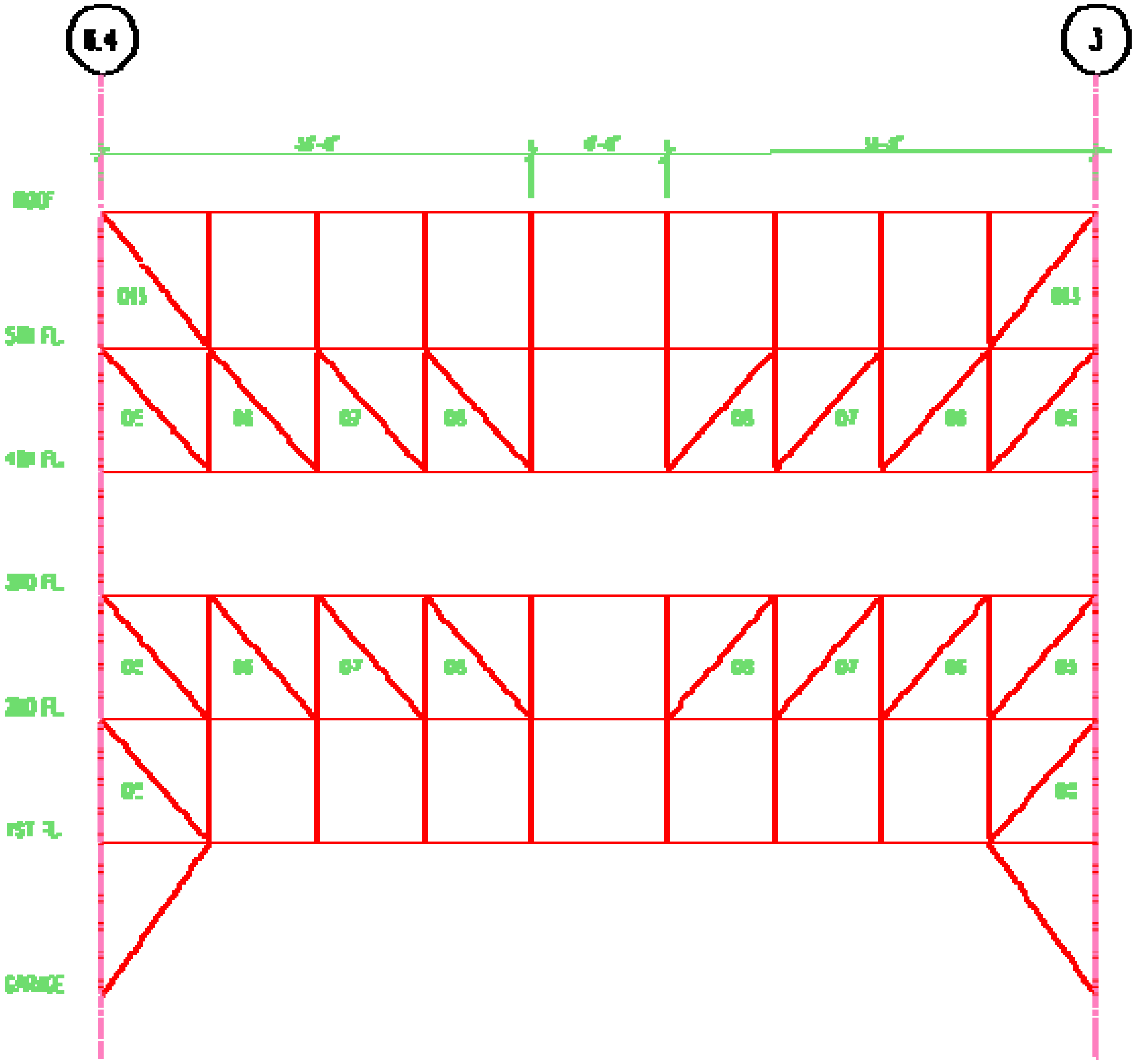


Figure 14: Staggered Truss Type T2

Gravity Loads

The following outlines the loads used in the calculation of this redesign. The existing floor system, 8" untopped hollow-core precast plank, was utilized to keep the depth of the structural sandwich minimal. To allow for the assumption of the floor acting as a rigid diaphragm, all shear keys will be grouted to ensure shear transfer between planks (PCI 3.6.3). Loads were determined using ASCE 7-02 and manufacturer data.

<u>Dead Loads</u>

Floor

- 8" Hollow Core Plank – 60 PSF
- HVAC – 5 PSF
- Ceiling – 2 PSF
- Partitions – 20 PSF
- Misc. – 3 PSF
- Structural Steel – 5 PSF

Total – 95 PSF

Plate

- Precast Plank – 60 PSF
- Structural Steel – 5 PSF

Total – 65 PSF

Roof

- Roofing – 2 PSF
- HVAC – 5 PSF
- Ceiling – 2 PSF
- Insulation – 3 PSF
- Precast Plank – 60 PSF
- Misc. – 3 PSF

Total - 75

Wall

- Manufact. Stone – 12 PSF
- Studs – 3 PSF
- Sheetrock – 3 PSF
- Insulation – 2 PSF

Total – 20 PSF

<u>Live Loads</u>

Floor - 40 PSF
Roof – 30PSF

Determination of Controlling Lateral Load

Lateral loads were determined in accordance with ASCE 7-02

Seismic

Seismic base shear was determined using Method 3: Equivalent Lateral Force (ELF). S_s and S_1 were determined using the USGS Hazards Program, inputting Lat 40.5^o and Long 75.6^o. Conservatively, a seismic response modification factor (R) of 3 was used for the overall behavior of the staggered truss system so no special seismic detailing would be required. This follows the recommendations of Design Guide 14 for areas of low to moderate seismic activity. In order to reduce base shear, the period was also assumed to be T=Cu*Ta.

Floor Area=	13398	sq.ft. per floor
Wall Length=	570	ft
Story Height=	9.67	ft

	Floor DL	Floor Wt.	Wall DL	Wall Wt.	Wx	Hx
	(PSF)	(k)	(PSF)	(k)	(k)	(ft)
Roof	75	1004.85			1004.85	49.35
5	95	1272.81	20	110.24	1383.05	38.68
4	95	1272.81	20	110.24	1383.05	29.01
3	95	1272.81	20	110.24	1383.05	19.34
2	95	1272.81	20	110.24	1383.05	9.67
1	95	1272.81	20	110.24	1383.05	0.00
Sum					7920.09	

Table 2: Building Weight Determination for Seismic Analysis

IBC 2003 (ASCE 7-02)

Seismic Analysis, Equivalent Lateral Force

Seismic Design Category	**B**
S_1 =	**0.081** g
S_{MS} =	**0.4614** g
S_{M1} =	**0.1944** g
S_{DS} =	0.3076 g
S_{D1} =	0.1296 g
I_E =	**1**
No. Stories = N =	**5**
C_u =	**1.64**
R =	**3**
h_n =	49.35 ft
C_t =	**0.02**
x =	0.75
$T_a = C_t (h_n)^x$ =	0.3724 sec
$T_{max} = C_u T_a$ =	0.611 sec
T =	0.6107 sec
$C_s = S_{DS} / (R / I_E)$ =	0.1025
$C_{s\,max} = S_{D1} / (R / I_E) / T$ =	0.0707
$C_{s\,min} = 0.044\, S_{DS}\, I_E$ =	0.0135
$C_{s\,min} = 0.5\, S_1 / (R / I_E)$ =	N/A
C_s =	0.0707

$F_x = C_{vx} V$ k = 1.06

$C_{vx} = w_x h_x^k / \text{Sum}(w_i h_i^k)$ (from i to n)

x = 1	w_x = **1383** kips	h_x = **9.67** ft	$w_x h_x^k$ = 15164	C_{vx} = 0.0683	F_x = 31.6 kips
x = 2	w_x = **1383** kips	h_x = **19.34** ft	$w_x h_x^k$ = 31513	C_{vx} = 0.1419	F_x = 65.6 kips
x = 3	w_x = **1383** kips	h_x = **29.01** ft	$w_x h_x^k$ = 48343	C_{vx} = 0.2177	F_x = 100.7 kips
x = 4	w_x = **1383** kips	h_x = **38.68** ft	$w_x h_x^k$ = 65492	C_{vx} = 0.2949	F_x = 136.4 kips
x = 5	w_x = **1005** kips	h_x = **49.35** ft	$w_x h_x^k$ = 61545	C_{vx} = 0.2772	F_x = 128.2 kips
x = 0	w_x = **0** kips	h_x = **0** ft	$w_x h_x^k$ = 0	C_{vx} = 0	F_x = 0.0 kips
x = 0	w_x = **0** kips	h_x = **0** ft	$w_x h_x^k$ = 0	C_{vx} = 0	F_x = 0.0 kips
x = 0	w_x = **0** kips	h_x = **0** ft	$w_x h_x^k$ = 0	C_{vx} = 0	F_x = 0.0 kips
			Sum $(w_i h_i^k)$ = 222057	V = Sum (F_i) =	462 kips
				Check:	OK

Structure Weight	W = Sum (w_i) =	6537 kips
Base Shear	V = Cs W =	462 kips

Per ASCE table 9.5.5.3.1					
S_{D1}	>=0.4	0.3	0.2	0.15	<=0.1
C_u	1.4	1.4	1.5	1.6	1.7

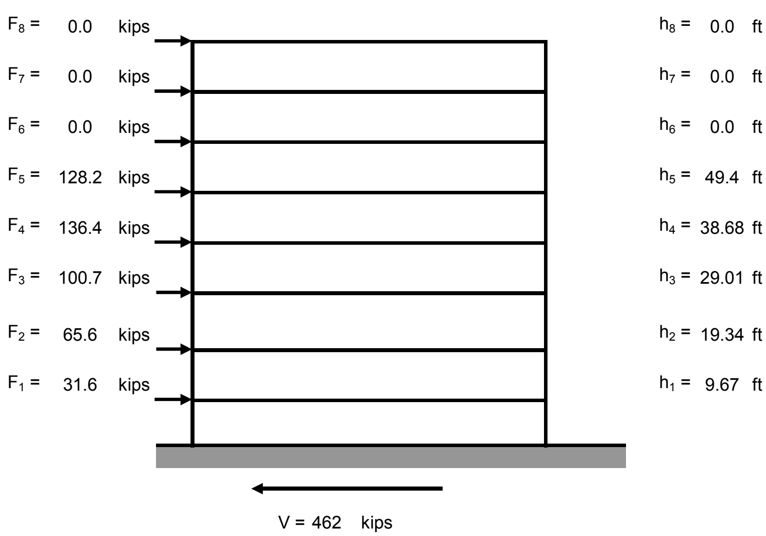


Figure 15: Seismic Load Calculations for Staggered Truss System

Wind

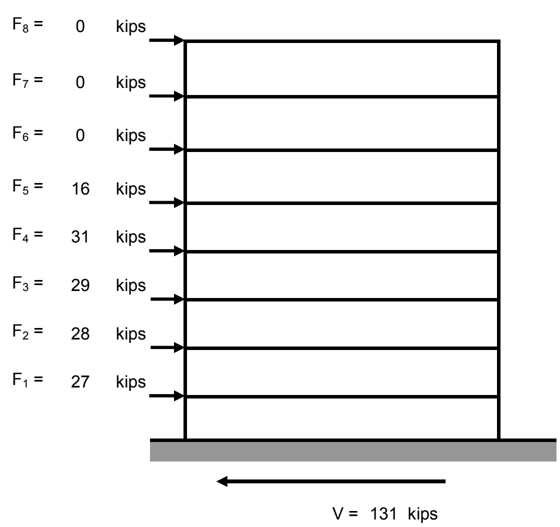
IBC 2003 (ASCE 7-02)

Wind Analysis, Main Windforce Resisting System, Analytical Procedure, EW Direction

Parameter	Value
h =	**52.7** ft
No. Stories = n =	**6**
L =	**91.00** ft
B =	**150.80** ft
L / B =	0.60
V =	**90.0** mph
K_d =	**0.85**
I =	**1.00**
Exposure Category	**C**
Enclosure Classification	**E**
Exposure Case	Case 1
Π =	9.5
z_g =	900 ft
K_1 =	**0**
K_2 =	**0**
K_3 =	**0**
K_{zt} =	1
T =	**0.6107** sec
f =	1.64 hz
G =	0.85
Structure is rigid. G = 0.85.	

$p_W = q_z G C_{pW}$	$q_z = 0.00256 K_z K_{zt} K_d V^2 I$	K_h = 1.11
$p_L = q_h G C_{pL}$	(Windward) C_{pW} = 0.8	q_h = 19.5 psf
$p_{TOT} = p_W - p_L$	(Leeward) C_{pL} = **-0.5**	p_L = -8.29 psf

	Height	K_z	q_z	p_W	p_{TOT}	F_x
	4.84 ft	0.85	14.96	10.17 psf	18.46 psf	
x = 1	h_x = **9.67** ft	0.85	14.96	10.17 psf	18.46 psf	27 kips
	14.51 ft	0.85	14.96	10.17 psf	18.46 psf	
x = 2	h_x = **19.34** ft	0.90	15.78	10.73 psf	19.02 psf	28 kips
	24.18 ft	0.94	16.54	11.25 psf	19.54 psf	
x = 3	h_x = **29.01** ft	0.98	17.19	11.69 psf	19.98 psf	29 kips
	33.85 ft	1.01	17.76	12.08 psf	20.36 psf	
x = 4	h_x = **38.68** ft	1.04	18.26	12.42 psf	20.71 psf	31 kips
	43.68 ft	1.06	18.74	12.74 psf	21.03 psf	
x = 5	h_x = **48.68** ft	1.09	19.17	13.04 psf	21.32 psf	16 kips
	0.00 ft	0.85	14.96	0.00 psf	0.00 psf	
x = 6	h_x = **0.00** ft	0.85	14.96	0.00 psf	0.00 psf	0 kips
	0.00 ft	0.85	14.96	0.00 psf	0.00 psf	
x = 0	h_x = **0.00** ft	0.85	14.96	0.00 psf	0.00 psf	0 kips
	0.00 ft	0.85	14.96	0.00 psf	0.00 psf	
x = 0	h_x = **0.00** ft	0.85	14.96	0.00 psf	0.00 psf	0 kips
*Note: B > L produces maximun base shear.					V = Sum (F_i) =	131 kips



Roof	
$p_R = q_h G C_{pR}$	
(Windward) C_{pWR} =	**-0.91**
(Leeward) C_{pLR} =	**-0.18**
Windward p_{WR} =	-15.1 psf
Leeward p_{LR} =	-3.0 psf

Figure 16: Wind Load Calculations for Staggered Truss System, EW Direction

IBC 2003 (ASCE 7-02)

Wind Analysis, Main Windforce Resisting System, Analytical Procedure, NS Direction

h =	**52.7** ft
No. Stories = n =	**6**
L =	**150.80** ft
B =	**91.00** ft
L / B =	1.66
V =	**90.0** mph
K_d =	**0.85**
I =	**1.00**
Exposure Category	**C**
Enclosure Classification	**E**
Exposure Case	Case 1
□ =	9.5
z_g =	900 ft
K_1 =	**0**
K_2 =	**0**
K_3 =	**0**
K_{zt} =	1
T =	**0.6107** sec
f =	1.64 hz
G =	0.85

Structure is rigid. G = 0.85.

