Nicole C. Drabousky

Senior Thesis, Spring 2006

Structural Option

Thomas E. Boothby

Wellington at Hershey's Mill

West Chester, Pennsylvania

Primary Project Team:
Owner: MSL Associates Ltd.
Architect: MSL Associates Ltd.
MEP Engineers: Sebastian & Sons, Inc.
General Contractor: Caldwell, Heckles & Egan Inc. (CH&E)
Electrical Contractor: State Electric
Structural Engineers: P.W. Moss & Associates

General Building Data:
Project Location: West Chester, Pa
Size (Total Sq. Ft.): 370,000 Sq. Ft.
Total Number of Stories: 5 Stories
Number of Stories Above Grade: 3-4 Stories
Dates of Construction: December 1, 2003 – August 15, 2005
Overall Project: $20,700,000

Construction

Wellington was a Design-Bid-Build project that included three phases of construction. Each phase was set to end twenty months after their start date. The construction of Wellington went only two weeks over the set end date.

Electrical

o Main service to the buildings is 35KV that feeds a 1000KVA transformer.
o Main Panel: 3000A switchboard, 480/277V, 65KAIC
o Each of 3 electric rooms: 500KVA transformer 480 TV 120/208 and 2000A switchboard to an 800A panel on each residential floor
o 300 KW generator for emergency power at 480V, one emergency panel for 480/277V lighting and a 75KW transformer 480 TV 120/208 panel for lighting
o Each apartment has 125A service and panel
o Generator also supplies power to a 40 HP fire pump

Lighting

o Common area and residential lighting at incandescent 120V
o Business offices, doctors' offices, game room and parking garage lighting at fluorescent 277V
o Each apartment has lighting fixtures and paddle fans
o Site lighting at metal halide 277V

Mechanical

o Common areas mixed split system and packaged rooftop heat pumps
o Residential areas mixed packaged thru-wall and packaged rooftop heat pumps
o Rooftop air conditioners and split system air handling units used for fresh air supply
o Exhaust fans in bathrooms, pool, parking garage, and beauty salon
o Electric wall and ceiling heaters for stairwells and exterior exits
o Stand alone thermostats control HVAC systems

Structural

o Foundation: 12" CMU foundation wall with 2' wide continuous footing and 4" slab on grade
o Lobby/Garage and First Levels: Structural steel framing of W-shape beams, open web joists, and metals studs on a 4" concrete slab over 1-1/2" metal deck
o Second and Third Levels: Wood framing made up of 2x8s and TJLs at 16" and plywood floor sheathing
o Roof framing: Sloped 24" wood trusses at 24"

Architecture

o Exterior walls finished with red and white stucco
o Balconies and a veranda with built up columns finished in stucco
o Porte Cochere at main entrance also with a stucco finish

Special Features

o Unique shape provides an interior courtyard
o Historical surroundings and a section of protected vegetation running along the site

Wellington at Hershey's Mill

Nicole Drabousky
Structural Option
http://www.arche.psu.edu/thesis/eportfolio/current/portfolios/ncd123/

Table of Contents

Executive Summary

Wellington at Hershey's Mill is a recently completed 370,000 square feet retirement community consisting of 5 stories and 197 independent living units located in West Chester, Pennsylvania. Wellington's structure consists of a non-composite steel framing system for the lobby and first floor and a wood floor joist, wood framed system on the top three residential levels.

The depth study for this thesis is the design of an alternate structural system that is more appropriate for Wellington's intended use. A less combustible material was preferred for a retirement community, therefore the chosen system is a hollowcore floor system supported by masonry bearing walls.

Two additional analyses were performed for the breadth study of this thesis. The intention of these studies was to determine which system offered an improved standard of living for the occupants of Wellington. An acoustical analysis of the floor system between the garage and the first floor residential section was completed to verify the amount of noise transferred through the two floor systems. To determine the amount of heat lost through the exterior walls of the apartments, a building envelope heat transfer analysis was also conducted.

A summary of my findings are:

- An initial design of the alternate system showed the need for the masonry bearing walls of the residential levels to bear directly on the columns in the garage.
- The weight of the structure increased significantly, making the current lateral system fail after the application of the new seismic loads. Reinforcing the masonry of the lateral system will allow the system to resist the loads.
- The acoustical analysis showed the alternate system to be a superior acoustic barrier between the garage and first floor.
- Heat loss calculations proved the original system was better for slowing heat loss.

Project Background

On a site between protected vegetation on a section of Serpentine Stone Ridge and major roadways in West Chester, Pennsylvania, Wellington at Hershey's Mill's construction began in December of 2003 and consisted of three phases. The first phase was to be finished within 8 months of the start date and the second and third phases were scheduled for 20 months after the start date.

Due to miscommunications mainly between the architect, MSL Associates Ltd., and the general contractor, Caldwell, Heckles & Egan Inc. (CH&E), the first phase was not finished in the allotted time. After coming to an agreement, the contractors worked well together and completed the next two phases by August 15, 2005, just two weeks later than the goal of August 1, 2005.

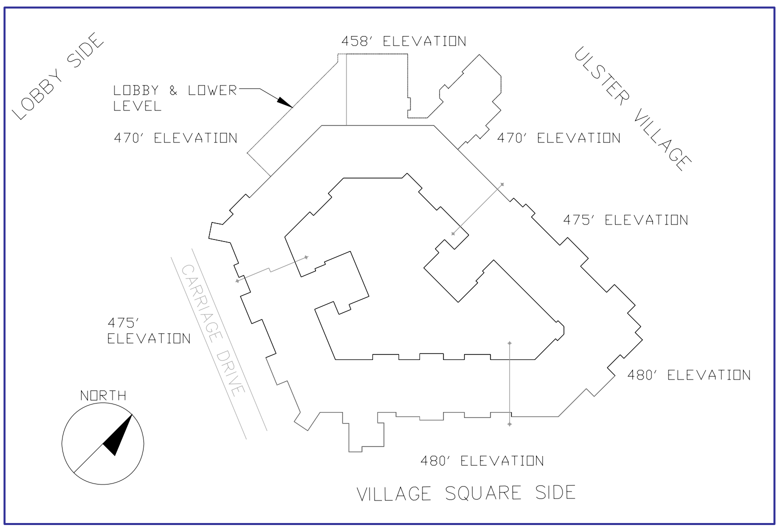
The GC's contract with the owner was a bid guaranteed max price contract and the original cost was set at $19,400,000. Setbacks and change orders caused the final price to increase to $20,700,000.

MSL Associates Ltd. is the owner of Wellington as well as the architect. The rest of the project team consisted of CH&E as the general contractor, Sebastian & Sons, Inc. as the mechanical engineers, State Electric as the electrical contractors, and P.W. Moss & Associates as the structural engineers. All of the contractors are located in the tri-state area.

General Architecture

Building Description

Wellington at Hershey's Mill is comprised of five levels but alternating ground elevation only allows all five levels to be above ground on the south side of the building. The structure can essentially be thought of as two separate buildings; a garage level with the three residential floors above as one building and the lower level with the lobby above as the second building. The lower level is a full floor below the garage level elevation, so the lobby is at the same elevation as the garage. The shape of Wellington is similar to that of a doughnut, permitting a courtyard to be encompassed by the three residential levels. The image to the left illustrates the elevation change of the land, the shape of the structure, and the location of the lobby and lower level with respect to the residential section of Wellington.



Envelope

The façade of Wellington incorporates a white stucco finish with white balconies and verandas. The stucco was conventionally applied to paper covered sheathing and lath. Metal stud exterior walls on the lower, lobby and garage levels and wood stud exterior walls for the residential levels made up most of the building's envelope with the exception of the concrete masonry unit (CMU) foundation walls.

Foundation & Structure

Wellington's foundation is made up of a 12" CMU foundation wall with 2' wide strip footings and 4" slab on grade with 6x6-W2.0xW2.0 WWF over 2-4" porous fill. The first floor's interior steel columns sit on concrete spread footings ranging in size from 3'x3' to 4'-6"x4'-6". The rest of the first floor framing are steel girders supporting steel joists which in turn hold a 4" concrete slab over 1-1/2" metal floor deck (galvanized) with 6x6-W2.9xW2.9 WWF. The lobby continues with the same framing as the first floor.

Wood framing is used for the second and third floor and the roof of the residential part of Wellington. Open web wood trusses, TJLs, at 16" on center in the apartments and 2x8's at 16" on center in the corridor make up the floor framing for the second and third floor. The floor system bears on 2x6 wood stud walls. Wellington's roof is similar to the second and third floor framing except there are 24" sloped roof trusses at 24" on center. Wellington's lateral load resisting system is a combination of wood framed gypsum shear walls and masonry towers located at the elevator shafts and stairwells.