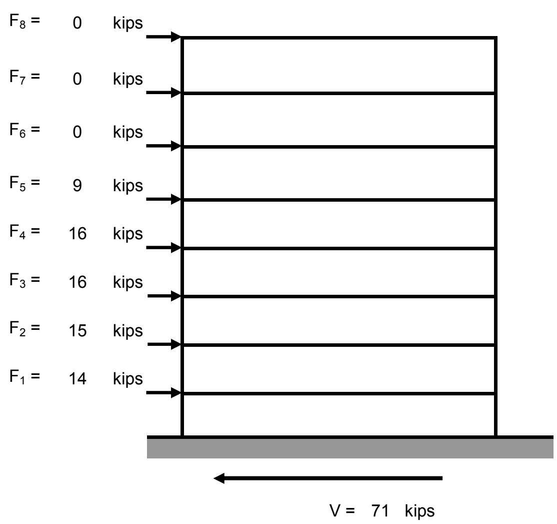
$p_W = q_z G C_{pW}$ $q_z = 0.00256 K_z K_{zt} K_d V^2 I$ K_h = 1.11

$p_L = q_h G C_{pL}$ (Windward) C_{pW} = 0.8 q_h = 19.5 psf

$p_{TOT} = p_W - p_L$ (Leeward) C_{pL} = **-0.37** p_L = -6.13 psf

	4.84 ft	K_z = 0.85	q_z = 14.96	p_W = 10.17 psf	p_{TOT} = 16.31 psf	
x = 1	h_x = **9.67** ft	K_z = 0.85	q_z = 14.96	p_W = 10.17 psf	p_{TOT} = 16.31 psf	F_x = 14 kips
	14.50 ft	K_z = 0.85	q_z = 14.96	p_W = 10.17 psf	p_{TOT} = 16.31 psf	
x = 2	h_x = **19.33** ft	K_z = 0.90	q_z = 15.78	p_W = 10.73 psf	p_{TOT} = 16.86 psf	F_x = 15 kips
	24.33 ft	K_z = 0.94	q_z = 16.57	p_W = 11.26 psf	p_{TOT} = 17.40 psf	
x = 3	h_x = **29.33** ft	K_z = 0.98	q_z = 17.23	p_W = 11.72 psf	p_{TOT} = 17.85 psf	F_x = 16 kips
	34.00 ft	K_z = 1.01	q_z = 17.78	p_W = 12.09 psf	p_{TOT} = 18.22 psf	
x = 4	h_x = **38.67** ft	K_z = 1.04	q_z = 18.26	p_W = 12.42 psf	p_{TOT} = 18.55 psf	F_x = 16 kips
	43.67 ft	K_z = 1.06	q_z = 18.74	p_W = 12.74 psf	p_{TOT} = 18.87 psf	
x = 5	h_x = **48.67** ft	K_z = 1.09	q_z = 19.17	p_W = 13.04 psf	p_{TOT} = 19.17 psf	F_x = 9 kips
	0.00 ft	K_z = 0.85	q_z = 14.96	p_W = 0.00 psf	p_{TOT} = 0.00 psf	
x = 6	h_x = **0.00** ft	K_z = 0.85	q_z = 14.96	p_W = 0.00 psf	p_{TOT} = 0.00 psf	F_x = 0 kips
	0.00 ft	K_z = 0.85	q_z = 14.96	p_W = 0.00 psf	p_{TOT} = 0.00 psf	
x = 0	h_x = **0.00** ft	K_z = 0.85	q_z = 14.96	p_W = 0.00 psf	p_{TOT} = 0.00 psf	F_x = 0 kips
	0.00 ft	K_z = 0.85	q_z = 14.96	p_W = 0.00 psf	p_{TOT} = 0.00 psf	
x = 0	h_x = **0.00** ft	K_z = 0.85	q_z = 14.96	p_W = 0.00 psf	p_{TOT} = 0.00 psf	F_x = 0 kips

*Note: B > L produces maximun base shear. V = Sum (F_i) = 71 kips



Roof		
$p_R = q_h G C_{pR}$		
(Windward) C_{pWR} =	**0.51**	
(Leeward) C_{pLR} =	**-0.6**	
Windward p_{WR} =	8.5	psf
Leeward p_{LR} =	-9.9	psf

Figure 17: Wind Load Calculations for Staggered Truss System, NS Direction

Comparison

Seismic analysis yielded a base shear of 462 k. Although seismic controls design base shear, load combinations using wind will be included in the staggered truss analysis to ensure the system can resist all the various loading combinations outlined in ASCE 7-02. A summary of the wind and seismic forces is provided in Table 3.

Wind and Seismic Forces (All loads are service loads)						
	Wind (on wide direction)			Seismic (both directions)		
	Lateral Load	Story Shear	ϕ_h	Lateral Load	Story Shear	ϕ_h
Floor	Vj (k)	Vw (k)	(%)	Vj (k)	Vw (k)	(%)
Roof	16.00	16.00	12.21	128.20	128.20	27.72
5	31.00	47.00	35.88	136.40	264.60	57.21
4	29.00	76.00	58.02	100.70	365.30	78.98
3	28.00	104.00	79.39	65.60	430.90	93.17
2	27.00	131.00	100.00	31.60	462.50	100.00
Ground						

Table 3: Wind and Seismic Forces for Staggered Truss

Diaphragm Design

Diaphragm design in staggered truss systems is more critical than many other hollow-core slabs because it must transmit the lateral load from truss to truss. While many design elements can be left to the supplier, it is important for the engineer to relay all information as to the magnitude and location of forces that the diaphragm is required to transmit. As previously stated, it is assumed that the floor will act as a rigid diaphragm, even though it is untopped, because it would be specified that all shear keys are to be grouted solid. This meets the requirements of PCI 3.6.2, making a rigid diaphragm a valid assumption.

Since the building is almost symmetrical, it is assumed that the center of mass is in the center of the building footprint. The center of rigidity was calculated separately for even and odd floors. All trusses were assumed to have equal rigidity for these calculations. It was determined that the center of rigidities were $x_o = 60$ft and $x_e = 90$ft. Load eccentricity was then calculated and 5% eccentricity for accidental torsion added.
This resulted in the following:

$$e_e = 15 \pm 7.5\,ft$$
$$e_o = -15 \pm 7.5\,ft$$

The base torsion was calculated as base shear multiplied by eccentricity, using the base shear of 462.5k from Figure 14.

T=	462.5	*15=	6938	ft-k
T=	462.5	*7.5=	3469	ft-k
Vs=	462.5	/3=	154	k

Torsional rigidity was calculated in order to determine the torsional shear component. It is then added or subtracted from the translational shear component. The results are summarized in the following table.

Shear Force in Each Truss due to Lateral Loads (Bottom Floor)								
	x_i (ft)	Vs	T (ft-k) = 6938		T (ft-k) = 3469		Design Shear	
Truss			V_{tors}	V_i	V_{tors}	V_i	V_i (k)	ϕ_{ecc}
T1E.9	-60	154	-57.8125	96	-28.9063	125	125	1.00
T1D	0	154	0	154	0	154	154	1.23
T1B	60	154	57.8125	212	28.90625	183	212	1.70
T2E	-60	154	57.8125	212	28.90625	183	212	1.00
T2C	0	154	0	154	0	154	154	1.23
T2A	60	154	-57.8125	96	-28.9063	125	125	1.70

Table 4: Shear Force in Each Truss due to Lateral Loads

In order to complete the diaphragm design, transverse shear must be considered. In order to have this type of lateral system work, the diaphragm must act as a deep beam. In this configuration, the trusses above the diaphragm act as drag struts to engage the entire length of the diaphragm in transferring shear to the trusses below. The required shear strength of the diaphragm is calculated as follows:

$Vu = 1.7 \times \Phi h \times V \times 0.75 = 159.38k$

where V=125k is the maximum shear force in the diaphragm, Φh is from Table 3, and the 0.75 is applied for wind or seismic loads. The provided shear strength is calculated based on ACI 318:

ϕVn=	ϕ(Vc+Vs)			
Assuming	effective thickness of 4.8"			
	effective depth 80% of total depth			
	f'c=5000 psi			
ϕVc=	ϕ*2√f'c bd			
ϕVc=	386.44 k			
ϕVs=	ϕ*A_{VF}fyμ			
	No. of planks =	78'/8'=	10	planks
	No. of joints =	10-1=	9	joints
	A_{VF} =	0.2*9=	1.8	in^2
ϕVs=	128.52 k			
ϕVn=	514.96 k			

Since ΦVn>Vu, the diaphragm is acceptable with the outlined assumptions. Perimeter steel beams are designed to act as diaphragm chords in staggered truss systems. The chord forces can be approximated using the equation H=M/D, where M is the moment applied and D is the depth of diaphragm. A constant 0.75 was applied to the equation results for seismic loading. In addition to chord forces, shear flows were also calculated.

A summary of the diaphragm chord forces and required shear flows follows in Table 5. In order to ensure that the shear in the diaphragm transfers to the truss, shear studs along the truss chords are included. They were added to the design using the industry rule of thumb which is one stud per foot minimum.

	ϕ_h (%)	H(kips)		f_H (k/ft)	
		H1	H2	a	b
Roof	27.72	13.65	17.77	0.20	0.30
5	57.21	28.17	36.67	0.41	0.61
4	78.98	38.88	50.63	0.56	0.84
3	93.17	45.87	59.72	0.66	1.00
2	100.00	49.23	64.10	0.71	1.07
Ground					

Table 5: Diaphragm Chord Forces and Design Shear Flow

Truss Member Design

In order to design the truss members, the following material properties were assumed:

Material Guide			
	Section	ASTM	Fy (ksi)
Columns and Chords	W-Shape	A992	50
Web Members	HSS	A550 Grade B	46 (rectangular)
Gusset Plates	Plate	A572	50

Table 6: Material Properties Guide

Since the building is symmetrical, hand calculations are used to obtain preliminary sizes of members, which are then verified in a 3-D model. This approach is logical since, for symmetrical buildings, 2-D analysis and design is considered sufficient.

<u>Gravity Loads</u>

The gravity loading for a single story truss was calculated by converting the uniform full service gravity load to point loads at each joint. Under this loading, the shear forces are neglected so the truss becomes statically determinate. Method of joints was then used to calculate member forces, and a RAM Advanse model was created to verify the results. Figure 18 shows a summary of the axial forces on the truss at the bottom floor.

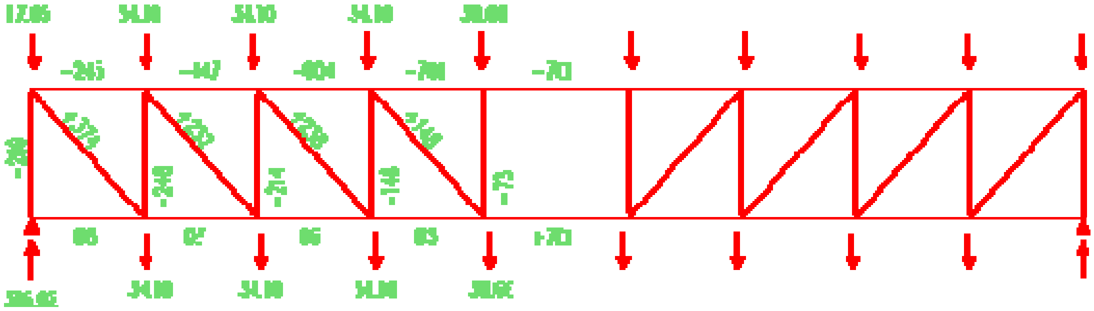


Figure 18: Axial Forces in Members due to Gravity Loads (k)

Lateral Loads

Lateral loads are distributed to the individual trusses based on relative stiffness and plan location. Figure 19 summarizes the member forces in truss T1E.9 at the bottom floor, using a design shear of 125k (see Table 3). Shear and moment were calculated as follows:

For the Vierendeel panel
V=1/2(62.5k) (2) (9.67')/39' = 7.75k
M=7.75k (10.67')/2 = 41.3ftk

For the adjacent panel
M=41.3ftk
V= (41.3'k+0)/ (8.42') =4.91k

Axial forces for the members were determined using method of joints and verified through a RAM Advanse model. The resulting forces are summarized in the figure below.

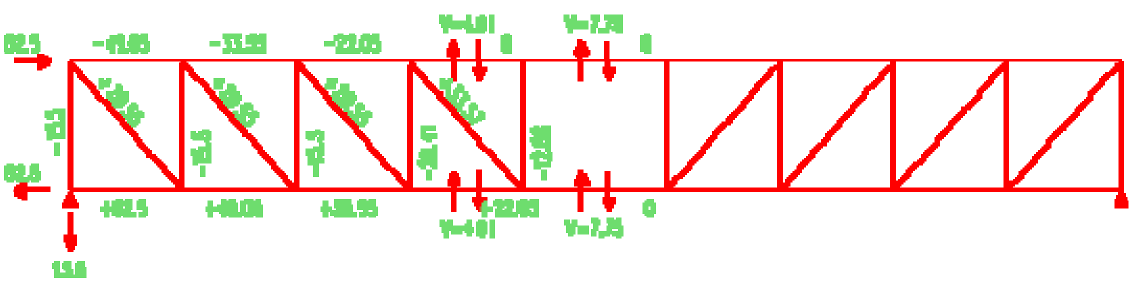


Figure 19: Axial Forces in Members due to Lateral Loads (k)

Vertical and Diagonal Members

Sizing of vertical and diagonal members was done using the five load combinations specified in ASCE 7-02. A 50% reduction in live load was used in this design. The worst case diagonal, with a governing tensile axial force of +374k under gravity loading and +20.55k under lateral loads, was used to size all members. The value used for Φ_{ecc} in this calculation was the maximum, 1.70 (see Table 3). The results of this calculation are summarized in Table 7.

	Wind (k)		Seismic (k)		Load Combinations (k)					
Floor	Φh	ΦeccΦhFw	Φh	ΦeccΦhFe	1	2	3	4	5	Member Sizes
Roof	12.21	0.87	27.72	7.01	368.46	**404.47**	344.66	350.53	Does	HSS 8x6x5/8
5	35.88	2.57	57.21	14.46	368.46	**404.47**	346.87	357.99	not	HSS 8x6x5/8
4	58.02	4.15	78.98	19.96	368.46	**404.47**	348.93	363.49	govern	HSS 8x6x5/8
3	79.39	5.68	93.17	23.55	368.46	**404.47**	350.91	367.08		HSS 8x6x5/8
2	100.00	7.16	100.00	25.28	368.46	**404.47**	352.83	368.80		HSS 8x6x5/8
Ground										

Table 7: Design of Diagonal Member D1 on T1B

Since this is the governing loading condition, all diagonal members in the trusses are sized according to these calculations. However, the 3D RAM model indicated that the size of ground floor braces should be increased to HSS 8x8x5/8. This change incorporated in the overall design.

Chord Members

Chord members were designed to comply with AISC Equation H1-1a for combined loading. Gravity and wind loads were calculated separately initially, and then combined to determine the design loads. Chords were limited to W10 sizes, in order to stay below the maximum structural depth of 1-8". Table 8 shows the governing chord truss load case.

Truss	T1B	
w=	4.05	k/ft
Panel width	9.67	ft
M=	37.87	ftk
P,grav=	701	k
Mu, grav=	34.78531	ftk
Pu,grav=	643.8815	k

Floor	Φh	M	Mu,lat	Mu	Pu	Section	AISC Eq H1-1a
Roof	27.72	59	76.39	111.18	643.88	W10x77	0.9643
5	57.21	121	157.67	192.46	643.88	W10x88	0.9889
4	78.98	167	217.68	252.46	643.88	W10x100	0.9751
3	93.17	198	256.77	291.56	643.88	W10x112	
2	100.00	212	275.60	310.39	643.88	W10x112	0.9607
Ground							

Table 8: Design of Chord Members on Truss T1B

Connection Design

For this design, a slotted HSS to gusset plate connection was used to connect the vertical and diagonal truss members to the chords. The design methodology follows the recommendations of the AISC Hollow Structural Sections Connections Manual. Four failure cases were considered:

- Shear lag fracture strength in HSS
- Shear strength of HSS at weld
- Strength of the weld connecting the gusset plate to HSS
- Shear strength of gusset plate

The typical connection detail is included in Figure 20.

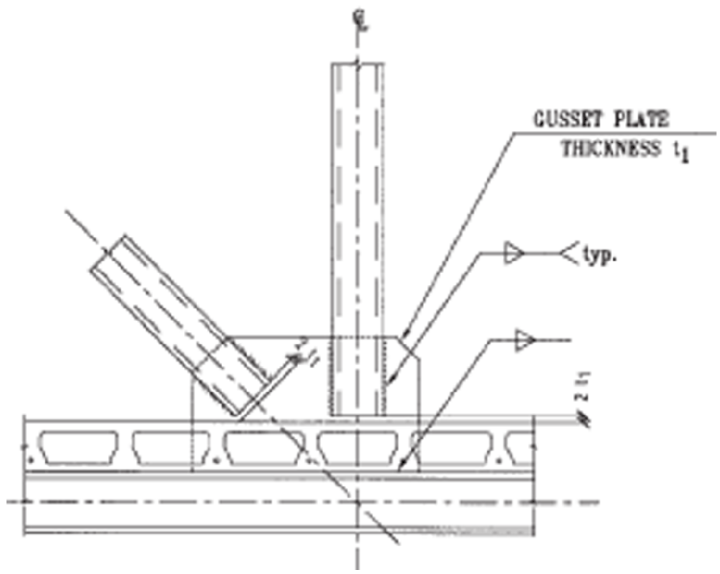


Figure 20: Typical Detail of Slotted HSS and Gusset Plate Connection

It was assumed for the calculations that the length of weld was 20”, F_{EXX} = 70ksi, weld width is 3/8”, and plate thickness is ½”. The governing strength condition was shear lag fracture, where ΦRn = 488k. This is greater than Pu = 457k, the governing condition from Table 5. No further investigation into framing connections was conducted.

Foundation Design

There is a significant impact on foundation design as a result of switching from a shear wall system to a steel one. Instead of using strip footings under each load-bearing wall, spread footings can be used. This significantly can reduce the amount of concrete needed to complete the design. According to the geotechnical reports, a bearing capacity of 5 ksf can be assumed for design. Additionally, all exterior footings need to be located at least 3’ below finished grade, while interior footings should be at least 2’ below finish grade. For the gravity columns, a 4’x4’x1’ spread footing with (4) #5 bars each way could be used. A 7’x7’x1’10” spread footing with (6) #7 bars each way would be sufficient for the truss columns.

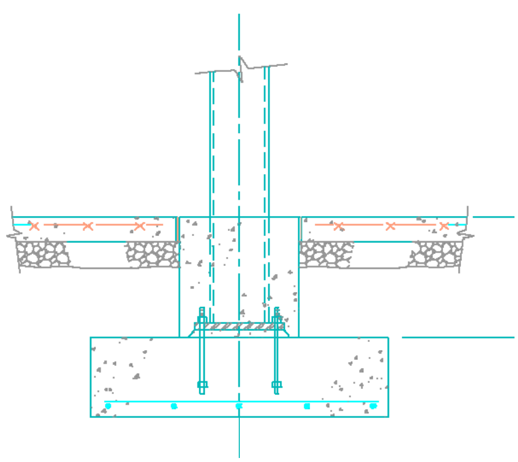


Figure 21: Typical Detail of Steel Column Footing

Overall Design Summary

To complete the analysis of the proposed staggered truss system, the procedures written in AISC Design Guide 8 were followed. This resulted in a 2-D analysis of the structure. This lateral redesign of Whiteland Village was then verified and completed using a RAM Structural System model. Using this model, it was determined that lateral force-resisting columns are W10x100, while beams in the moment frames are W10x68. Drift was limited to H/400, an industry standard, with a maximum drift of 1.47".

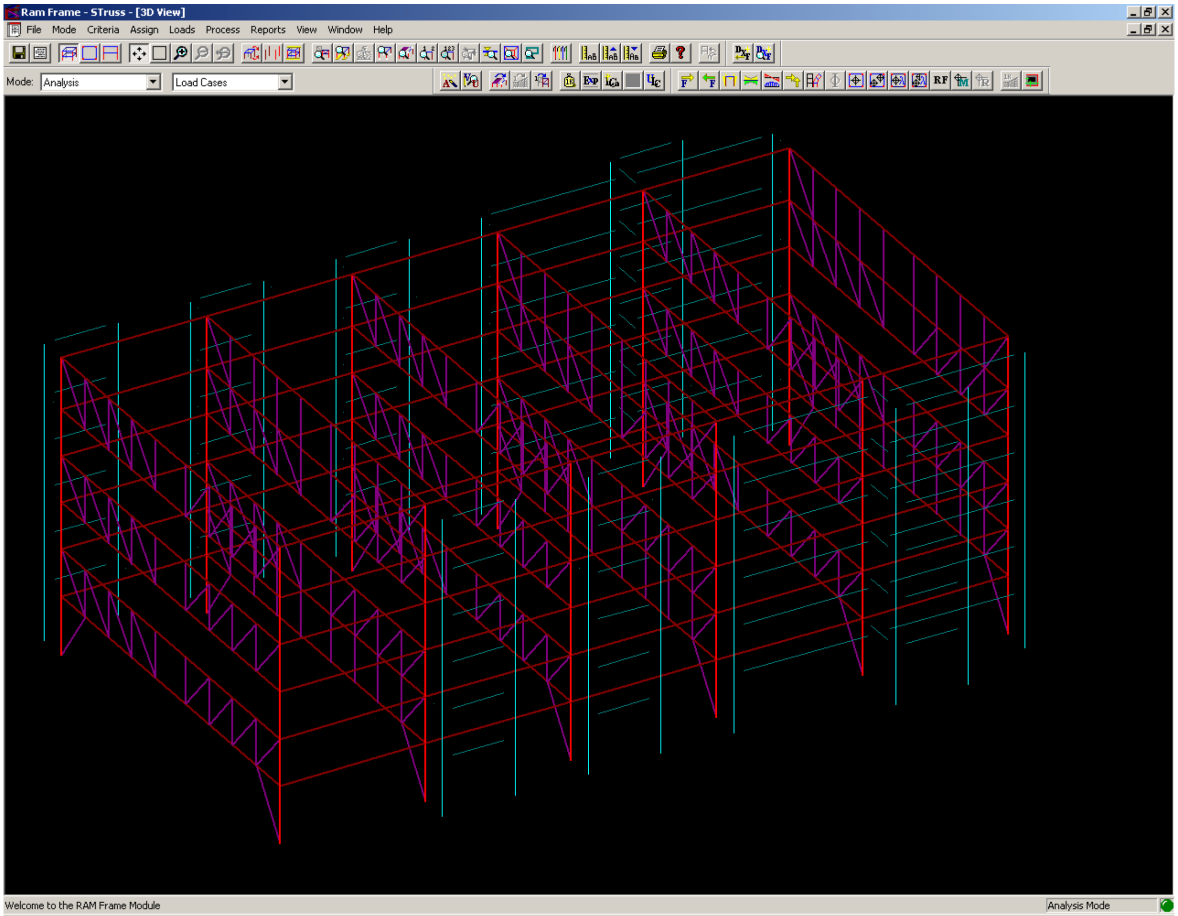


Figure 22: RAM Model of Staggered Truss System

Staggered trusses span 78' across the short dimension of the building to resist lateral loads. Resistance in the long direction is provided by two 150' moment frames. Truss chords vary in size from W10x77 to W10x112. TheW10 sizes were determined by the need to keep the structural depth as shallow as possible. W10 is also the typical shape for truss chords, as noted in AISC Design Guide 14.

Composite studs were added to the connection of the plank and truss chords in order to ensure that the shear fully transferred from the planks into the staggered truss. The number of studs required was approximated by using the industry rule of thumb of one shear stud per foot. A graphical summary of the staggered truss design is included in Figures 23, 24, and 25. Footings for the W10x100 columns are 7'x7'x1'10" spread footing with (6) #7 bars each way.

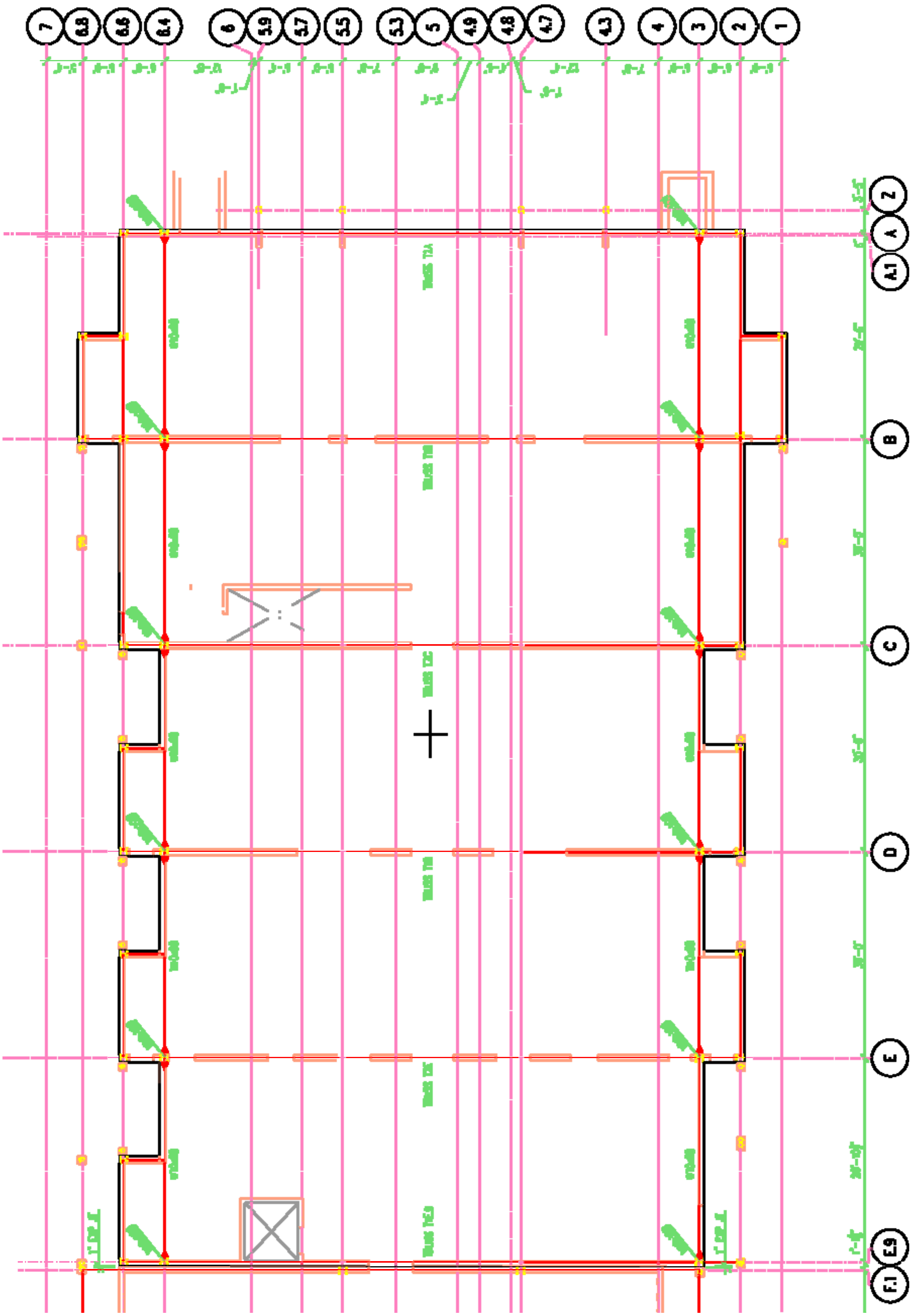


Figure 23: Floor Plan of Staggered Truss System

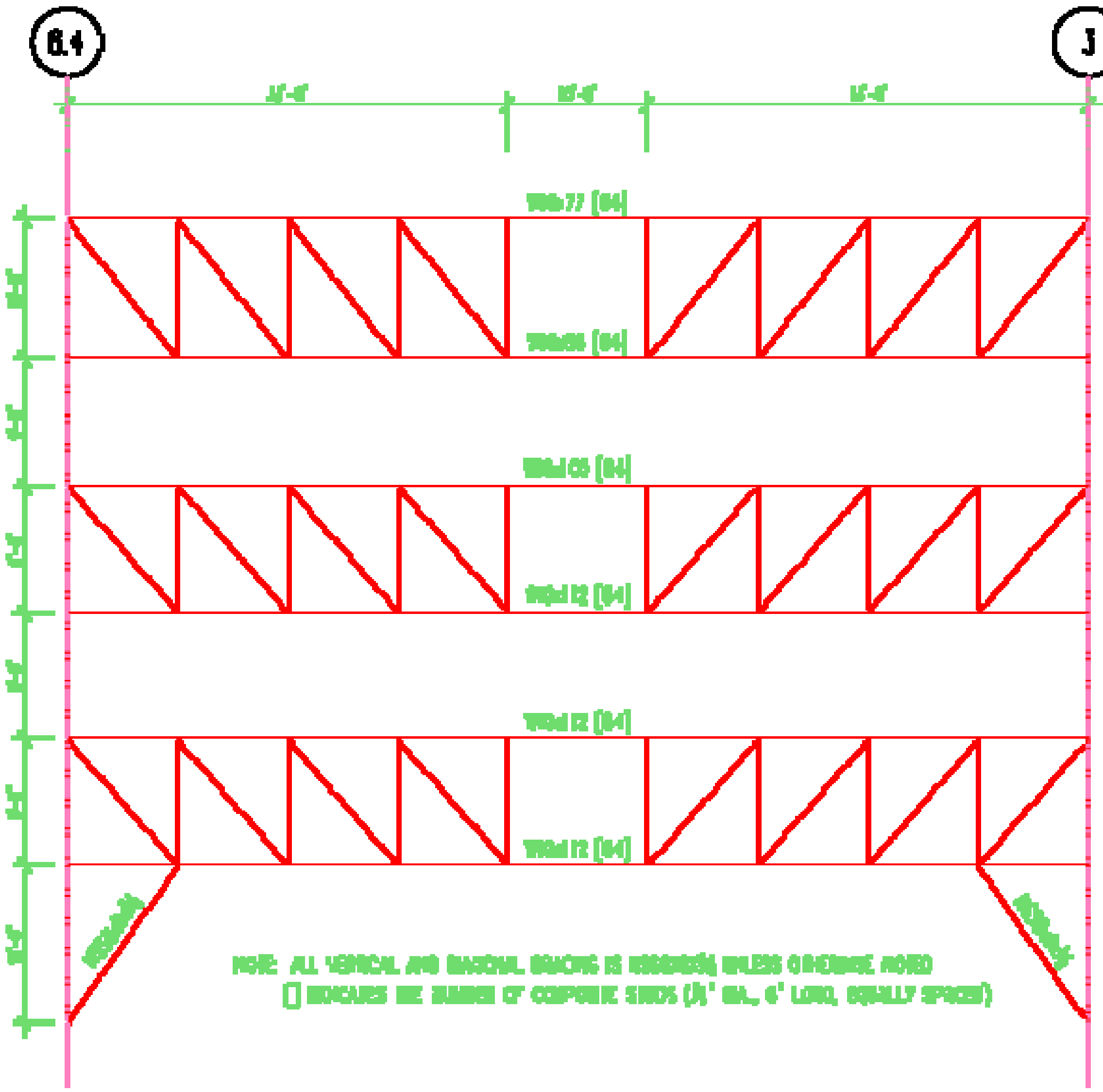


Figure 24: Elevation of Truss T1

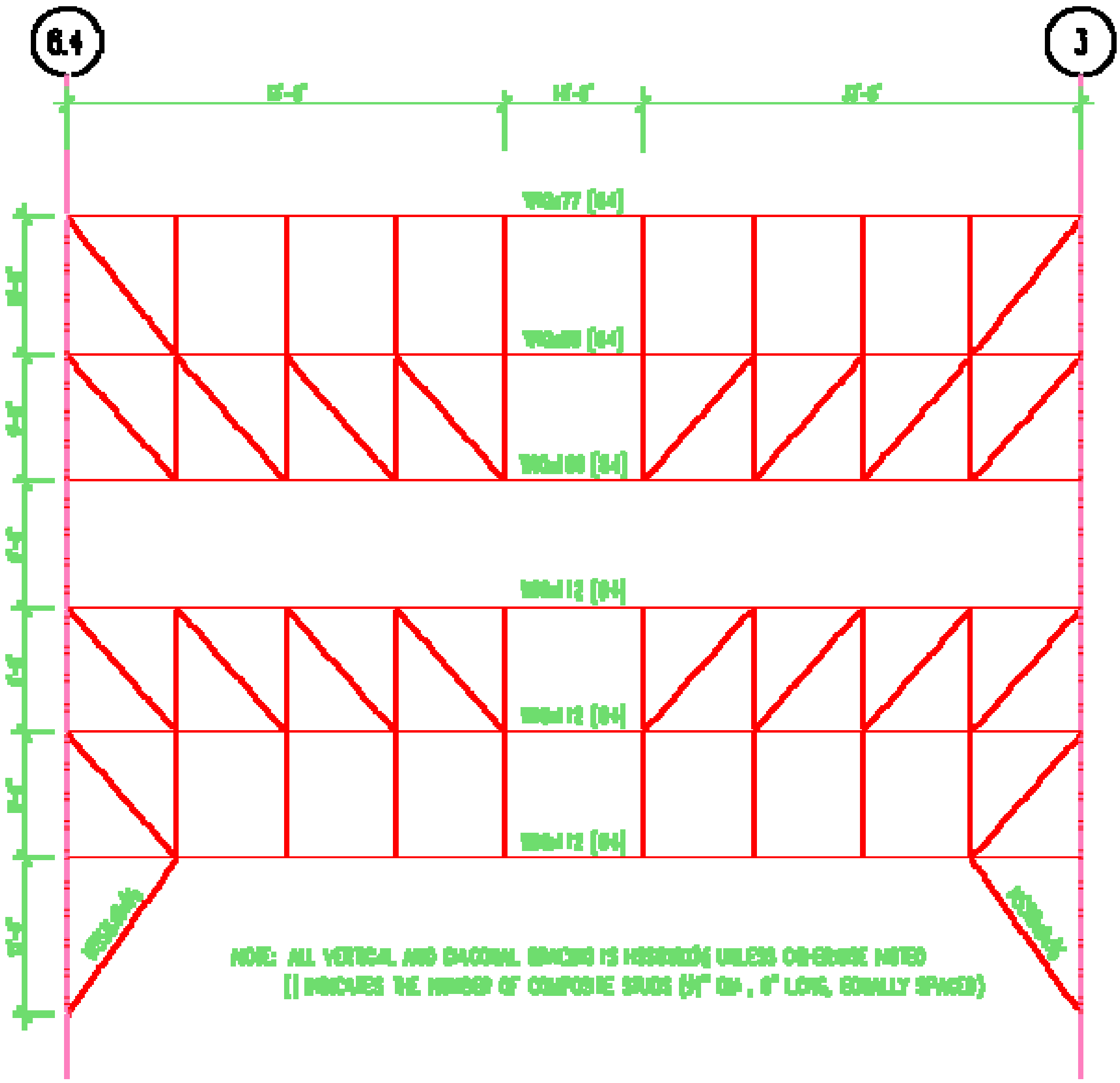


Figure 25: Elevation of Truss T2

Conclusions

In order to meet the height requirements of local zoning, the maximum allowable structural depth was 1'-8". This was met by limiting truss chords and moment frame beams to W10 shapes. Residence plan dimensions were not greatly impacted by the switch to a steel system. Originally, the shear walls were 10" CMU. With the orientation of the columns, the flange width would determine wall width between units. Since the flange width is 10.3", a minimal reduction in available square footage would occur. Using this system would also require the additional time and expense of fireproofing.