Depth Study

The depth study involves an alternate design for the structure of Wellington.

Introduction

For the depth study of this thesis, an alternative structure for Wellington will be designed. While the existing structure is sufficient, the building's use as a retirement community requires a design with a less combustible material. For this reason, the alternative design employs masonry and concrete as the main components.

The intention of this depth study is to design the floor system, bearing walls, and foundation walls of the alternate system. The masonry towers of the lateral load resisting system remain the same as the original because it is appropriate for the architectural layout of the building. The seismic load was recalculated with the new structure weights and applied to the building. The original masonry towers failed with the increased seismic load. This is because they are constructed of unreinforced masonry; reinforcing it would allow the system to resist the load. The calculations are not included in this report, but can be seen upon request.

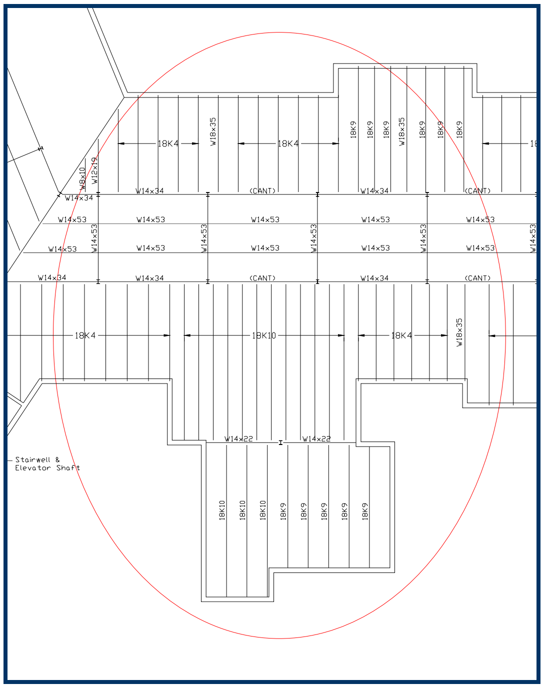
The alternate system was designed keeping the breadth analyses in mind. The design was restricted to the residential section of Wellington along with the garage directly below. Although the original intention was to keep the architecture as it was designed, it was necessary to alter the width of the hallways as well as the center section of the garage because the masonry walls of the residential floors formerly bore onto hollowcore plank. The alternate design allows the masonry walls to bear directly onto the columns in the garage. The hallways and center section of the garage were adjusted to a width of 12 feet. The weight of the alternate design was larger than the original and this approach was a

reasonable way to solve the problem of how to manage the increased weight. The increased hallway width also fits for the practical use of the space as the elderly often have motorized scooters or wheelchairs and will benefit from the increased space in the hallway. The tradeoff of gaining this increased hallways space is that each apartment has lost 3 feet in length, a small loss compared to the increased building stability and improved common hallway area.

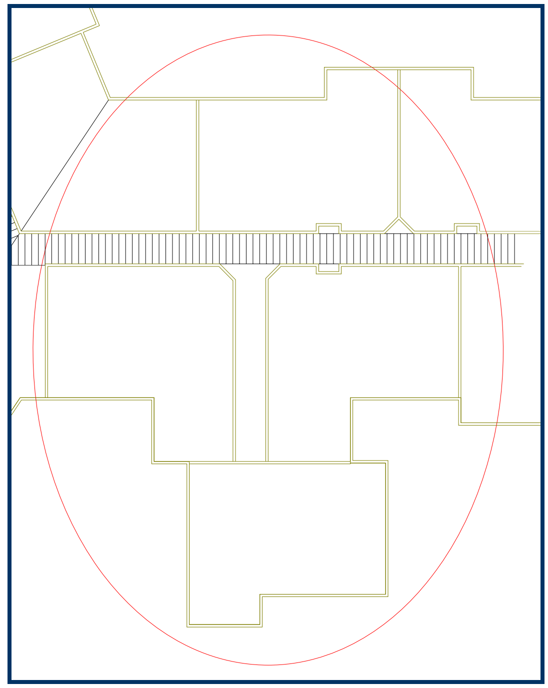
The design live loads used for both the original system and the alternate system (ASCE 7 -02 Table 4-1):

Roof:	**20 psf**
Private Rooms & the corridors that serve them:	**40 psf**

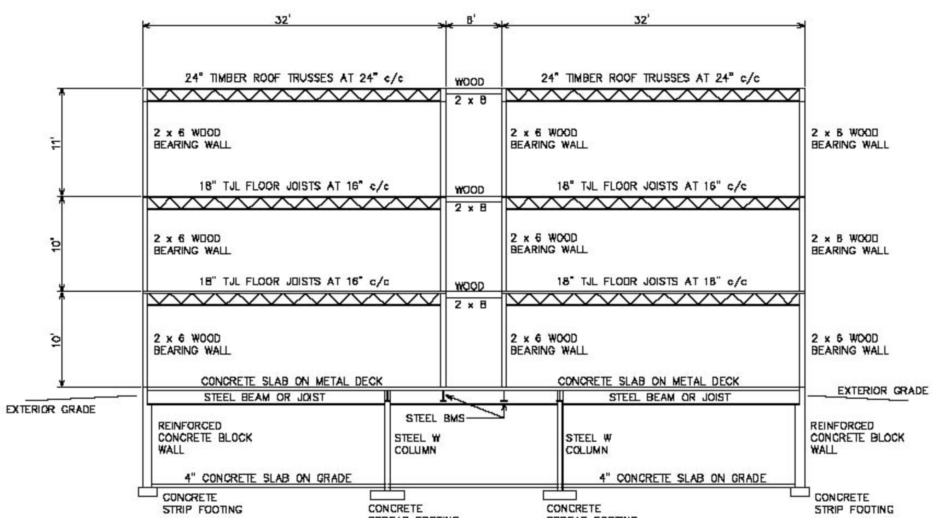
Existing Structural System



First Floor Framing

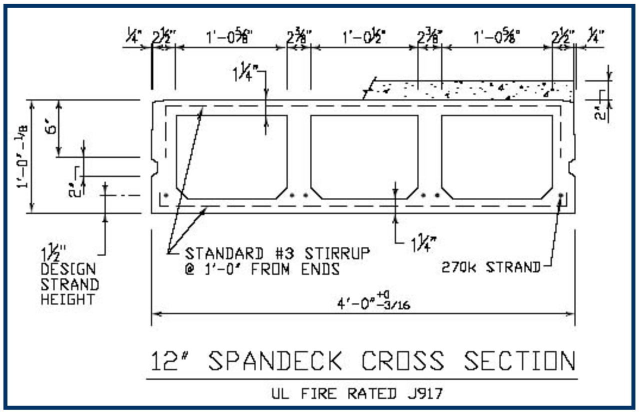


Second Floor Framing



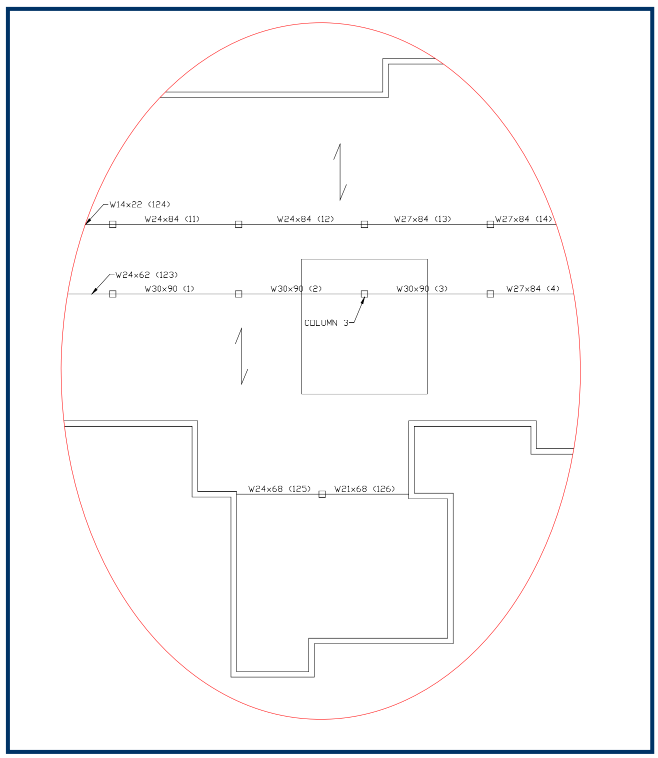
Original Building Section

Alternate Structural System



Nitterhouse pre-cast concrete hollowcore planks were chosen for the alternate floor system. Using the Nitterhouse load tables and a maximum span of 36 feet, the 12"x4' SpanDeck with 2 inch topping was chosen for the first through third floors. The topping allows the planks to behave uniformly. The roof will use the same plank without the 2 inch topping.