In order for this system to work effectively, however, the engineer would have needed to work closely with the architect to change the condominium layouts to fit the proposed truss configuration. This may have been a viable situation since the project delivery method was design-build.

Partially Restrained Composite Connection Alternate

The braced frame system utilizing partially restrained composite connections (PRCCs) was designed using the Load and Resistance Factor Design (LRFD) Method and the procedures outlined in *AISC Steel Design Guide 8: Partially Restrained Composite Connections*.

Framing Layout

In order to analyze a section of Whiteland Village for the use of PRCCs, a steel framing layout needed to be assumed. In the NS direction, four frames with inverted chevron bracing were located on column lines E, D, C, and B. On column lines 4.7 and 4.5, braced frames were also created using inverted chevrons. This configuration was chosen because it allowed for door openings on either side of the column, where doors are located on the existing floor plan. In addition, the traffic lane in the garage level in unaffected by the switch. Exterior bays are 30'x32' and interior bays are sized at 30'x26'. Figure 26 shows the plan for this framing scenario, while elevations for the NS and EW braced frames are shown in Figures 27 and 28, respectively.

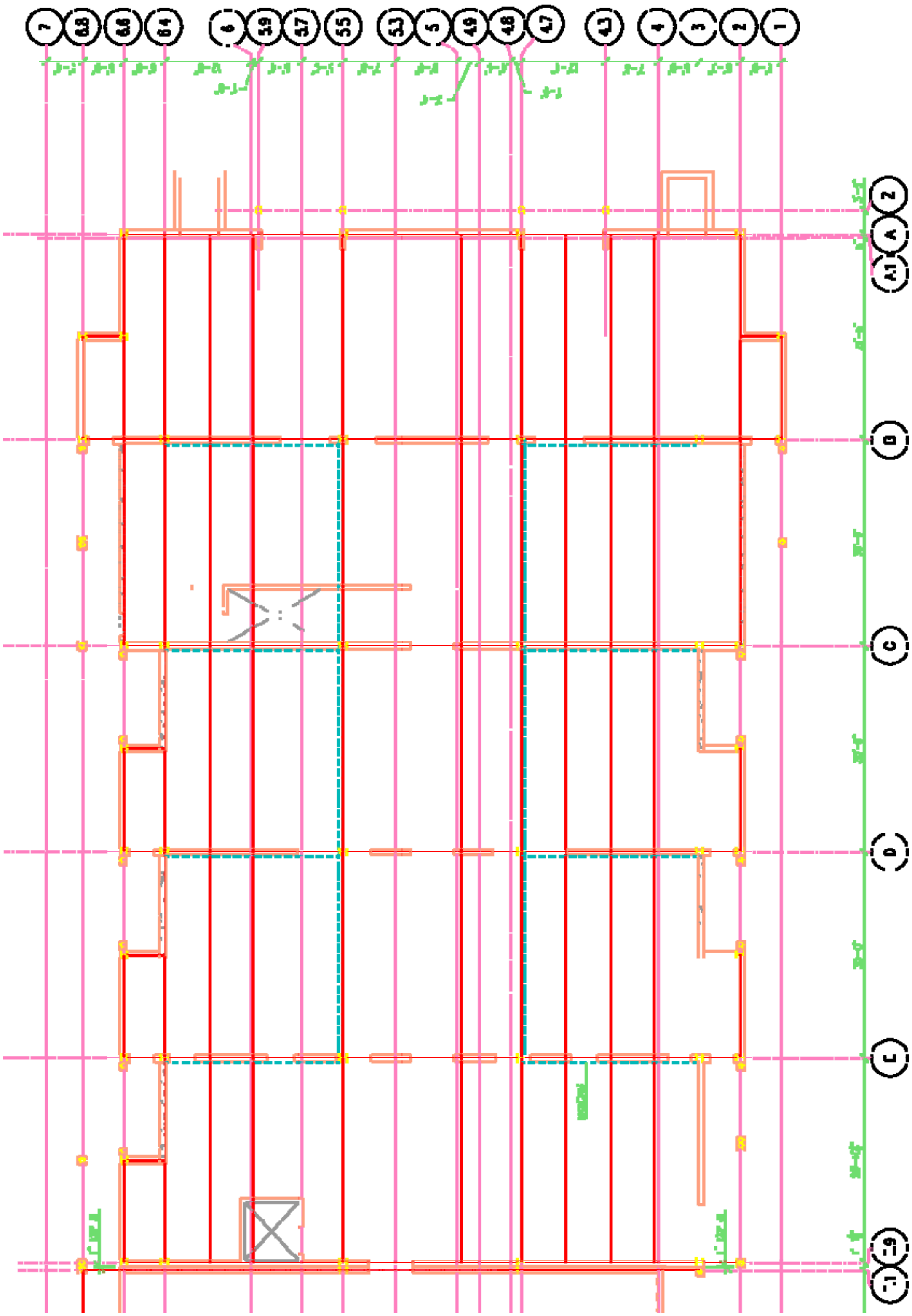


Figure 26: Structural Plan for Proposed PRCC System

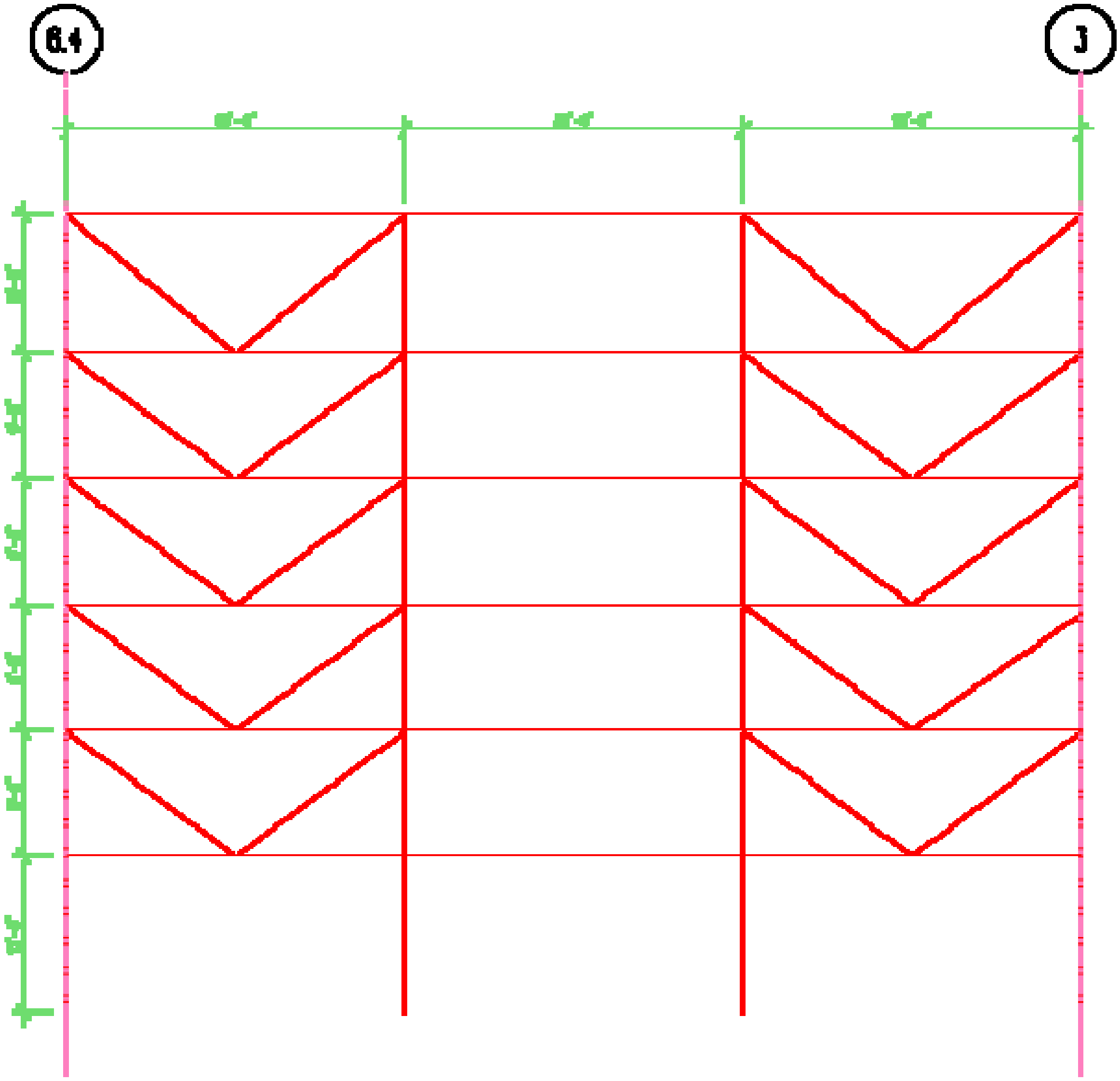


Figure 27: PRCC Braced Frame – NS Direction

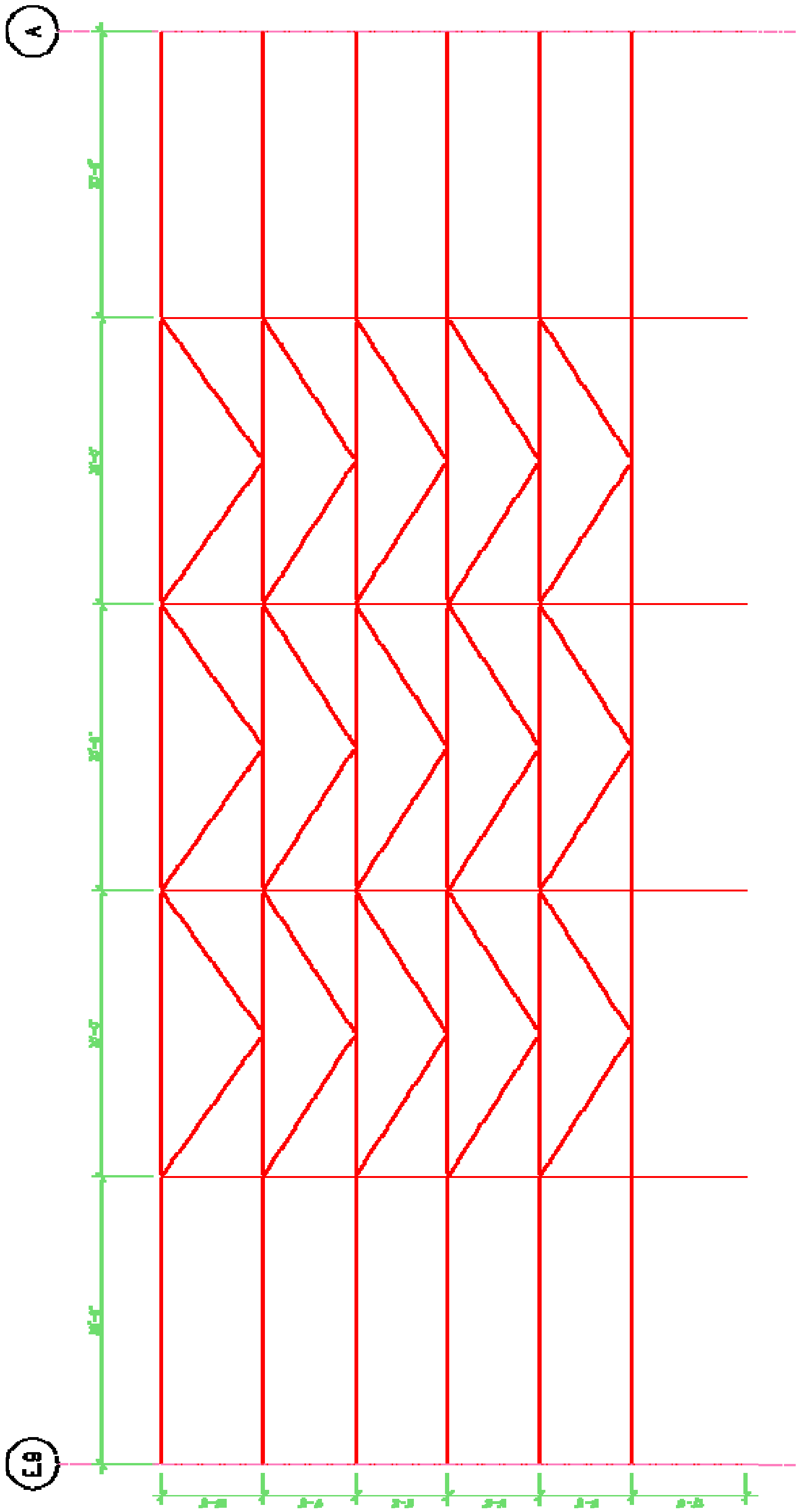


Figure 28: PRCC Braced Frame – EW Direction

Floor Vibration Analysis

In addition to choosing an appropriate deck and concrete slab thickness combination for the composite floor system based on strength, floor vibration was also considered. Walking vibration is a very real concern for an upscale retirement community. Since the layout of the building requires the corridor to be in the middle of a central bay, vibration issues should be considered. The preliminary procedure for design outlined in *Floor Vibration Serviceability: Tips and Tools for Negotiating a Successful Design*, was followed to find a floor that should be acceptable. Based on the two typical bay sizes of 30'x26' and 30'x32', it was determined that an appropriate floor would require 5.5" total slab thickness. Therefore, for these calculations, a 1 ½" VLI composite deck with 5 ½" total slab thickness, weighing 57 PSF, was assumed.

Gravity Loads

Gravity loads were determined in conjunction with ASCE 7-02 and manufacturer data. A 5 ½" composite slab was used to calculate floor loads, while a 4" composite slab was assumed for the roof.

<u>Dead Loads</u>

Before Composite Action (DL_B)

- 5 ½" Composite Slab – 57 PSF
- Structural Steel – 5 PSF

Total – 62 PSF

After Composite Action (DL_A)

- HVAC – 5 PSF
- Ceiling – 2 PSF
- Misc. – 8 PSF

Total – 15 PSF

Roof

- HVAC – 10 PSF
- Ceiling – 2 PSF
- Roofing and Insulation – 10 PSF
- Misc. – 3 PSF

Total – 25 PSF

Wall

- Manufact. Stone – 12 PSF
- Studs – 3 PSF
- Sheetrock – 3 PSF
- Insulation – 2 PSF

Total – 20 PSF

<u>Live Loads</u>

Floor – 40 PSF
Roof – 30 PSF

Determination of Controlling Lateral Load

Lateral loads were determined in accordance with ASCE 7-02.

Seismic

Seismic base shear was determined using Method 3: Equivalent Lateral Force (ELF). S_s and S_1 were determined using the USGS Hazards Program, inputting Lat 40.5^o and Long 75.6^o. Conservatively, a seismic response modification factor (R) of 3 was used for ordinary composite braced frames. In order to reduce base shear, the period was also assumed to be T=Cu*Ta.