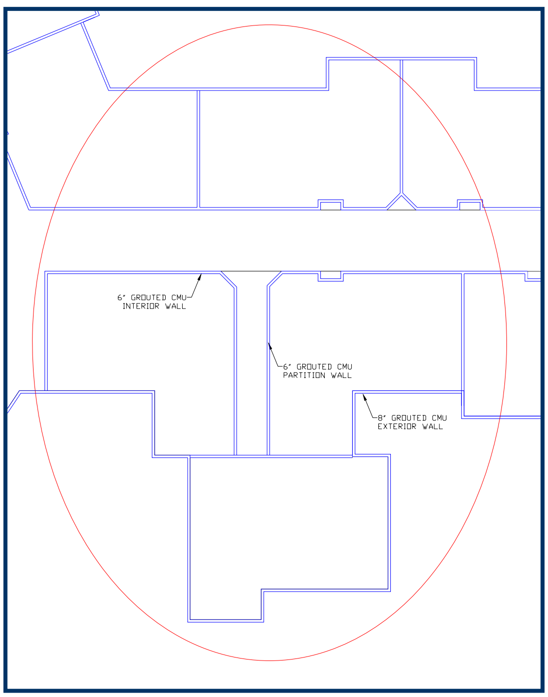
The first floor framing required concrete columns and steel beams to support the floor system because of the garage. LRFD was used to design the steel beams and the calculations were performed with the aid of a spreadsheet. A section of the designed system is shown to the right.



The concrete columns were sized using the CRSI and a calculated point load from the designed structure. Different columns were chosen to check any large differences in tributary area and load but there was not a significant difference between columns, so alternate reinforcing for each column was not necessary. A 14"x14" square tied column with 4-#10's was chosen even though a 12"x12" with 8-#9's was adequate for the load. This is in case a minor accident occurs in the garage damaging a column; a wider column would be more durable and less likely to fail, preventing further damage and possible loss of life.

The second and third floors have masonry bearing walls to support the hollowcore planks. Empirical masonry design along with a spreadsheet for the calculations was used to size the walls. The exterior bearing walls were determined to be 8" grouted CMU with Type N Mortar and the interior bearing walls 6" grouted CMU with Type N Mortar. These widths were chosen to match the original design. Partition walls will be 6" grouted CMU as well to provide a sound barrier. A section of the designed structure is shown below.

The garage walls were designed using the worst case section where the full height of the wall, 15 feet, retained the earth. The soil information used for West Chester, Pennsylvania was:

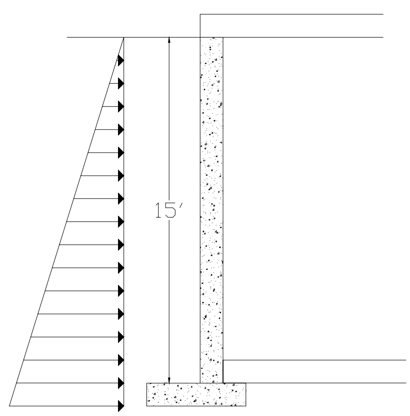


At rest: equivalent fluid pressure = 57 psf

Total unit weight = 125 pcf

Frost depth = 36 inches

It was assumed that there was no cohesion or surcharge present in the soil. The final design was a 12" concrete wall with #7's at 12 inches.



Breadth Study

The breadth study involves acoustical & envelope heat transfer analyses and comparison of the original and alternate system.

Acoustical Analysis

An acoustical analysis of the floor system between the garage and the first floor residential space was performed to assess how adequate it was at preventing noise from transferring between the two. A comparison between the original and alternate systems shows the alternate design, masonry, to be a superior sound barrier.

The effective transmission loss (TL) of each system was found from *Architectural Acoustics (*Egan, M. David, Architectural Acoustics, 1988, McGraw-Hill, Inc., New York.). Although the exact systems were not listed, it was assumed the differences between the chosen systems were negligible. After the actual TL for each system is calculated, it was compared to the TL of the floor systems, shown on the following page.

Sound absorption coefficient is a property of a material that indicates the amount of sound that is absorbed by the materials. The construction of the floor, ceiling, and walls of the apartment were listed and the sound absorption coefficients of each were found for different frequencies. The coefficients were multiplied by the area of the spaces and the sum was taken for each frequency. These numbers, represented by the variable a_2, are inserted into the equation $10\log a_2/S$ where S is the surface area.

The calculations begin with the source noise in decibels coming from the garage; again there's a different dB for each frequency. The background noise level in the apartments is subtracted from this number, also in dB for each frequency. This value is known as the required Noise Rating (NR). To calculate the final system TL, $10\log a_2/S$ is equated and subtracted from the required NR. Appendix 3 contains the complete calculations.

Comparison

Alternative

4 in reinforced concrete slab (54 lb/ft^2) (actual system: hollowcore plank system)

	125 Hz	250 Hz	500 Hz	1000 Hz	2000 Hz	4000 Hz
Req'd TL (dB):	26.24453039	45.75724	48.24939	53.02373	55.13228	55.13228
System TL (dB):	48	42	45	56	57	66

Original

18 in steel joists 16” o.c. with 1 5/8” concrete on 5/8 in plywood under heavy carpet laid on pad, and 5/8 in gypsum board attached to joists on ceiling side (20 lb/ft^2)
(actual system: 18 in steel joists 16” o.c. with 4” concrete slab, heavy carpet laid on pad and gypsum board attached to joists on ceiling side)

	125 Hz	250 Hz	500 Hz	1000 Hz	2000 Hz	4000 Hz
Req'd TL (dB):	27.22276395	45.67606	48.42668	52.91515	55.36212	55
System TL (dB):	27	37	45	54	60	65

Conclusion

Both systems have frequencies in which the TL is not adequate for sound isolation, but it can be assumed that the performance of each system is actually better than calculated because of likely real world conditions. The alternative system will have more concrete thickness than the system used in the calculations, especially with the addition of a two inch topping. The original system also has more concrete thickness than the example system because of the 4” concrete slab; over 2” inches thicker.

Comparing the systems with the calculations above would suggest the alternative system was a better sound barrier for the floor between the garage and first level.

Building Envelope Heat Transfer Analysis

A building envelope analysis was performed to assess the heat transfer through both the original and alternate systems. Since the most important section of Wellington is the living area of its residents, the analysis was through the exterior walls of the same sized room as in the acoustics analysis. The heat was calculated assuming an average January winter day in West Chester, Pennsylvania (average high = 39ºF, average low = 21ºF). (www.weather.com)

The calculation for the heat lost through the exterior walls involves the overall change in temperature from the interior to the exterior, the area of the wall, and the overall U-value of all the wall layers. The U-value for each system was found with a software program called *Carrier Hourly Wall Analysis.* The calculations can be found in Appendix 4.

Original System

Wall Details

Outside Surface Color .... **Light**
Absorptivity **0.450**
Overall U-Value .... **0.046** BTU/(hr-ft²-°F)

Wall Layers Details (Inside to Outside)

Layers	Thickness in	Density lb/ft³	Specific Ht. BTU / (lb - °F)	R-Value (hr-ft²-°F)/BTU	Weight lb/ft²
Inside surface resistance	0.000	0.0	0.00	0.68500	0.0
Gypsum board	0.500	50.0	0.26	0.44803	2.1
Air space	0.000	0.0	0.00	0.91000	0.0
R-13 batt insulation	6.000	0.5	0.20	19.23077	0.3
1-in stucco	1.000	116.0	0.20	0.19984	9.7
Outside surface resistance	0.000	0.0	0.00	0.33300	0.0
Totals	**7.500**	-		**21.80664**	**12.0**

A comparison of the two systems show the heat loss for the alternate system is twice as large as the one for the original. The larger thickness of insulation in the original system significantly increased the overall thermal resistance of the wall. The U-value for the alternate system can be decreased with the replacement of the R-13 insulation with a higher R-value insulation. Increasing the thickness of the insulation, and therefore the wall, may be an unwelcome option.

Summary & Conclusion

The purpose of this thesis was to take Wellington's use as a retirement community into account and design a system constructed of a less combustible material. Pre-cast concrete hollowcore planks were chosen for the floor system due to the fire resistance and ease of construction. To size the planks, the Nitterhouse load tables were used after an allowable load and span were chosen. It was determined the 12"x4' SpanDeck with 2 inch topping was sufficient for all three residential floors and the same plank without topping for the roof system. The first floor framing was then designed to have square concrete columns supporting the beams that would hold the planks. LRFD was used to design the first beam, as shown in a sample calculation in Appendix 2, and a spreadsheet was set up to design the rest of the beams on the first level. The concrete columns were sized by choosing one column that received the highest load and had the largest tributary area and choosing a size and reinforcement from the *CRSI Handbook*. The chosen column was a square tied 14"x14" with 4-#10s.

Masonry walls were chosen to support the planks on the residential floors above the garage. The interior bearing walls were lined up with the columns in the garage in the alternate design because of the need for improved stability in the structure. Empiracal masonry design along with a spreadsheet was used to size the walls. The results were 8" CMU exterior walls and 6" CMU interior and partition walls.

The foundation wall was designed assuming the full height of the wall retained soil. It was also assumed that there was no cohesion or surcharge present in the soil. The final design was a 12" concrete wall with #7's at 12 inches.

An acoustical analysis of the floor system between the garage and first floor showed that the alternate system design was a better acoustic barrier than the original. The heat loss calculation performed showed that the original system was better than the alternate. The purpose of these analyses was to determine which design offered a better standard of living to the retirees. The results were mixed as both designs had their strong points. The alternate design offers more resistance to fire and better acoustical properties, while it is less efficient considering heat loss.

References

AISC Committee on Manuals and Textbooks (Eds.)(2003). *Manual of Steel Construction: Load and Resistance Factor Design* (3rd ed.). United States of America.

American Society of Civil Engineers (Eds.)(2003). *Minimum Design Loads for Buildings and Other Structures: ASCE 7-02.* (Revision of ASCE 7-98). United States of America.

Concrete Reinforcing Steel Institute (2001). *CRSI Design Handbook* (9th ed.). United States of America.

Egan, M. David, Architectural Acoustics. (1988) McGraw-Hill, Inc., New York. *Architectural Acoustics.*

Nitterhouse.com. Last retrieved March 31, 2006, from http://www.nitterhouse.com.

BeavertownBlock.com. Last retrieved March 31, 2006, from http://www.beavertownblock.com

Credits/Acknowledgements

I would like to thank the following people…

Greg Del Nero at MSL Associates for providing me with Wellington's information.

Mike Merwin from East Goshen Township for providing the zoning and building codes for Wellington.

Patrick Vander Neut from Sebastian & Sons, Inc. for all the mechanical information.

Philip Moss from P.W. Moss & Associates for all the structural information and help.

Professor Boothby, my faculty consultant, for pushing me to do things he believed I could do from the beginning.

Professor Parfitt, for patiently explaining everything.

Professor Hanagan for answering my questions without even knowing my building.

Jeff Kobilka, my boyfriend, for his support and help with my report.

And **Katia & Nick Drabousky,** my parents, for their support as well as the electrical information from my dad, the electrical contractor for Wellington.

Appendix

Calculations, spreadsheets,

& tables used in this thesis study.

Appendix 1: Gravity Loads – Alt. System – ASCE 7-02

Live Loads (ASCE 7 -02 Table 4-1):

Roof: **20 psf**

Private Rooms & the corridors that serve them: **40 psf**

Dead Loads (used for Seismic load):

Hollowcore Roof:	*Ceiling*	*1 psf*
	MEP	*10 psf*
	Hollowcore	*77.5 psf*
	Total:	***88.5 psf***
Hollowcore Floors:	*Carpet*	*1 psf*
	Ceiling	*1 psf*
	MEP	*10 psf*
	Hollowcore	*102.5 psf*
	Total:	***114.5 psf***

Steel + Hollowcore Floors:	*Carpet*	*1 psf*
	Ceiling	*1 psf*
	MEP	*10 psf*
	Steel	*10 psf*
	Hollowcore	*102.5 psf*
	Total:	***124.5 psf***

Snow Load (ASCE 7 -02):

$p_f = 0.7\ C_e C_t I p_g$

$C_e = 0.7$ (table 7-2)

$C_t = 1.0$ (table 7-3)

$I = 1.1$ (table 7-4)

$p_g = 30$ psf

$p_f = 0.7\ (0.7)(1.0)(1.1)(30\text{ psf}) = 16.17\text{ psf} < p_f = 20I = 22\text{ psf}$

Therefore, ***$p_f = 22$ psf***

Appendix 2: Depth Study Calculations

First Floor Framing

Largest hollowcore plank span = 36'-0"

LL = 40 psf

Superimposed dead load:	*MEP*	*10 psf*
	Ceiling + Carpet	*2 psf*
	Partition walls	*20 psf*
	Total:	***32 psf***

w_u = 1.2(32 psf) + 1.6(40psf) = 102.4 psf

From Nitterhouse Hollowcore Load Tables

Span = 36'-0"

Allowable Superimposed Load (psf): flexure 6 – ½" ø = 104 psf > 102.4 psf

shear 6 – ½" ø = 136 psf > 102.4 psf

***Use 12" x 4' SpanDeck – U.L. – J917 w/2" topping**

*For all floors. Roof will use same size w/o topping.

Design Beams w/LRFD

Example Calculation – Beam 1

Span = 22'-8", Tributary width = 23.96'

Plank weight = (102.5 psf * 3 floors) + 77.5 psf (roof) = 385 psf

Interior masonry bearing wall weight = 62 psf (10')*(2 floors) + 62 psf (11') = 1922 plf

w_u = 1.2[((385+27)*23.96') + 1922 plf] + 1.6(40psf)(23.96') = 15685.664 plf (factored)

w_u = ((385+27)*23.96') + 1922 plf + (40psf)(23.96') = 12751.92 plf (unfactored)

$M_u = wL^2/8$ = [15685.664 plf $(22.67')^2$]/8 = 1007664.506'lb = 1007.66'k

$V_u = wL/2$ = [15685.664 plf (22.67')]/2 = 177797lb = 177.8k

For Δ_{max} = l/240, $I_{req'd}$ = [5(12.752klf)$(22.67')^4$$(12in/ft)^3$]/[384(29,000ksi)(1.1335")] = *2305.41 in*^{4}

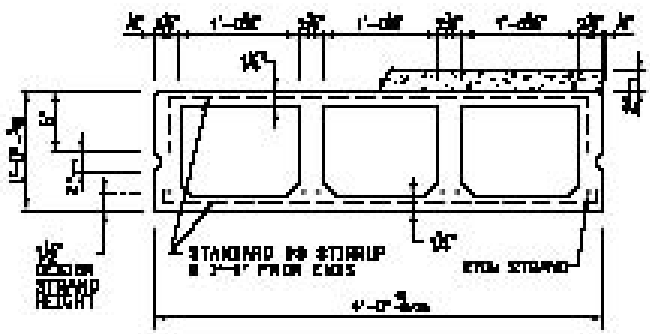
Use W30x90 (table 5-3, AISC LRFD Manual of Steel Construction)

Prestressed Concrete
12"x4' SpanDeck–U.L.–J917
(2" C.I.P. TOPPING)

PHYSICAL PROPERTIES

Composite

A' = 283 in.²	S'b = 836 in.³
I' = 7164 in.⁴	S't = 1649 in.³ (At Top of SpanDeck)
Y'b = 7.65 in.	S'tt = 1129 in.³ (At Top of Topping)
Y't = 4.35 in. (To Top of SpanDeck)	Wt. = 410 PLF
Y'tt = 6.35 in. (To Top of Topping)	Wt. = 102.5 PSF



12" SPANDECK CROSS SECTION

UL FIRE RATED J917

DESIGN DATA

1. Precast Strength @ 28 days = 5000 PSI.
2. Precast Density = 150 PCF.
3. Strand = 1/2" ϕ, 270 K Lo-Relaxation.
4. Composite Strength = 3000 PSI.
5. Composite Density = 150 PCF.
6. Strand Height = 1.5 in.
7. Ultimate moment capacities (when fully developed)...
   4 – 1/2"ϕ, 270K = 148.3'K
   6 – 1/2"ϕ, 270K = 208.1'K
8. Maximum bottom tensile stress is $6\sqrt{f'c}$ = 424 PSI.
9. All superimposed load is treated as live load in the strength analysis of flexure and shear.
10. Flexural strength capacity is based on stress/strain strand relationships.
11. Load values to the left of the solid line are controlled by ultimate strength. Load values to the right are controlled by service stress.
12. Shear values are the maximum allowable before shear reinforcement is required.
13. Deflection limits were not considered when determining allowable loads in this table.
14. All loads shown refer to allowable loads applied after the topping has hardened.