Floor Area=	13398	sq.ft. per floor
Wall Length=	570	ft
Story Height=	9.67	ft

	Floor DL	Floor Wt.	Wall DL	Wall Wt.	Wx	Hx
	(PSF)	(k)	(PSF)	(k)	(k)	(ft)
Roof	54	723.49			723.49	49.35
5	92	1232.62	20	110.24	1342.85	38.68
4	92	1232.62	20	110.24	1342.85	29.01
3	92	1232.62	20	110.24	1342.85	19.34
2	92	1232.62	20	110.24	1342.85	9.67
1	92	1232.62	20	110.24	1342.85	0.00
Sum					7437.76	

Table 8: Building Weight Determination for Seismic Analysis, PRCC System

IBC 2003 (ASCE 7-02)

Seismic Analysis, Equivalent Lateral Force

Seismic Design Category	B
S_1 =	0.081 g
S_{MS} =	0.4614 g
S_{M1} =	0.1944 g
S_{DS} =	0.3076 g
S_{D1} =	0.1296 g
I_E =	1
No. Stories = N =	5
C_u =	1.64
R =	3
h_n =	49.35 ft
C_t =	0.02
x =	0.75
$T_a = C_t (h_n)^x$ =	0.3724 sec
$T_{max} = C_u T_a$ =	0.611 sec
T =	0.611 sec
$C_s = S_{DS} / (R / I_E)$ =	0.1025
$C_{s\,max} = S_{D1} / (R / I_E) / T$ =	0.0707
$C_{s\,min} = 0.044\, S_{DS}\, I_E$ =	0.0135
$C_{s\,min} = 0.5\, S_1 / (R / I_E)$ =	N/A
C_s =	0.0707

$F_x = C_{vx} V$ k = 1.06

$C_{vx} = w_x h_x^k / \text{Sum}(w_i h_i^k)$ (from i to n)

x = 1	w_x = 1343 kips	h_x = 9.67 ft	$w_x h_x^k$ = 14730	C_{vx} = 0.0735	F_x = 31.7 kips
x = 2	w_x = 1343 kips	h_x = 19.34 ft	$w_x h_x^k$ = 30615	C_{vx} = 0.1528	F_x = 65.9 kips
x = 3	w_x = 1343 kips	h_x = 29.01 ft	$w_x h_x^k$ = 46967	C_{vx} = 0.2345	F_x = 101.1 kips
x = 4	w_x = 1343 kips	h_x = 38.68 ft	$w_x h_x^k$ = 63631	C_{vx} = 0.3177	F_x = 136.9 kips
x = 5	w_x = 724 kips	h_x = 49.35 ft	$w_x h_x^k$ = 44361	C_{vx} = 0.2215	F_x = 95.5 kips
x = 0	w_x = 0 kips	h_x = 0 ft	$w_x h_x^k$ = 0	C_{vx} = 0	F_x = 0.0 kips
x = 0	w_x = 0 kips	h_x = 0 ft	$w_x h_x^k$ = 0	C_{vx} = 0	F_x = 0.0 kips
x = 0	w_x = 0 kips	h_x = 0 ft	$w_x h_x^k$ = 0	C_{vx} = 0	F_x = 0.0 kips
			Sum $(w_i h_i^k)$ = 200304	V = Sum (F_i) =	431 kips
				Check:	OK

Structure Weight	W = Sum (w_i) =	6096 kips
Base Shear	V = Cs W =	431 kips

Per ASCE table 9.5.5.3.1					
S_{D1}	>=0.4	0.3	0.2	0.15	<=0.1
C_u	1.4	1.4	1.5	1.6	1.7

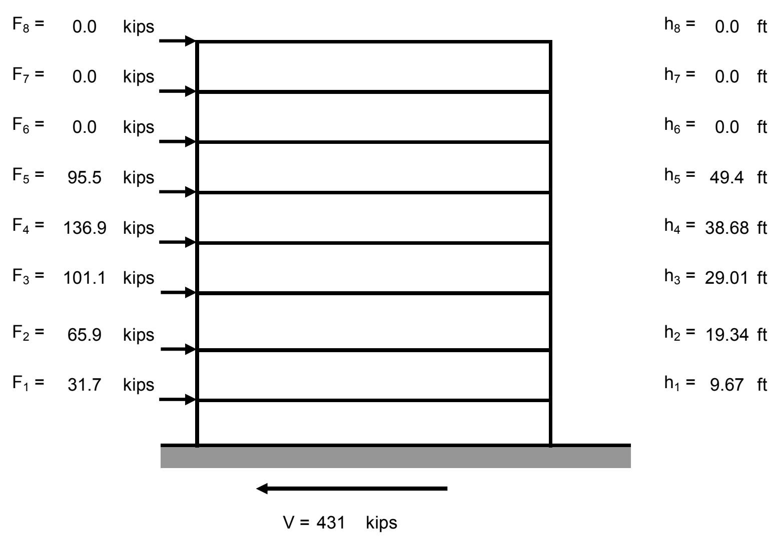


Figure 30: Seismic Load Calculations for PRCC System

Wind

IBC 2003 (ASCE 7-02)

Wind Analysis, Main Windforce Resisting System, Analytical Procedure, EW Direction

h =	**52.7** ft
No. Stories = n =	**6**
L =	**91.00** ft
B =	**150.80** ft
L / B =	0.60
V =	**90.0** mph
K_d =	**0.85**
I =	**1.00**
Exposure Category	**C**
Enclosure Classification	**E**
Exposure Case	Case 1
□ =	9.5
z_g =	900 ft
K_1 =	**0**
K_2 =	**0**
K_3 =	**0**
K_{zt} =	1
T =	**0.6110** sec
f =	1.64 hz
G =	0.85

Structure is rigid. G = 0.85.

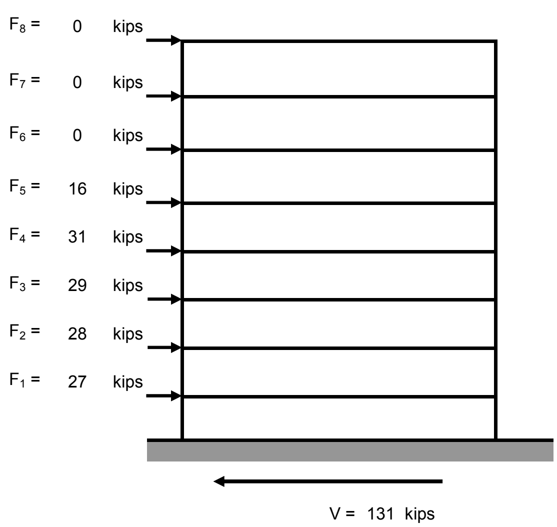
$p_W = q_z G C_{pW}$ $q_z = 0.00256 K_z K_{zt} K_d V^2 I$ K_h = 1.11

$p_L = q_h G C_{pL}$ (Windward) C_{pW} = 0.8 q_h = 19.5 psf

$p_{TOT} = p_W - p_L$ (Leeward) C_{pL} = **-0.5** p_L = -8.29 psf

	4.84 ft	K_z = 0.85	q_z = 14.96	p_W = 10.17 psf	p_{TOT} = 18.46 psf	
x = 1	h_x = **9.67** ft	K_z = 0.85	q_z = 14.96	p_W = 10.17 psf	p_{TOT} = 18.46 psf	F_x = 27 kips
	14.51 ft	K_z = 0.85	q_z = 14.96	p_W = 10.17 psf	p_{TOT} = 18.46 psf	
x = 2	h_x = **19.34** ft	K_z = 0.90	q_z = 15.78	p_W = 10.73 psf	p_{TOT} = 19.02 psf	F_x = 28 kips
	24.18 ft	K_z = 0.94	q_z = 16.54	p_W = 11.25 psf	p_{TOT} = 19.54 psf	
x = 3	h_x = **29.01** ft	K_z = 0.98	q_z = 17.19	p_W = 11.69 psf	p_{TOT} = 19.98 psf	F_x = 29 kips
	33.85 ft	K_z = 1.01	q_z = 17.76	p_W = 12.08 psf	p_{TOT} = 20.36 psf	
x = 4	h_x = **38.68** ft	K_z = 1.04	q_z = 18.26	p_W = 12.42 psf	p_{TOT} = 20.71 psf	F_x = 31 kips
	43.68 ft	K_z = 1.06	q_z = 18.74	p_W = 12.74 psf	p_{TOT} = 21.03 psf	
x = 5	h_x = **48.68** ft	K_z = 1.09	q_z = 19.17	p_W = 13.04 psf	p_{TOT} = 21.32 psf	F_x = 16 kips
	0.00 ft	K_z = 0.85	q_z = 14.96	p_W = 0.00 psf	p_{TOT} = 0.00 psf	
x = 6	h_x = **0.00** ft	K_z = 0.85	q_z = 14.96	p_W = 0.00 psf	p_{TOT} = 0.00 psf	F_x = 0 kips
	0.00 ft	K_z = 0.85	q_z = 14.96	p_W = 0.00 psf	p_{TOT} = 0.00 psf	
x = 0	h_x = **0.00** ft	K_z = 0.85	q_z = 14.96	p_W = 0.00 psf	p_{TOT} = 0.00 psf	F_x = 0 kips
	0.00 ft	K_z = 0.85	q_z = 14.96	p_W = 0.00 psf	p_{TOT} = 0.00 psf	
x = 0	h_x = **0.00** ft	K_z = 0.85	q_z = 14.96	p_W = 0.00 psf	p_{TOT} = 0.00 psf	F_x = 0 kips

*Note: B > L produces maximun base shear. V = Sum (F_i) = 131 kips



Roof	
$p_R = q_h G C_{pR}$	
(Windward) C_{pWR} =	**-0.91**
(Leeward) C_{pLR} =	**-0.18**
Windward p_{WR} =	-15.1 psf
Leeward p_{LR} =	-3.0 psf

Figure 30: Wind Load Calculation for PRCC System, EW Direction

IBC 2003 (ASCE 7-02)

Wind Analysis, Main Windforce Resisting System, Analytical Procedure, NS Direction

$p_W = q_z\, G\, C_{pW}$ $q_z = 0.00256\, K_z\, K_{zt}\, K_d\, V^2\, I$ $K_h =$ 1.11

$p_L = q_h\, G\, C_{pL}$ (Windward) $C_{pW} =$ 0.8 $q_h =$ 19.5 psf

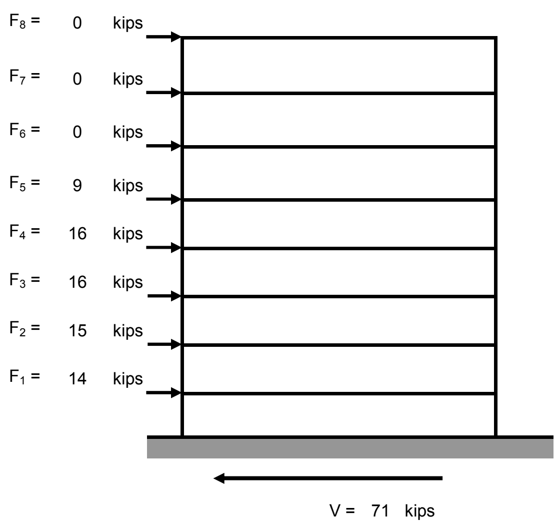
$p_{TOT} = p_W - p_L$ (Leeward) $C_{pL} =$ **-0.37** $p_L =$ -6.13 psf

h =	**52.7**	ft
No. Stories = n =	**6**	
L =	**150.80**	ft
B =	**91.00**	ft
L / B =	1.66	
V =	**90.0**	mph
K_d =	**0.85**	
I =	**1.00**	
Exposure Category	**C**	
Enclosure Classification	**E**	
Exposure Case	Case 1	
÷ =	9.5	
z_g =	900	ft
K_1 =	**0**	
K_2 =	**0**	
K_3 =	**0**	
K_{zt} =	1	
T =	**0.6110**	sec
f =	1.64	hz
G =	0.85	

Structure is rigid. G = 0.85.

	4.84 ft	K_z = 0.85	q_z = 14.96	p_W = 10.17 psf	p_{TOT} = 16.31 psf	
x = 1	h_x = **9.67** ft	K_z = 0.85	q_z = 14.96	p_W = 10.17 psf	p_{TOT} = 16.31 psf	F_x = 14 kips
	14.50 ft	K_z = 0.85	q_z = 14.96	p_W = 10.17 psf	p_{TOT} = 16.31 psf	
x = 2	h_x = **19.33** ft	K_z = 0.90	q_z = 15.78	p_W = 10.73 psf	p_{TOT} = 16.86 psf	F_x = 15 kips
	24.33 ft	K_z = 0.94	q_z = 16.57	p_W = 11.26 psf	p_{TOT} = 17.40 psf	
x = 3	h_x = **29.33** ft	K_z = 0.98	q_z = 17.23	p_W = 11.72 psf	p_{TOT} = 17.85 psf	F_x = 16 kips
	34.00 ft	K_z = 1.01	q_z = 17.78	p_W = 12.09 psf	p_{TOT} = 18.22 psf	
x = 4	h_x = **38.67** ft	K_z = 1.04	q_z = 18.26	p_W = 12.42 psf	p_{TOT} = 18.55 psf	F_x = 16 kips
	43.67 ft	K_z = 1.06	q_z = 18.74	p_W = 12.74 psf	p_{TOT} = 18.87 psf	
x = 5	h_x = **48.67** ft	K_z = 1.09	q_z = 19.17	p_W = 13.04 psf	p_{TOT} = 19.17 psf	F_x = 9 kips
	0.00 ft	K_z = 0.85	q_z = 14.96	p_W = 0.00 psf	p_{TOT} = 0.00 psf	
x = 6	h_x = **0.00** ft	K_z = 0.85	q_z = 14.96	p_W = 0.00 psf	p_{TOT} = 0.00 psf	F_x = 0 kips
	0.00 ft	K_z = 0.85	q_z = 14.96	p_W = 0.00 psf	p_{TOT} = 0.00 psf	
x = 0	h_x = **0.00** ft	K_z = 0.85	q_z = 14.96	p_W = 0.00 psf	p_{TOT} = 0.00 psf	F_x = 0 kips
	0.00 ft	K_z = 0.85	q_z = 14.96	p_W = 0.00 psf	p_{TOT} = 0.00 psf	
x = 0	h_x = **0.00** ft	K_z = 0.85	q_z = 14.96	p_W = 0.00 psf	p_{TOT} = 0.00 psf	F_x = 0 kips

*Note: B > L produces maximun base shear. V = Sum (F_i) = 71 kips



Roof	
$p_R = q_h\, G\, C_{pR}$	
(Windward) C_{pWR} =	**0.51**
(Leeward) C_{pLR} =	**-0.6**
Windward p_{WR} =	8.5 psf
Leeward p_{LR} =	-9.9 psf

Figure 31: Wind Load Calculation for PRCC System, NS Direction

Comparison

Seismic loads control the lateral resistance design for this building. The seismic base shear is 431k, while the largest wind base shear is approximately a third of that magnitude. As a result, 431k will be the design base shear used in this section of analysis.

Determination of PRCC Necessity

Initially, beams in the braced frames are sized for gravity loading. The design loads used are summarized in Table 9.

NS Braced Frame

NS Braced Frames		
Beam Label	Typ. Bay Purlin	
Span	26	ft
Tributary Width	30	ft
Influence Area	1560	sf
LL Reduction	62.5	%
Slab Thickness	5.5	in

Design Loads							
Load Case	Load (PSF)	Trib. Width (ft)	Distributed Load (lb/ft)	V (k)	M (k*ft)	LF	Mu (k*ft)
DL_B	62	30	1860	24.2	157.2	1.2	188.6
DL_A	35	30	1050	13.7	88.7	1.2	106.5
LL	40	30	750	9.8	63.4	1.6	101.4
Total				47.6	309.3		396.5
Construction Loads							
DL_B	62	30	1860		157.2	1.4	220.0
CLL	20	30	600		50.7	1.6	81.1
Total					207.9		301.2

Table 9: Design Loads for Typ. Beam in NS Braced Frame

Following the procedures outlined in AISC Design Guide 8, construction requirements are the initial determination of beam size. In addition to meeting strength requirements, a construction deflection check that utilizes $1.0DL_B$ and 1.0CLL is completed, using L/240 as a limit. The following calculations are for a typical beam in the NS braced frame.