12" SPANDECK W/2" TOPPING	ALLOWABLE SUPERIMPOSED LOAD (PSF)																						
STRAND PATTERN	SPAN (FEET)																						
	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
Flexure 4 – 1/2"ϕ	446	382	343	303	271	240	214	190	170	151	134	120	108	94	83	73	64						
Shear 4 – 1/2"ϕ	436	405	379	357	335	308	281	257	234	214	183	181	172	162	148	130	123						
Flexure 6 – 1/2"ϕ	571	563	527	470	421	378	340	307	277	251	227	208	187	170	154	140	127	116	104	94	85	76	66
Shear 6 – 1/2"ϕ	463	423	397	373	351	331	313	297	282	268	255	237	218	201	186	171	158	146	135	124	118	113	108

NITTERHOUSE CONCRETE PRODUCTS

This table is for simple spans and uniform loads. design data for any of these span–load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths.

2655 MOLLY PITCHER HWY. SOUTH, BOX N
CHAMBERSBURG, PA 17201-0813
717-267-4505 • FAX: 717-267-4518

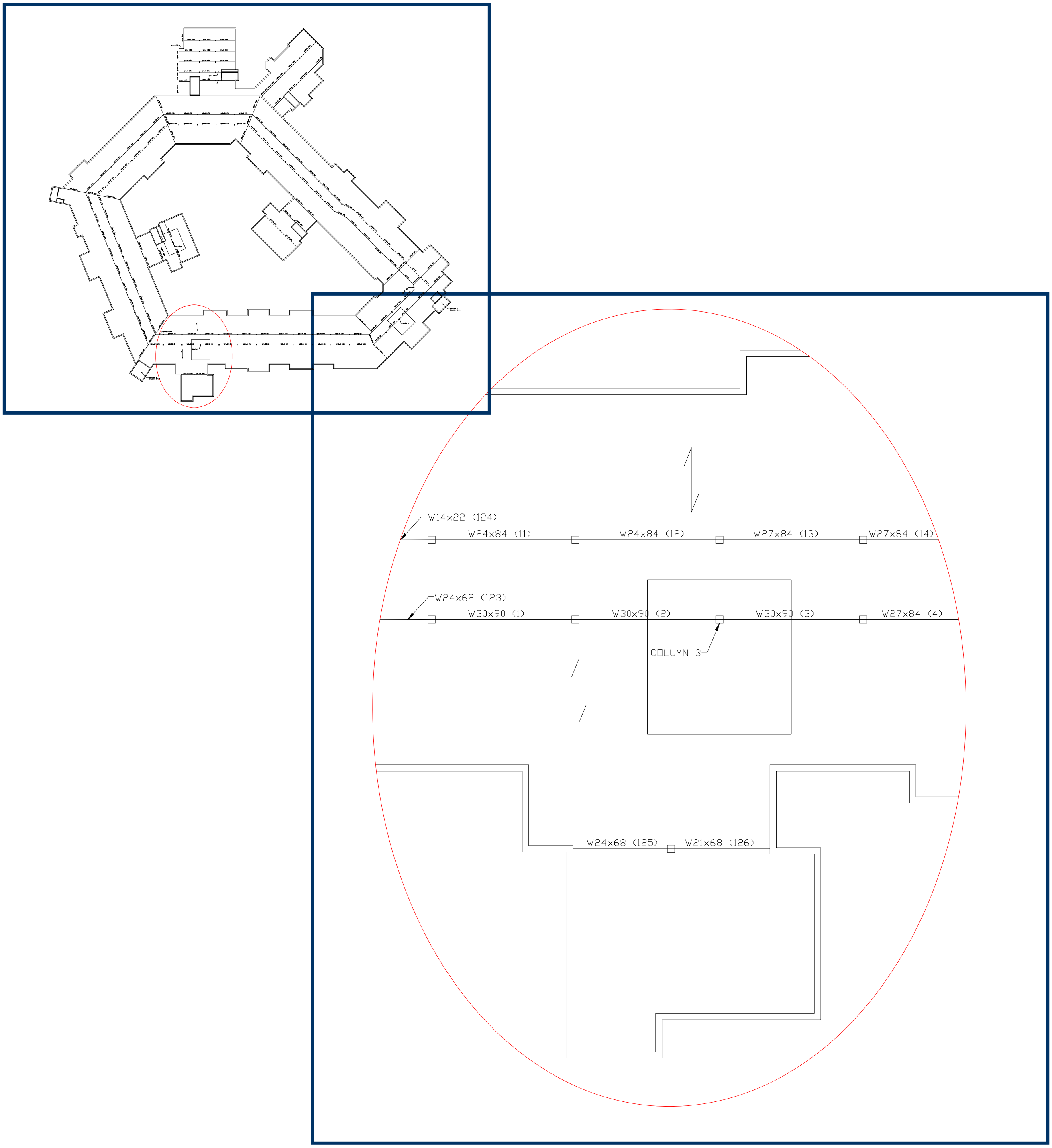
REVISED 12/02

Section of First Floor - Alternate System

Beam #	Span (ft)	Trib Width (ft)	w (lb/ft) factored	w (k/ft) factored	Mu ('k)	Vu (k)
1	22.67	23.96	15685.664	15.685664	1007.664506	177.7970014
2	22.67	23.96	15685.664	15.685664	1007.664506	177.7970014
3	22.67	23.96	15685.664	15.685664	1007.664506	177.7970014
4	22.67	20.125	13544.2	13.5442	870.0944759	153.523507
11	22.67	17.625	12148.2	12.1482	780.4138829	137.699847
12	22.67	17.625	12148.2	12.1482	780.4138829	137.699847
13	22.67	20.625	13823.4	13.8234	888.0305945	156.688239
14	22.67	20.625	13823.4	13.8234	888.0305945	156.688239
123	18.17	20.12	13541.408	13.541408	558.8351195	123.0236917
124	9.8	13.6	9900.64	9.90064	118.8571832	48.513136
125	15.42	33.875	21222.2	21.2222	630.7672895	163.623162
126	15.58	30.875	19547	19.547	593.0960514	152.27113

w (lb/ft) (unfactored)	w (k/ft) (unfactored)	L/240	L/360	I req'd	Designed Beam
12751.92	12.75192	1.1335	0.755666667	2305.391331	W30x90
12751.92	12.75192	1.1335	0.755666667	2305.391331	W30x90
12751.92	12.75192	1.1335	0.755666667	2305.391331	W30x90
11018.5	11.0185	1.1335	0.755666667	1992.010174	W27x84
9888.5	9.8885	1.1335	0.755666667	1787.719981	W24x84
9888.5	9.8885	1.1335	0.755666667	1787.719981	W24x84
11244.5	11.2445	1.1335	0.755666667	2032.868213	W27x84
11244.5	11.2445	1.1335	0.755666667	2032.868213	W27x84
11016.24	11.01624	0.9085	0.605666667	1025.445743	W24x62
8069.2	8.0692	0.49	0.326666667	117.8482731	W14x22
17233.5	17.2335	0.771	0.514	980.4853873	W24x68
15877.5	15.8775	0.779	0.519333333	931.749133	W21x68

Wellington at Hershey's Mill



W14x22 (124)
W24x84 (11)
W24x84 (12)
W27x84 (13)
W27x84 (14)
W24x62 (123)
W30x90 (1)
W30x90 (2)
W30x90 (3)
W27x84 (4)
COLUMN 3
W24x68 (125)
W21x68 (126)

Concrete Column Design

Column 3 Example Design Tributary width = 24.21' Tributary area = 548.72 ft^2

Roof loads: plank, walls, + gravity

DL = 77.5 psf(24.21') + 62 psf (11') = 2558.275 plf

SL = 22 psf(24.21') = 532.62 plf

LL = 20 psf(24.21') = 484.2 plf

Total = 1.2D + 1.6L + 0.5S = 4164.222 plf (22'-8") = 94389.032 lb = ***94.4 k***

2^{nd} & 3^{rd} floor loads: plank, walls, + gravity

DL = (27 + 102.5)(24.21') + 62 psf (10') = 3755.195 plf

LL = 40psf (24.21') = 968.4 plf

Total = 1.2D + 1.6L = 6055.674 plf (22'-8") = 137261.944 lb = ***137.3 k***

1^{st} floor loads: plank + gravity

DL = 27 psf + 102.5 psf = 129.5 psf

LL = 40 psf

Total = 1.2D + 1.6L = 219.4 psf (548.72 ft^2) = 120389.168 lb = ***120.4 k***

Load from beams:

90 plf (22'-8") = 2040 lb = ***2.04 k***

Total point load on column 3 = 94.4 + 137.3(2) + 120.4 + 2.04(2) = 493.48 k

Column Design: 14" x 14" SQUARE TIED COLUMN WITH 4-#10's

3-12

CONCRETE REINFORCING STEEL INSTITUTE

SQUARE TIED COLUMNS 10″ × 10″

Short columns – no sidesway
Bars symmetrical in 4 faces

f'_c = 4,000 psi f_y = 60,000 psi
ϕM in inch-kips ϕP in kips

BARS	RHO	Max Cap		0% f_y		25% f_y		50% f_y		100% f_y		.1f'_c Ag		Zero Axial Load
		ϕM	ϕP	ϕM	ϕP	ϕM	ϕP	ϕM	ϕP	ϕM	ϕP	ϕM	ϕP	ϕM
4-# 5	1.24	215	230	335	183	378	152	396	127	409	86	315	40	254
4-# 6	1.76	223	246	363	192	410	158	432	129	454	81	371	40	335
4-# 7	2.40	233	266	397	203	447	164	474	131	507	75	438	40	429
4-# 8	3.16	244	291	435	217	488	172	521	134	565	67	513	40	534
4-# 9	4.00	255	317	472	231	530	180	569	136	625	57	592	40	643
4-#10	5.08	269	351	516	248	580	190	626	138	697	44	689	40	776
4-#11	6.24	277	388	538	260	601	193	648	132	723	20	710	40	879
8-# 5	2.48	220	269	369	207	420	169	445	136	471	79	420	40	449
8-# 6	3.52	230	302	411	226	468	181	500	142	540	70	502	40	572
8-# 7	4.80	244	343	461	250	524	195	563	148	619	59	597	40	700
8-# 8	6.32	260	391	517	277	585	211	633	155	707	44	703	40	841
SQUARE TIED COLUMNS 12″ × 12″														
4-# 6	1.22	380	330	570	272	663	228	714	192	750	134	561	58	442
4-# 7	1.67	395	350	618	283	721	236	779	196	832	130	654	58	572
4-# 8	2.19	412	374	673	296	787	245	854	201	925	124	760	58	721
4-# 9	2.78	430	401	732	311	859	256	932	206	1023	118	874	58	878
4-#10	3.53	451	435	804	331	947	270	1027	212	1142	109	1016	58	1072
4-#11	4.33	465	472	870	349	1002	277	1086	210	1215	92	1133	58	1241
4-#14	6.25	511	559	1031	400	1173	306	1280	219	1459	62	1449	58	1668
8-# 5	1.72	375	353	576	288	674	239	728	200	773	134	646	58	598
8-# 6	2.44	392	386	635	308	746	253	814	208	877	128	766	58	803
8-# 7	3.33	412	426	706	331	832	270	912	217	1000	121	905	58	1031
8-# 8	4.39	436	474	787	360	931	290	1024	227	1140	112	1062	58	1242
8-# 9	5.56	462	528	873	392	1037	313	1140	237	1287	101	1227	58	1462
8-#10	7.06	494	596	980	432	1169	341	1283	255	1467	85	1430	58	1729
SQUARE TIED COLUMNS 14″ × 14″														
4-# 7	1.22	615	449	896	379	1062	317	1155	268	1246	191	918	78	715
4-# 8	1.61	640	473	968	392	1149	327	1257	274	1375	188	1055	78	907
4-# 9	2.04	665	500	1045	407	1242	338	1368	281	1512	183	1204	78	1112
4-#10	2.59	695	534	1141	426	1358	352	1507	289	1682	177	1390	78	1366
4-#11	3.18	716	571	1235	444	1464	364	1619	294	1804	164	1554	78	1603
4-#14	4.59	781	658	1454	494	1733	400	1912	310	2172	143	1984	78	2183
8-# 5	1.27	587	452	840	385	1000	322	1084	272	1162	196	925	78	745
8-# 6	1.80	612	485	916	405	1092	336	1193	281	1303	192	1086	78	1011
8-# 7	2.45	640	525	1007	429	1203	354	1325	291	1472	188	1273	78	1323
8-# 8	3.22	672	573	1112	458	1332	374	1479	303	1666	183	1485	78	1677
8-# 9	4.08	707	627	1225	490	1470	397	1644	317	1871	175	1708	78	1994
8-#10	5.18	751	695	1367	531	1644	427	1852	335	2127	165	1985	78	2361
8-#11	6.37	783	769	1500	570	1800	454	2020	346	2313	142	2210	78	2661

(1) "0% f_y" indicates zero tension in bars on the tension side, "50% f_y" indicates 50% f_y stress in bars on the tension side, and "100% f_y" indicates 100% f_y stress (i.e., balance point) in bars on the tension side.

2nd & 3rd floors – Masonry Empirical design

Exterior Bearing Walls

Floor level supported	Plank size (in)	self-weight (psf)	Total DL (psf)	Snow (psf)	LL (psf)	load (wall above) (plf)	Load (supported) (plf)	Estimated wall weight (plf)	Wall load (plf)	A/ft (in^2 / ft)	Wall stress (psi)
roof	12	77.5	92.5	22	20	-	3194.375	840	4034.38	128	31.51855
3	12+2	102.5	117.5		40	4034.375	3740.625	840	8615	128	67.30469
2	12+2	102.5	117.5		40	8615	3740.625	840	13195.6	128	103.0908

Average Tributary Width: 23.75'

Design: 8" grouted CMU (2500 psi), Type N Mortar (140 psi max wall stress)

Interior Bearing Walls

Floor level supported	Plank size (in)	self-weight (psf)	Total DL (psf)	Snow (psf)	LL (psf)	load (wall above) (plf)	Load (supported) (plf)	Estimated wall weight (plf)	Wall load (plf)	A/ft (in^2 / ft)	Wall stress (psi)
roof	12	77.5	92.5	22	20	-	3228	620	3848	96	40.08333
3	12+2	102.5	117.5		40	3848	3780	620	8248	96	85.91667
2	12+2	102.5	117.5		40	8248	3780	620	12648	96	131.75

Average Tributary Width: 35.75'

Design: 6" grouted CMU (4500 psi), Type N Mortar (200 psi max wall stress)

Table 1—Wall Lateral Support Requirements (ref. 1)

Construction	Maximum wall length-to thickness or height-to thickness ratio[a]
Bearing walls	
Solid or solid grouted	20
All other	18
Nonbearing walls	
Exterior	18
Interior	36
Cantilever Walls[b]	
Solid	6
Hollow	4
Parapets (8-in. (203-mm) thick min.)[b]	3

[a] Ratios are determined using nominal dimensions. For multiwythe walls where wythes are bonded by masonry headers, the thickness is the nominal wall thickness. When multiwythe walls are bonded by metal wall ties, the thickness is taken as the sum of the wythe thicknesses.

[b] The ratios are maximum height-to-thickness ratios and do not limit wall length.

Table 2—Maximum Wall Spans, ft (m)

Wall thickness, in. (mm)	6 (152)	8 (203)	10 (254)	12 (305)
Bearing walls				
Solid or solid grouted	10 (3.0)[a]	13.3 (4.1)	16.6 (5.1)	20 (6.1)
All other	9 (2.7)[a]	12 (3.7)	15 (4.5)	18 (5.5)
Nonbearing walls				
Exterior	9 (2.7)	12 (3.7)	15 (4.5)	18 (5.5)
Interior	18 (5.5)	24 (7.3)	30 (9.1)	36 (11)
Cantilever Walls[b]				
Solid	3 (0.9)	4 (1.2)	5 (1.5)	6 (1.8)
Hollow	2 (0.6)	2.6 (0.8)	3.3 (1.0)	4 (1.2)
Parapets[b]	1.5 (0.5)	2 (0.6)	2.5 (0.8)	3 (0.9)

[a] 6-in. (152-mm) thick bearing walls are limited to one story in height.

[b] For these cases, spans are maximum wall heights.