$M_{ser,const}$ = M(DL_B+CL)= 207.9 k*ft

$I_{s,min}$ = (Mconst*240L)/(161*12)= 671 in^4

Because Ix_{req} is 671 in^4, the most economical section choice was to use a W21x44. Calculations for ultimate strength and serviceability of the beam in the composite structure were then completed. To check the ultimate strength, Y2=4.5” and PNA=7. Since ΦMn = 483ftk > 209.9ftk, the section strength capacity is acceptable.

Capacity of shear studs with ¾” diameter, f’c=3000ksi and concrete weight of 145 pcf is 12.1k per ASIC. Assuming the steel deck has fusion welds to supporting steel at 18” on center, the maximum stud spacing is 8 times the total slab thickness. When PNA=7, ΣQn=163k for W14x22. Therefore, minimum spacing controls and the beam should have 14 studs total.

Deflection was also checked using DL_A and live service loads,

ζ_{LL+DLA}= 0.45 in

which meets the deflection criteria of L/360.

This process was also repeated for beams on the EW braced frames. After initial analysis, it was determined that none of the beam connections necessitate partially restrained composite connections. A partially restrained composite connection would have been warranted had the nominal moment capacity of the composite section not been sufficient to support the design moment. A summary of the final member sizes follows.

Frame	Beam Location	Connection	Beam and Studs	Mu (ftk)	I_{LB} (in^4)
NS	Typ Bay Floor	PIN-PIN	W21x44 (14)	397	1420
EW	Int. Bay Floor	PIN-PIN	W16x40 (16)	277	886
	Ext. Bay Floor	PIN-PIN	W16x40 (16)	277	886

Table 10: Summary of Beam Sizes and Connections for PRCC System

Overall Design Summary and Conclusions

A composite floor system consisting of 1 ½" VLI deck with 5 ½" total slab thickness was chosen based on recommendations found in *Floor Vibration Serviceability: Tips and Tools for Negotiating a Successful Design*. This was done in an attempt to prevent future issues with walking vibration in the structure.

As a result of following the procedures outlined in AISC Design Guide 8, it was determined that none of the connections in the proposed braced frames required partially restrained composite connections. These types of connections are only used when the composite section does not have enough capacity to carry all of the design moment. Since this did not happen with the current braced frame layout, the use of partially restrained composite connections was not warranted. The sizes and capacities of the designed beams are found in Table 10.

Because of the magnitude of the required Ix of the beams due to construction loading, this system would not be a feasible alternate. While it uses a lighter flooring system than the existing precast one, the required structural depth would be approximately 30". This is considerably larger than the allowable limit of 1'-8". Additionally, some doorways would have to be relocated in order to fit into the bracing layout. Overall, the use of braced frames and partially restrained composite connections is not a feasible alternate and not studied further.

Construction Management Issues

Sweeping changes in the structural system by converting to the staggered truss system will result in significant changes in the constructability of Whiteland Village. In order to gain a better understanding of the impact of the proposed changes, an analysis of the cost and schedule implications will be completed.

Cost Analysis

RSMeans Building Cost Data, as well as pricing listed in *Engineering News Record*, was used to complete the cost analysis. Takeoffs were completed for the lateral force-resisting systems only. For the existing system, the CMU walls, concrete walls, scaffolding, and concrete footings were considered as part of the costs. Steel framing, fireproofing, and concrete footings were considered for the staggered truss system. Since the precast plank is the same for both systems, it was not included in the pricing.

Pricing was adjusted using the Turner construction cost index for inflation and using the city cost index provided by RSMeans for Philadelphia. All costs in this analysis are bare costs that include labor; no overhead or profit was included. The most expensive part of the existing lateral system is the cost of the 10" CMU shear walls, with a $106,236 (Table 11). However, scaffold rental does impact the cost greatly, almost as much as the concrete shear walls, at $60,255.

Material	Total Cost
CMU Walls	106236
Conc Walls	63120
Scaffold	60255
Conc Ftgs	43285
Total	272896
Adjusted Total	362618

Table 11: Summary of Costs for Existing Shear Wall Lateral System

The summary of costs for the alternate staggered truss system is found in Table 12. Not surprisingly, the truss chords are the most expensive part of the design. Considering these beams are W12x112 spanning 78', it makes sense that they would be the most expensive. The price of steel has been fluctuating over the past year. In 2006, the price rebounded to record levels in some areas. However, steel prices have been slowly settling to more normal levels. With this degree of fluctuation in the market, the accuracy of the RSMeans cost data for raw materials may be questionable. Footings are significantly cheaper in this system, because columns are not carrying the weight of five stories of masonry.

Material	Total Cost
STL Cols	36786
Fireproofing	40582
HSS Bracing	96941
STL Beams	279590
Conc Ftgs	5356
Total Cost	459255
Adjusted Total	610246

Table 12: Summary of Costs Staggered Truss Lateral System

The adjusted total for the existing system is $362,618, while the alternate staggered truss system costs $610,246. Obviously, the masonry system is considerably cheaper, although it is also more labor-intensive.

Schedule Implications

It was determined that the labor hours required to complete the masonry system are over three times as many as the staggered truss requires. Less field work is required with a staggered truss system because the trusses are typically shop fabricated and are ready to erect when brought on site. The critical path is considerably shorter with the staggered truss system, simply because it involves less field work. Smaller footings mean less time spent getting above grade. In addition, since the site is so large staging for a staggered truss system will not be a concern.

Building Envelope Analysis

Typically, 10-22% of the initial cost of a building is spent on the building envelope, according to the *Whole Building Design Guide*. The term building envelope encompasses all of the following:

- Below grade systems
- Exterior wall systems
- Fenestration systems
- Roofing systems
- Atria systems

Unfortunately, often architects and engineers overlook the building envelope in terms of design. For this analysis, the focus will be on the existing exterior wall systems and roofing systems. Both bulk water and moisture control will be studied and changes proposed, if necessary.

Bulk water is a critical issue for any envelope system. This is especially true in barrier wall systems, was appear to be used for this project. Figure 32 is a section of exterior wall just at the top of a window. In this section, there is no indication of an allowance for a drainage cavity behind the manufactured stone. Water that penetrates the cladding hits a water-resistant house wrap, but membranes are easily compromised during the construction process. Any tears in the wrap could result in water penetration into the plywood backing. This could result in wet insulation and a possible mold hazard. The drainage plane with drip edge provided at the base of the stone does not even effectively divert water from the wall, since the drip edge does not extend as far as the housing for the window below. This section could easily be improved by the addition of a drainage cavity, thus creating a drainage plane on the interior of the cladding and a greater obstacle to the water-resistant air barrier. In terms of water resistance, the use redundant systems are critical since a barrier wall system is easily compromised. Extending the drip edge past the window housing would also help to prevent water from corroding the window.

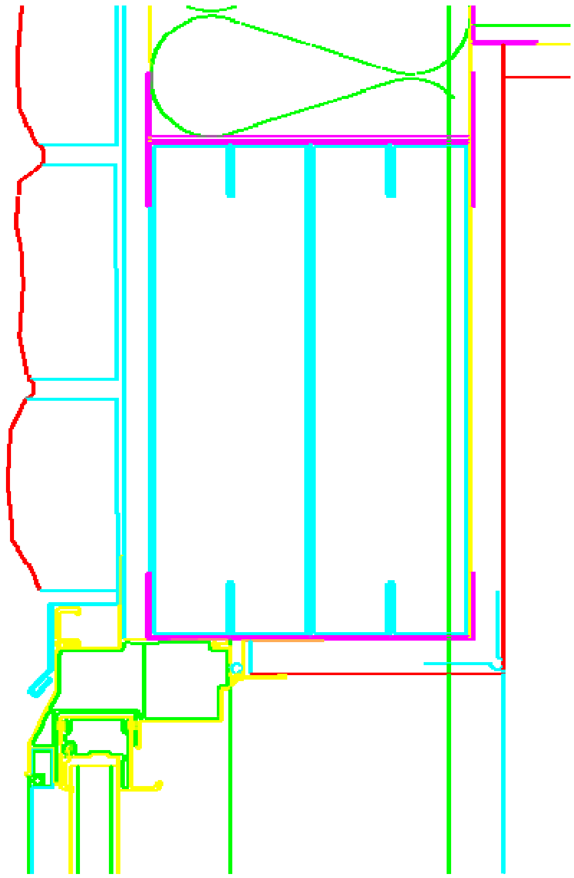


Figure 32: Exterior Wall Section above Window

In the specifications for the exterior walls, the manufactured stone is to be attached to metal lathe over house wrap. Exton Pennsylvania is in a moderate climate, where residents experience significant winter and summer, as well as humidity. This means that it is not desirable to use an air barrier that has vapor retarding properties, as it could hamper drying to the interior in warmer months and drying to the exterior in cooler ones. Tyvek house wrap specifies that it has vapor permeability while acting as an air barrier. This makes it a logical choice for this application. Since it is not acting as a vapor retarder, the location of the air barrier is not climate dependent, and can be placed in the wall according to ease of detailing.

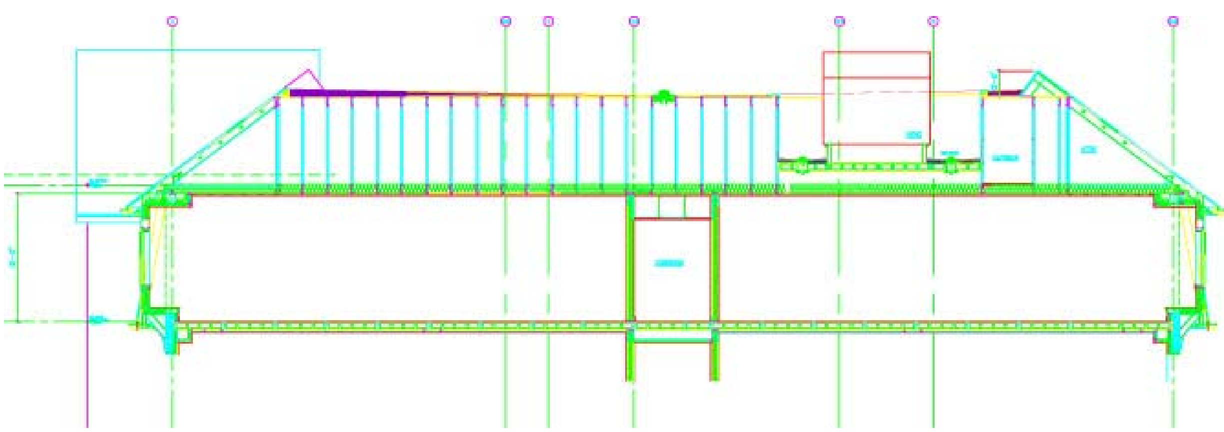


Figure 33: Roof Section showing Wells for Mechanical Units

Another section where bulk water is critical for this project is the roof. To allow for rooftop mechanical units, the metal stud over-framing is cut into, creating a well in the roof as seen in the section in Figure 33. There is huge potential for water and moisture failure at the mechanical units without proper detailing. Figure 34 shows the detailing specified in this project.

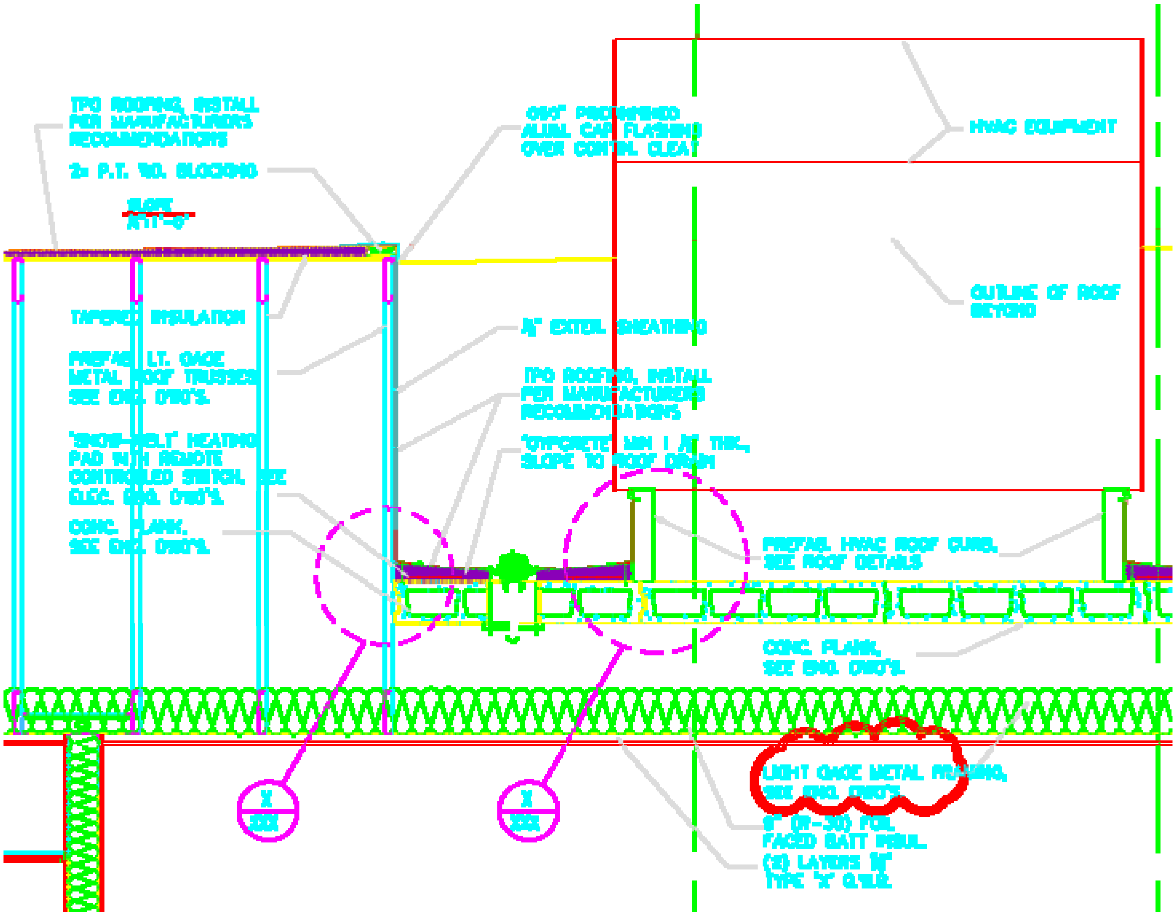


Figure 34: Detailed Section of Mechanical Unit Wells

The greatest concern when using a rooftop well is how to divert rain and snow buildup. Obviously the designer went to great lengths to prevent this from becoming an issue. The concrete topping is sloped to the roof drain from both the metal studs and concrete curb. A heating pad was included on the roof in order to ensure that snow drift buildup does not occur. Upon closer inspection, metal flashing is indicated at the connection from the metal stud wall to the concrete roof to prevent water penetration at that interface. Provided the roof drains have routine cleanings, the mechanical wells should not pose a serious concern.

Summary and Conclusions

Whiteland Village is an upscale residential complex consisting of three 5 story residence buildings, a commons building, and a healthcare facility. The basement level of the entire footprint serves as covered parking and utility spaces. Initial construction phases include the three residence buildings. The current structural system consists of 8" hollow core precast plank, spanning 30' between 10" CMU bearing walls. Lateral loads are resisted by a combination of concrete and masonry shear walls. In a small section of the J-Building, the structure is framed in steel moment frames and braced frames.

In an effort to reduce building weight and increase potential for future renovations, the possibility of changing Whiteland Village entirely to steel frames was investigated. To determine the feasibility, two different lateral systems were analyzed: staggered truss and partially restrained composite connections.

Staggered truss was a viable alternative structurally. Six trusses spanned the building's short direction, while two moment frames resisted lateral loads in the long direction. In this system, columns were W10x100 A992 for the full height of the truss, having minimal impact on wall thickness. Truss chords varied from W10x77 to W10x112 A992 members. The depth of the structural section was kept below the 1'-8" limit using these sizes. Diagonal and vertical bracing was determined to be HSS 8x6x5/8 A500 Gr B, except in the basement level where HSS 8x8x5/8 was required. The design was initially completed using 2-D analysis outlined in AISC Design Guide 14 and verified using a 3-D RAM model.

To analyze the use partially restrained composite connections, a braced frame layout using 30'x32' exterior bays and 30'x26' interior bays was analyzed. A floor system was chosen in order to prevent future walking vibration issues in the building. A 1 ½" composite deck with 5 ½ " total slab depth, using normal weight concrete, was chosen. However, during analysis, it was determined that with the construction load requirements impacted member sizes to the point that partially restrained moment connections were not required. Additionally, the total structural depth was significantly bigger than the allowable limit. Therefore, further analysis was not completed on this alternate.