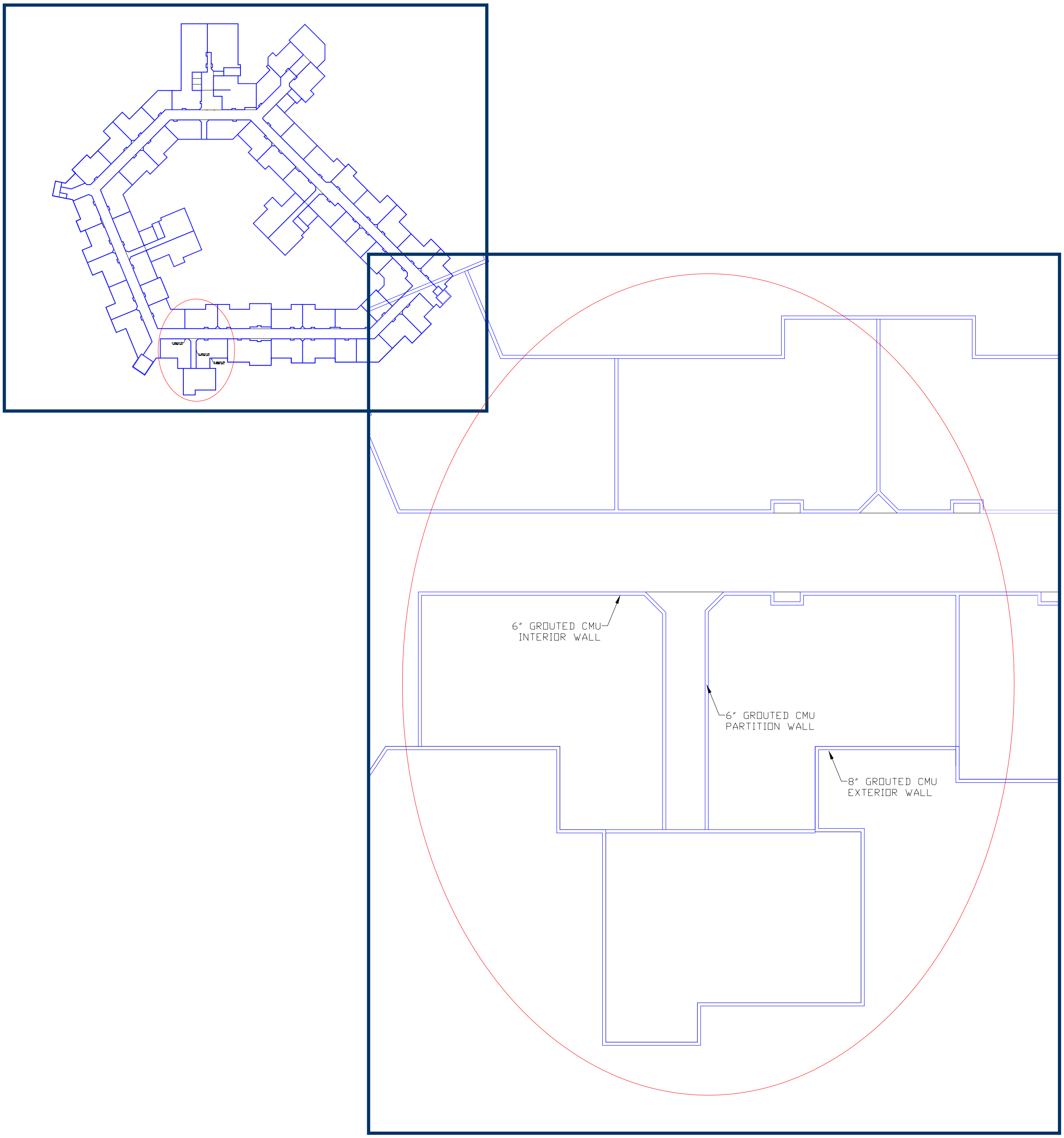
Table 3—Allowable Compressive Stress for Empirical Design of Masonry (ref. 1)

Gross area compressive strength of unit, psi (MPa)	Allowable compressive stresses based on gross cross-sectional area, psi (MPa)[a] Type M or S mortar	Type N mortar
Solid concrete brick:		
8000 (55) or greater	350 (2.41)	300 (2.07)
4500 (31)	225 (1.55)	200 (1.38)
2500 (17)	160 (1.10)	140 (0.97)
1500 (10)	115 (0.79)	100 (0.69)
Grouted concrete masonry:		
4500 (31) or greater	225 (1.55)	200 (1.38)
2500 (17)	160 (1.10)	140 (0.97)
1500 (10)	115 (0.79)	100 (0.69)
Solid concrete masonry units:		
3000 (21) or greater	225 (1.55)	200 (1.38)
2000 (14)	160 (1.10)	140 (0.97)
1200 (8.3)	115 (0.79)	100 (0.69)
Hollow concrete masonry units:		
2000 (14) or greater	140 (0.97)	120 (0.83)
1500 (10)	115 (0.79)	100 (0.69)
1000 (6.9)	75 (0.52)	70 (0.48)
700 (4.8)	60 (0.41)	55 (0.38)
Hollow walls (noncomposite masonry bonded[b])		
solid units:		
2500 (17) or greater	160 (1.10)	140 (0.97)
1500 (10)	115 (0.79)	100 (0.69)
hollow units	75 (0.52)	70 (0.48)

[a] Linear interpolation for intermediate values of compressive strength is permitted.

[b] Where floor and roof loads are carried on one wythe, the gross cross-sectional area is that of the wythe under load; if both wythes are loaded, the gross cross-sectional area is that of the wall minus the area of the cavity between the wythes. Walls bonded with metal ties shall be considered as noncomposite walls unless collar joints are filled with mortar or grout.

Section of First Floor - Alternate System



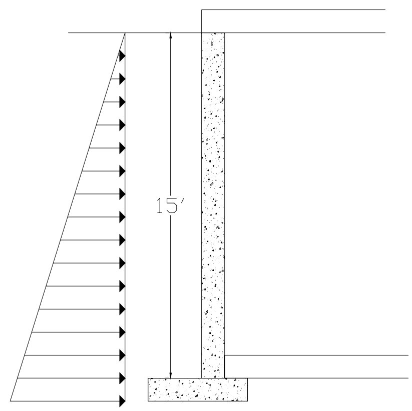
Foundation Wall Design

Floor height = 15' > 8' + Rigid Diaphragm → use @ Rest
γ_E = 57 pcf
γ = 125 pcf
$k_o \gamma = \gamma_E \rightarrow k_o$ = 57 pcf/125 pcf = 0.46
$k_o = 1 - \sin ø \rightarrow ø = 32.7^o$

<u>Design at full height retainage</u>

Assumptions: no cohesion or surcharge

P_{max} = 125 pcf (0.46)(15') = 862.5 psf
V_{max} = 2/3 (862.5(15'))/2 = 4312.5 plf
V_u = 4312.5 plf (1.6) = 6900 lbs = 6.9 k
M_{max} = 862.5 psf $(15')^2/(9\sqrt{3})$ = 12449.12'lbs.
M_u = 12449.12'lbs (1.6) = 19918.6'lbs = 19.9'k



<u>Unreinforced Cross Section</u>

$ø\, M_n = ø5\sqrt{f'_c}S$
19.9'k(12"/ft)(1000) ≤ $0.55(5)\sqrt{3000}(S_x)$
$S_x = 1585.41\ in^3 \rightarrow S_x = bh^2/6 = 12h^2/6$
h = 28.16" → *too large, reinforce wall*

<u>With Reinforcing</u>

$6900 = 0.75(2)\sqrt{3000}(12)d \rightarrow d = 6.999" \approx 7"$

Try h =12"

d = 12-1.5-0.25 = 10.25"

19.9'k(12"/ft) = $0.9A_s(60)(10.25" - 1.96A_s/2)$

238.8"k = $553.5A_s - 211.68A_s2 \rightarrow A_s \geq 0.545$ in2

Try #7s @ 12" → A_s = 0.60 in2

a = 1.96(0.60) = 1.176"

c = 1.176"/0.85 = 1.38"

ε_s = 0.003/1.38(10.25-1.38) = 0.0192 > 0.005 → OK

Use 12" concrete wall with #7s @ 12"

Appendix 3: Acoustic Analysis Calculations

Alternate System

	Sound Absorption					
Surface	**125 Hz**	**250 Hz**	**500 Hz**	**1000 Hz**	**2000 Hz**	**4000 Hz**
400 ft^2 ceiling, concrete	4	4	8	8	8	8
400 ft^2 floor, carpet, heavy, on foam rubber	32	96	228	276	284	292
800 ft^2 walls, gypsum board, 1 layer 5/8" thick (screwed to 1x3s, 16 oc with airspaces filled with fibrous insulation)	440	112	64	32	96	88
a_2 (sabins):	476	212	300	316	388	388