Structural issues were not the only features of the project that were considered. Construction management issues were also analyzed. It was determined that the truss alternate was almost twice as expensive as the existing system. However, it requires three times as many labor hours to implement. Building envelope was also analyzed for potential failures due to bulk water and moisture penetration. It was determined that while the detailing around mechanical wells on the roof was sufficient, there were recommendations on improving the detailing around window openings. This included extending drip edges past the window housing and the inclusion of a drainage cavity.

After completing all the aforementioned analysis, it was determined that staggered truss is a viable alternate for Whiteland Village. Switching the building to steel is an effective way to increase the potential for renovation and more variation on condominium layouts. However, due to cost and schedule concerns, it is not recommended.

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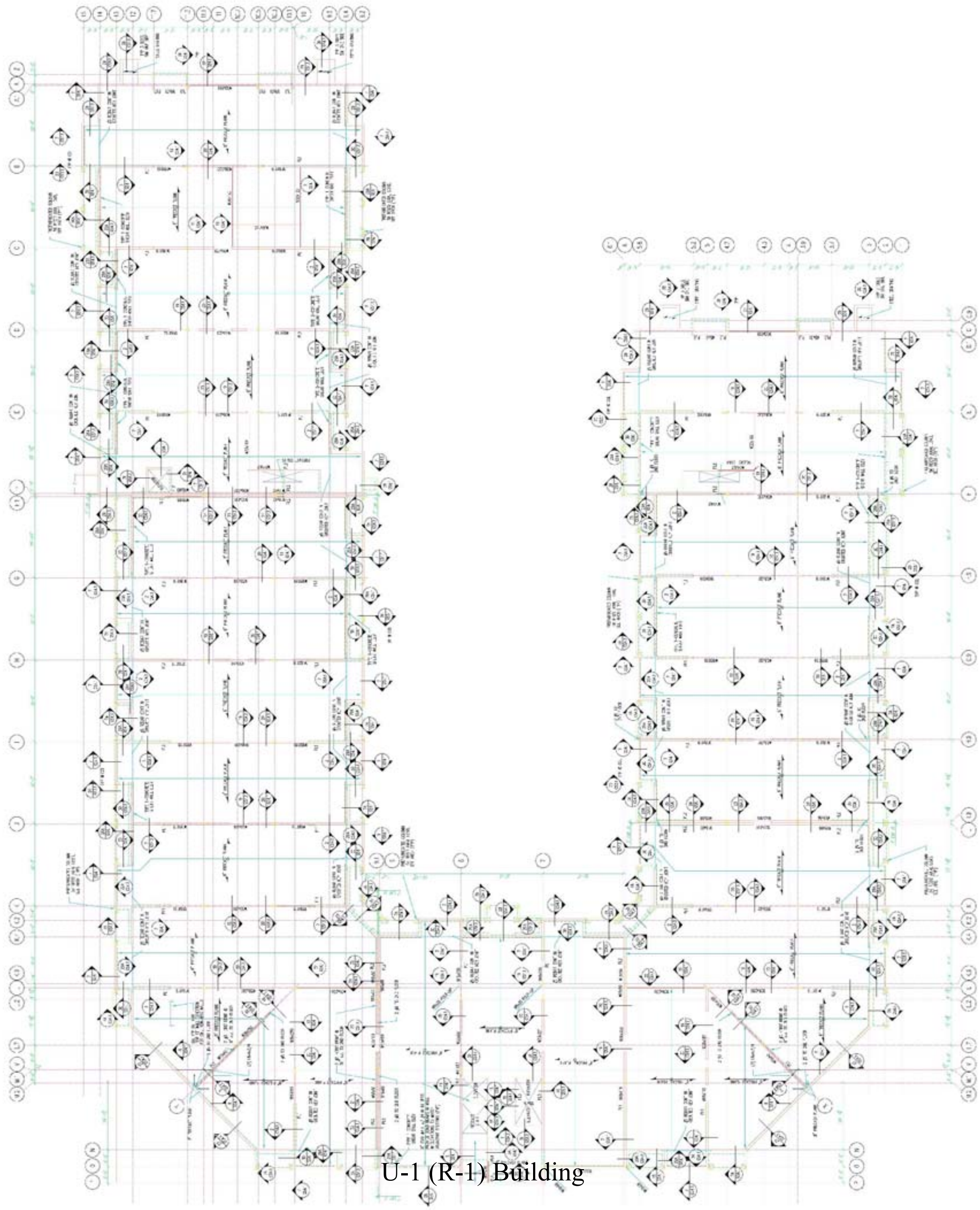
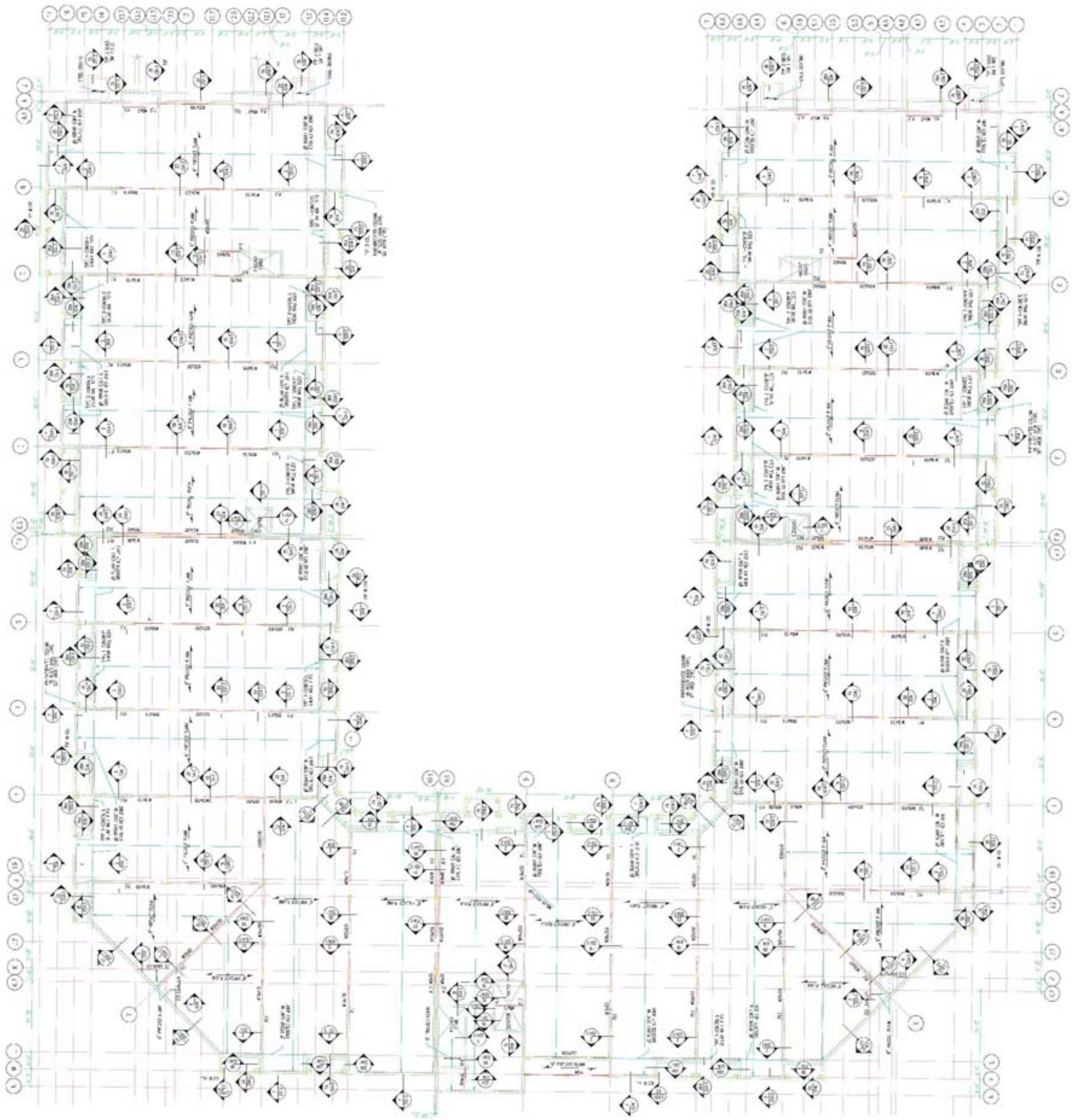
Special thanks to my friends and family, especially my parents, with all their endless support throughout my college career.

Appendix A: Master Plan of Whiteland Village

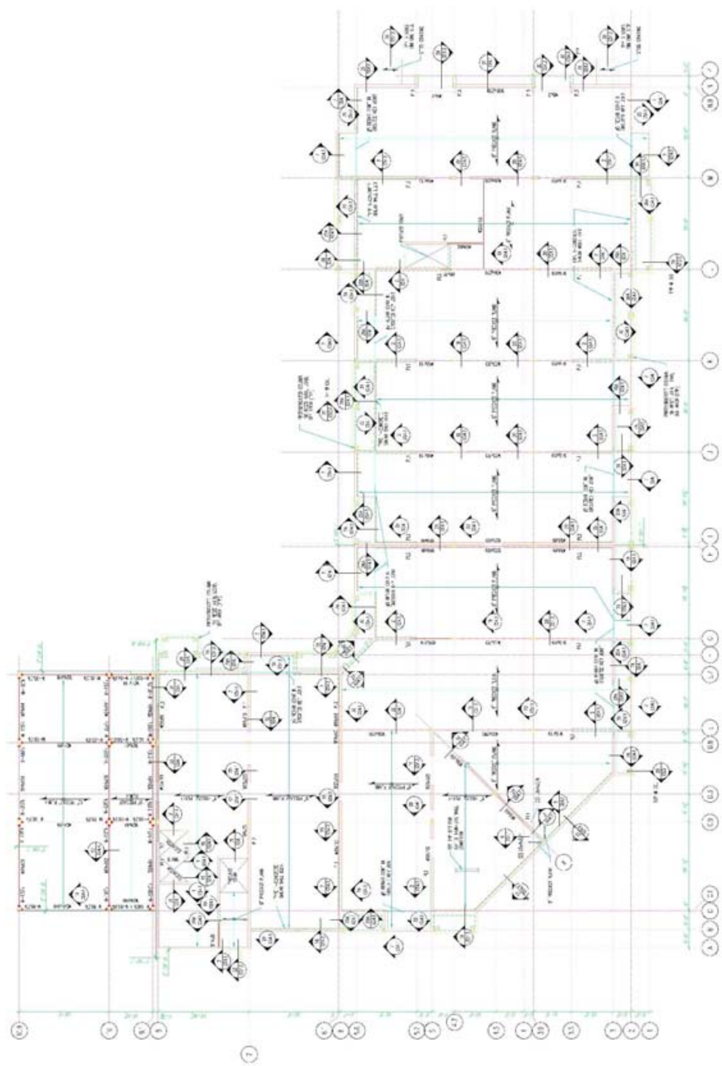
Appendix B: First Floor Plans of Residence Buildings



U-1 (R-1) Building



U-2 (R-4) Building



J (R-2) Building

Appendix C: Representative Calculations

The following includes calculations that are representative of those conducted in order to complete this analysis of Whiteland Village. Detailed calculations are available upon request.

Staggered Truss Design – Diaphragm Calculations

Center of Rigidity and Force Distribution

Odd Floors

Truss	x_i (ft)
T1E9	0
T1D	60
T1B	120
$\sum=$	180
x_o (ft)	60

Even Floors

Truss	x_i (ft)
T2E	30
T2C	90
T2A	150
$\sum=$	270
x_e (ft)	90

L (ft) =	150			
e_o (ft) =	15	±	7.5	
e_e (ft) =	-15	±	7.5	
T=	462.5	*15=	6938	ft-k
T=	462.5	*7.5=	3469	ft-k
Vs=	462.5	/3=	154	k

Transverse Shear in Diaphragm

Vu= $1.7*\Phi_{\eta}*\nabla*0.75$

Torsional Rigidity
Odd Floors

Truss	x_i (ft)	$x_i^{\ 2}$
T1E.9	-60	3600
T1D	0	0
T1B	60	3600
	$\sum=$	7200 ft^2

Even Floors

Truss	x_i (ft)	$x_i^{\ 2}$

T2E	-60	3600
T2C	0	0
T2A	60	3600

$\sum =$ 7200 ft^2

Shear Force in Each Truss due to Lateral Loads (Bottom Floor)								
	x_i (ft)	Vs	T (ft-k) = 6938		T (ft-k) = 3469		Design Shear	
Truss			V_{tors}	V_i	V_{tors}	V_i	V_i (k)	ϕ_{ecc}
T1E.9	-60	154	-57.8125	96	-28.9063	125	125	1.00
T1D	0	154	0	154	0	154	154	1.23
T1B	60	154	57.8125	212	28.90625	183	212	1.70
T2E	-60	154	57.8125	212	28.90625	183	212	1.00
T2C	0	154	0	154	0	154	154	1.23
T2A	60	154	-57.8125	96	-28.9063	125	125	1.70

V(min)= 125 k
Vu= 159.38 k
ϕVn= ϕ(Vc+Vs)

Assuming effective thickness of 4.8"
effective depth 80% of total depth
f'c=5000 psi

ϕVc= ϕ*2√f'c bd
ϕVc= 386.44 k

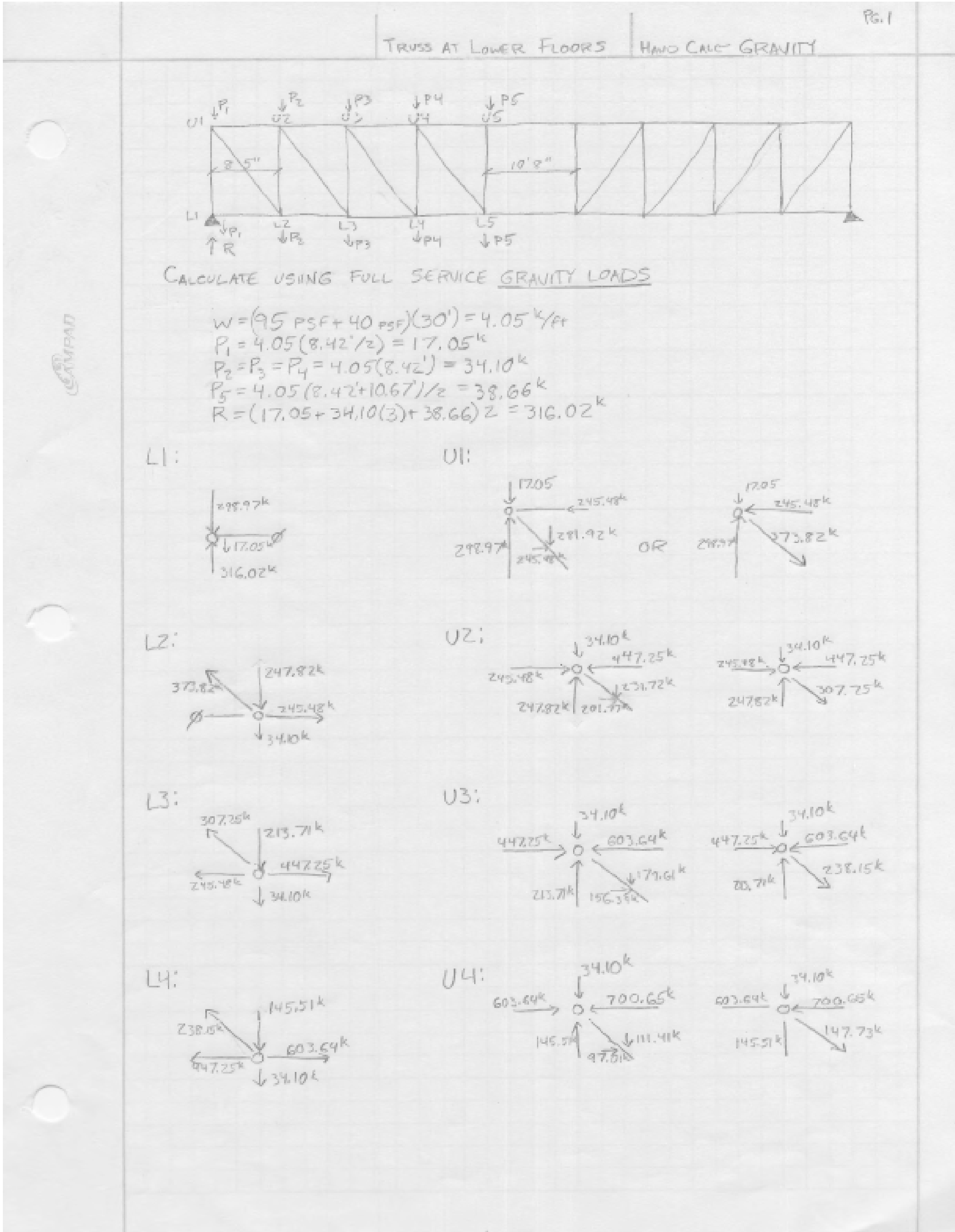
ϕVs= ϕ*A_{VF}fyμ

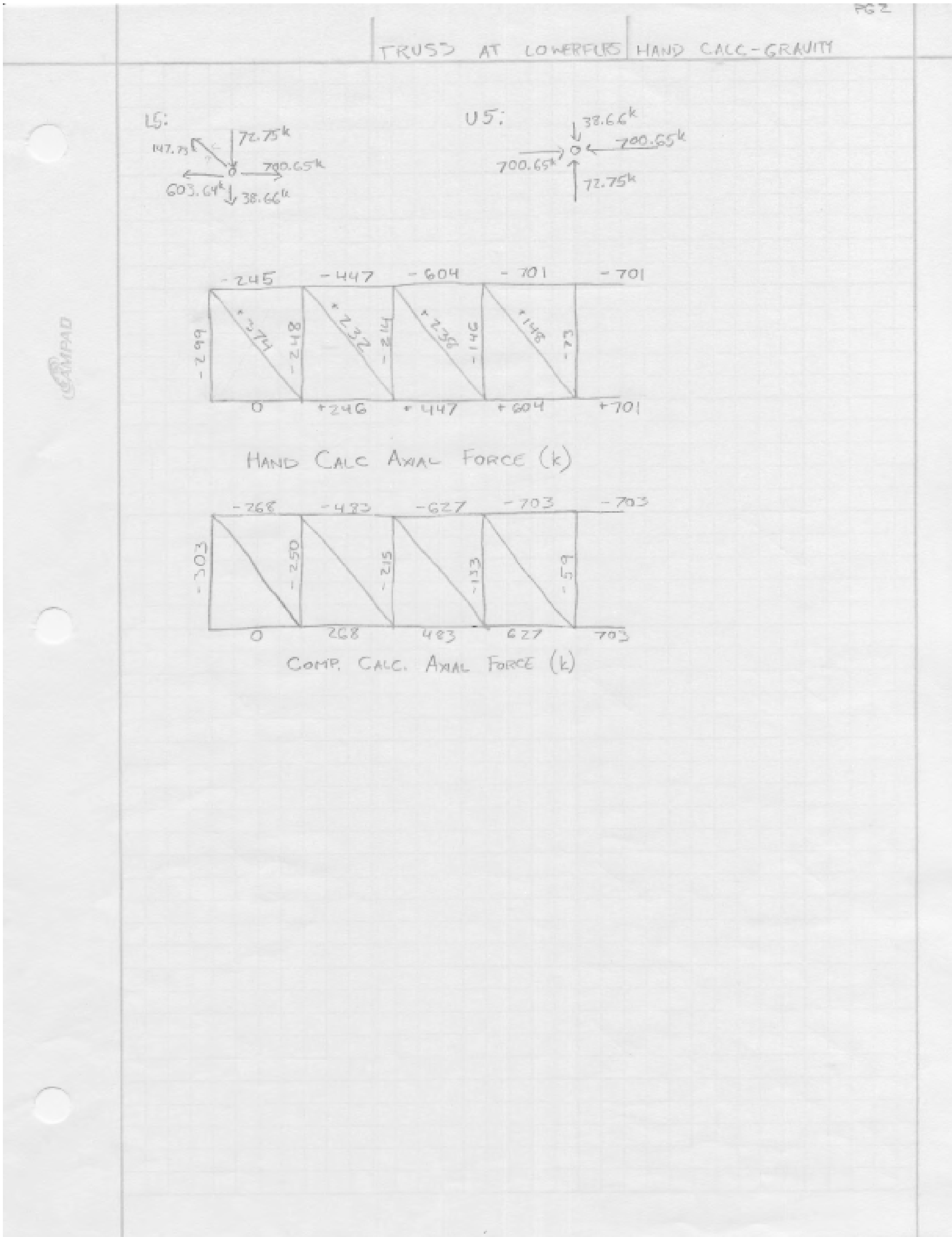
No. of planks =	78'/8'=	10	planks
No. of joints =	10-1=	9	joints
A_{VF} =	0.2*9=	1.8	in^2

ϕVs= 128.52 k

ϕVn= 514.96 k
Vu= 159.38 k OK

Staggered Truss Design – Single Story Truss Loads: Gravity and Lateral




PG 2
TRUSS AT LOWER FLRS
HAND CALC-GRAVITY
L5:
147.73
72.75k
700.65k
603.64k
38.66k
U5:
38.66k
700.65k
700.65k
72.75k
-245
-447
-604
-701
-701
-299
+374
-248
+232
-214
+230
-146
+148
-73
0
+246
+447
+604
+701
HAND CALC AXIAL FORCE (k)
-268
-483
-627
-703
-703
-303
-250
-215
-133
-59
0
268
483
627
703
COMP. CALC. AXIAL FORCE (k)

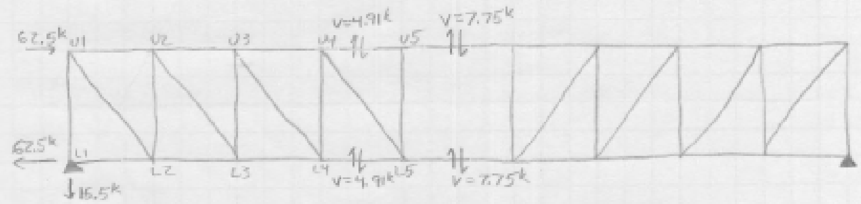
PG. 1

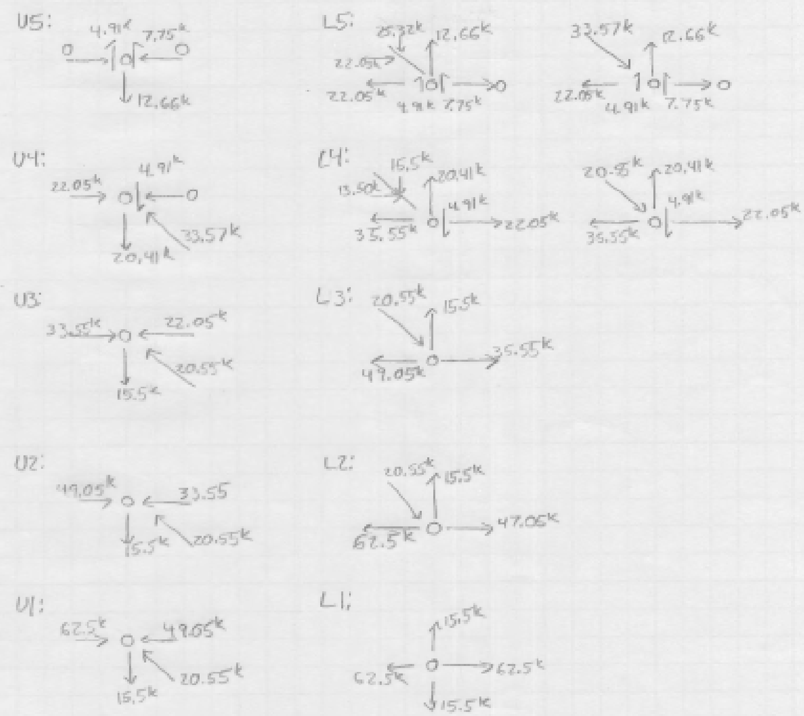
TRUSS AT LOW FLOORS | HANDCALC ~ LATERAL LOAD

$V = 125^k$ (T1 E.9)
$H = 62.5^k$
$R = [(62.5)(2)(9.67')]/78' = 15.5^k$

$V = \frac{1}{2}(62.5')(9.67')/39' = 7.75^k$ } FOR VIERENDEEL PANEL
$M = 7.75^k(10.67')/2 = 41.3'^k$

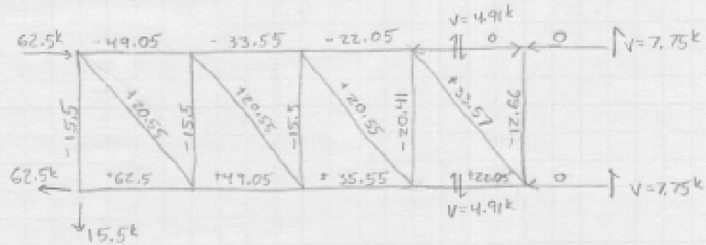
$M = 41.3'^k$ } FOR CHORDS ADJ. TO V. PNL
$V = (41.3'^k + 0)/8.42' = 4.91^k$





PG. 2

TRUSS AT LOW FLOORS | HAND CALC - LATERAL LOAD



AXIAL FORCE (IN k)

Staggered Truss Design – Chord Calculations

Truss	T1E.9	
w=	4.05	k/ft
Panel width	9.67	ft
M=	37.87	ftk
P,grav=	701	k
Mu, grav=	34.78531	ftk
Pu,grav=	643.8815	k

Floor	Φh	M	Mu,lat	Mu	Pu	Section	AISC Eq H1-1a
Roof	27.72	35	45.04	79.83	643.88	W10x68	0.9794
5	57.21	72	92.97	127.75	643.88	W10x88	
4	78.98	99	128.35	163.13	643.88	W10x88	
3	93.17	116	151.40	186.18	643.88	W10x88	
2	100.00	125	162.50	197.29	643.88	W10x88	0.9991
Ground							

Truss	T1D	
w=	4.05	k/ft
Panel width	9.67	ft
M=	37.87	ftk
P,grav=	701	k
Mu, grav=	34.78531	ftk
Pu,grav=	643.8815	k

Floor	Φh	M	Mu,lat	Mu	Pu	Section	AISC Eq H1-1a
Roof	27.72	43	55.49	90.28	643.88	W10x77	0.9090
5	57.21	88	114.54	149.32	643.88	W10x88	
4	78.98	122	158.13	192.91	643.88	W10x88	0.9899
3	93.17	143	186.52	221.31	643.88	W10x100	
2	100.00	154	200.20	234.99	643.88	W10x100	0.9432
Ground							

Truss	T1B	
w=	4.05	k/ft
Panel width	9.67	ft
M=	37.87	ftk
P,grav=	701	k
Mu, grav=	34.78531	ftk
Pu,grav=	643.8815	k

Floor	Φh	M	Mu,lat	Mu	Pu	Section	AISC Eq H1-1a
Roof	27.72	59	76.39	111.18	643.88	W10x77	0.9643
5	57.21	121	157.67	192.46	643.88	W10x88	0.9889
4	78.98	167	217.68	252.46	643.88	W10x100	0.9751
3	93.17	198	256.77	291.56	643.88	W10x112	
2	100.00	212	275.60	310.39	643.88	W10x112	0.9607
Ground							

Staggered Truss Design – Connection Calculations

CONN. DESIGN

DIAGONAL D1 TO CHORD

DIAG: HSS 8x6x5/8

$P_U = 457^k$ (TBL 3-4/IN LRFD)

CONN BTWN HSS & GUSSET PL

1) SHEAR LAG

$\phi R_n = \phi F_u A_e$
$= 0.75(62)(10.5)$
$= 488k$

2) SHEAR STRENGTH @ WELD

$\phi R_n = \phi(.6F_y)(4L_w t)$
$= 0.9(0.6)(46)(4)(20)(0.465)$
$= 924^k$

3) STRENGTH OF WELD CONN. G PL TO HSS

$\phi R_n = \phi F_w A_w$
$\theta = 0°$
$F_{EXX} = 70^{ksi}$
$F_w = 0.6(70)(1.0+0) = 42ksi$
$A_w = .707(3/8 - 1/16)(4)(20) = 17.68 in^2$
$\phi R_n = 0.75(42)(17.68)$
$= 557^k$

4) SHEAR STRENGTH OF GUSSET PL

$\phi R_n = \phi(0.6F_{y1})(2L_w t_1)$
$= 0.9(0.6)(50)(2)(20)(1/2)$
$= 540^k$

$\phi R_n > P_U$ ∴ USE 20" WIDE GUSSET PL A572 w/ 3/8" FILLET WELD

PRCC Design – Floor Vibration Calculations

PRELIM. VIBRATION ANALYSIS

FLOOR VIBRATION PRELIMINARY PROCEDURE
BASED ON "FLOOR VIBRATION SERVICEABILITY: TIPS & TOOLS FOR NEGOTIATING A SUCCESSFUL DESIGN" BY HANAGAN
NASC CONFERENCE, FEB 2-5 2003

CLASS 3 : BASED ON TBL. 1

$C_1 = U$ FOR 1.5" DECK w/ 4" TOTAL SLAB THICKNESS

∴ TRY 2" DECK w/ 4" TOTAL SLAB THICKNESS

$C_1 = U$

∴ USE A MIN OF 5" THICK SLAB

$C_1 = 0.422$ FOR 1.5" DECK
$C_1 = 0.451$ FOR 2" DECK

BAY SIZINGS & C_2 (FROM TBL. 5)

$L_j = 30'$, $L_g = 26'$ > $C_2 = 0.122$

$L_j = 30'$, $L_g = 32'$ > $C_2 = 0.077$

FOR 1.5" DECK w/ 4" TOT. SLAB THICKNESS
30'x26' : $C_1 + C_2 = 0.544 > .5$ ∴ FLR MAY BE UNACCEPTABLE
30'x32' : $C_1 + C_2 = 0.499 \leq .5$

FOR 2" DECK w/ 4" TOTAL SLAB THICKNESS
30'x26' : $C_1 + C_2 = .573 > .5$ ∴ FLR MAY BE UNACCEPTABLE
30'x32' : $C_1 + C_2 = .528 > .5$

TRY 5.5" TOT. SLAB THICKNESS
1.5" DECK
30'x26' : $C_1 + C_2 = 0.339 + 0.122 = 0.461 \leq .5$ ∴ FLR SHOULD BE ACCEPTABLE
30'x32' : $C_1 + C_2 = 0.339 + 0.077 = 0.416 \leq .5$
2" DECK
30'x26' : $C_1 + C_2 = 0.368 + 0.122 = 0.490 \leq .5$ ∴ FLR SHOULD BE ACCEPTABLE
30'x32' : $C_1 + C_2 = 0.368 + 0.077 = 0.445 \leq .5$

∴ USE 1½" VLI w/ 5.5" TOT. SLAB THICKNESS

PRCC Design – EW Exterior Beam Calculations

EW Braced Frames		
Beam Label	Typ. Ext. Bay Purlin	
Span	30	ft
Tributary Width	15	ft
Influence Area	900	sf
LL Reduction	75	%Lo in calcs
Slab Thickness	5.5	in

Design Loads							
Load Case	Load (PSF)	Trib. Width (ft)	Distributed Load (lb/ft)	V (k)	M (k*ft)	LF	Mu (k*ft)
DL_B	62	15	930	14.0	104.6	1.2	125.6
DL_A	35	15	525	7.9	59.1	1.2	70.9
LL	40	15	450	6.8	50.6	1.6	81.0
Total				28.6	214.3		277.4
Construction Loads							
DL_B	62	15	930		104.6	1.4	146.5
CLL	20	15	300		33.8	1.6	54.0
Total					138.4		200.5

Construction Requirements

$M_{ser,const}$ =	$M(DL_B+CL)$=	138.4	k*ft		
$I_{s,min}$ =	(Mconst*240L)/(161*12)=			516	in^4
Select Section					
W16x40	I_s =	518	in^4	ok	
	ΦM_p =	274	k*ft	ok	
	Y2=	4.5	in		
	PNA7=	4.56	in		
	ΦMn =	385	k*ft	ok	
	ΣQn=	147	k		
	I_{LB} =	886	in^4		
Stud Capacity					
12	studs >=	16	studs total		
Serviceability Deflection Checks					
ζ_{LL+DLA}=	0.69	in			
L /	520	<L/360	ok		
Checks if PRCCs Required					
I_{LB} =	886	in^4			
M_{ser} =	109.7	k*ft			
I_{LB}(ss) =	613	in^4			
I_{LB}(PR) =	351	in^4	For one end pinned		
I_{LB}(PR) =	198	in^4	For equal conn stiffnesses		