Original System

	Sound Absorption					
Surface	**125 Hz**	**250 Hz**	**500 Hz**	**1000 Hz**	**2000 Hz**	**4000 Hz**
400 ft^2 ceiling, gypsum board, 1/2 in thick	116	40	20	16	28	36
400 ft^2 floor, carpet, heavy, on foam rubber	32	96	228	276	284	292
800 ft^2 walls, gypsum board, 1/2" thick (nailed to 2x4s, 16 in oc)	232	80	40	32	56	72
a_2 (sabins):	380	216	288	324	368	400

	125 Hz	250 Hz	500 Hz	1000 Hz	2000 Hz	4000 Hz
Assuming the loudest noise to come from the garage would be a car stereo, the sound pressure level from the garage is:	72	83	82	82	80	75
Minus Background level in Apartments, RC-30:	45	40	35	30	25	20
Req'd NR (dB):	27	43	47	52	55	55
Alternative						
Minus 10log a_2/S:	0.755469614	-2.75724	-1.24939	-1.02373	-0.13228	-0.13228
Req'd TL (dB):	26.24453039	45.75724	48.24939	53.02373	55.13228	55.13228
Original						
Minus 10log a_2/S:	-0.22276395	-2.67606	-1.42668	-0.91515	-0.36212	0
Req'd TL (dB):	27.22276395	45.67606	48.42668	52.91515	55.36212	55

Comparison

Alternative

4 in reinforced concrete slab (54 lb/ft^2) (actual system: hollowcore plank system)

	125 Hz	250 Hz	500 Hz	1000 Hz	2000 Hz	4000 Hz
Req'd TL (dB):	26.24453039	45.75724	48.24939	53.02373	55.13228	55.13228
System TL (dB):	48	42	45	56	57	66

Original

18 in steel joists 16" o.c. with 1 5/8" concrete on 5/8 in plywood under heavy carpet laid on pad, and 5/8 in gypsum board attached to joists on ceiling side (20 lb/ft^2)
(actual system: 18 in steel joists 16" o.c. with 4" concrete slab, heavy carpet laid on pad and gypsum board attached to joists on ceiling side)

	125 Hz	250 Hz	500 Hz	1000 Hz	2000 Hz	4000 Hz
Req'd TL (dB):	27.22276395	45.67606	48.42668	52.91515	55.36212	55
System TL (dB):	27	37	45	54	60	65

Appendix 4: Building Envelope Analysis Calculations

Heat Transfer Rate (Heat loss)

Assumptions: 1-dimensional
Steady state
Constant properties

Original System

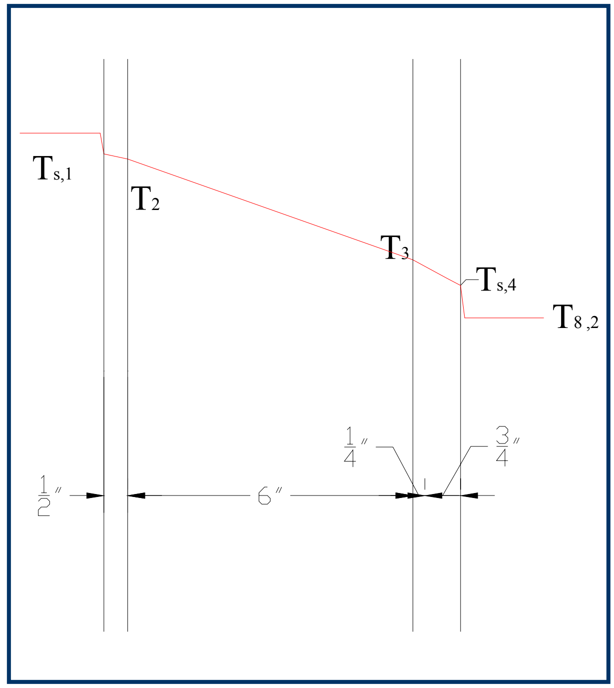
Wall area (A) = 100 ft^2

$T_{\infty,1}$ = 65ºF, $T_{\infty,2}$ = 21ºF

ΔT = 44ºF

U = 0.046 BTU/(hr-ft^2-ºF)

$q_x = UA\Delta T = 0.046(100)(44) = 202.4$ BTU/hr



Original System

Wall Details

Outside Surface Color .... **Light**
Absorptivity **0.450**
Overall U-Value .... **0.046** BTU/(hr-ft²-°F)

Wall Layers Details (Inside to Outside)

Layers	Thickness in	Density lb/ft³	Specific Ht. BTU / (lb - °F)	R-Value (hr-ft²-°F)/BTU	Weight lb/ft²
Inside surface resistance	0.000	0.0	0.00	0.68500	0.0
Gypsum board	0.500	50.0	0.26	0.44803	2.1
Air space	0.000	0.0	0.00	0.91000	0.0
R-13 batt insulation	6.000	0.5	0.20	19.23077	0.3
1-in stucco	1.000	116.0	0.20	0.19984	9.7
Outside surface resistance	0.000	0.0	0.00	0.33300	0.0
Totals	**7.500**	-		**21.80664**	**12.0**

Alternate System

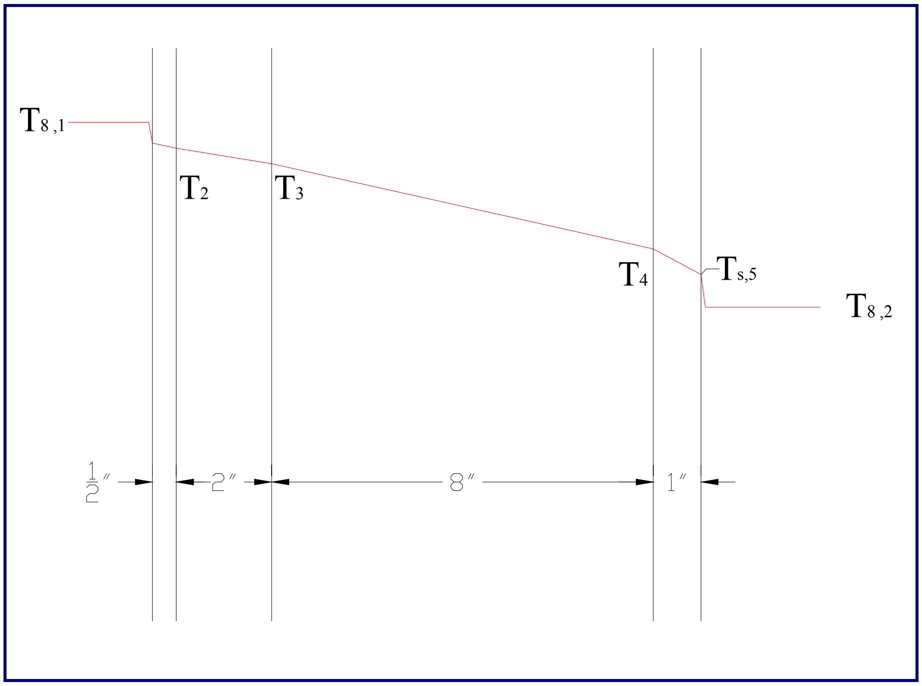
Wall area (A) = 100 ft^2

$T_{\infty,1}$ = 65ºF, $T_{\infty,2}$ = 21ºF

ΔT = 44ºF

U = 0.109 BTU/(hr-ft^2-ºF)

$q_x = UA\Delta T = 0.109(100)(44) = 479.6$ BTU/hr



Alternate System

Wall Details

Outside Surface Color .... **Light**
Absorptivity **0.450**
Overall U-Value .... **0.109** BTU/(hr-ft²-°F)

Wall Layers Details (Inside to Outside)

Layers	Thickness in	Density lb/ft³	Specific Ht. BTU / (lb - °F)	R-Value (hr-ft²-°F)/BTU	Weight lb/ft²
Inside surface resistance	0.000	0.0	0.00	0.68500	0.0
Gypsum board	0.500	50.0	0.26	0.44803	2.1
R-13 batt insulation	2.000	0.5	0.20	6.41026	0.1
8-in HW concrete block	8.000	61.0	0.20	1.11111	40.7
1-in stucco	1.000	116.0	0.20	0.19984	9.7
Outside surface resistance	0.000	0.0	0.00	0.33300	0.0
Totals	**11.500**	-		**9.18724**	**52.5**