Final Report

The Residences
Anne Arundel County, Maryland

4/7/2011
Faculty Advisor – Dr. Richard A. Behr
Ryan English-Structural Option

http://www.engr.psu.edu/ae/thesis/portfolios/2011/rje5020/index.html

The Residences at Arundel Preserve
Anne Arundel County, Maryland

Structural

The main structural system utilizes the Hambro joist system. This is a form of steel joist with a concrete slab on top.

The joist system is support by 6" light gage steel studs bearing walls.

The lateral loads are resisted by a light gage steel x bracing system.

Building Statistics

Occupancy: Mix use, Residential /Retail
Size: 300,000 gross s.f.
Height: 5 stories, 60 ft.
Dates of construction:
September 2009- February 2011

Project Team

Owner: Sumerset Construction
General Contractor/Developer:
Encore Developer
Architect: CE*X, Inc
Structural Engineer: Cates Engineering, Ltd.
Civil Engineer: Morris Ritchie Associates, Inc.
MEP Engineer:
Siegel, Rutherford, Bradstock Ridgway Inc.
Geotechnical: Geo-Technology Associates, Inc
Landscape Architect: The Faux Group, Inc

Architecture

"The Residences" is a new construction apartment and retail building, part of the Arundel Preserve Town Center Phase I project.

Along with the residential units the building also includeds a terrace level that contains a clubhouse, health center, and an outside pool.

Lighting/Electrical System

Electricity to the building is provided by two 277/480v 3 phase transformers, one for residential power and one for commercial power.

Fluorescents, compact fluorescents, and LED are used throughout the building.

Mechanical system

The hot and cold water is supplied by three centrifugal chiller units and by three natural gas hot water boilers.

Each of the apartment houses its own concealed modular type dual-coil vertical fan coil.

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Executive Summary

The redesign of the Residences compared a Two Way Concrete Slab (TWCS) design to a One Way Concrete Slab (OWCS) design. The slab thickness for the OWCS was determined to be 5" and was 10" for The TWCS. The OWCS was able to be integrated with the existing architectural design with minor architectural impact. As for the TWCS, to try and keep a square and regular bay, the system had more problems integrating with the existing architectural design. With keeping the floor to ceiling height as 24" as originally designed, the beams' minimal depth for the OWCS design reduced the space that could be utilized by other disciplines. Concrete shear walls were designed using the provisions and requirements from AIC 318-08. For the current location, ordinary reinforced concrete shear walls were designed, and for the high seismic location, special reinforced concrete shear walls were designed.

The use of Autodesk Robot Structural Analysis program was used throughout this thesis to analyze the redesign. This program was compared to SAP and was found that ARSA was similar in their basic elements but lacked the more advance features that SAP had.

For the green roof design, it was determined that most green roofs are comprised of three major layers: Vegetation, Growing Media, and Drainage. It was determined that grass would be able to resist the temperatures and the impact from occupants walking on it. The growing media was comprised of 50% -70% lightweight aggregate, 10%-20% organic material, and 20%-30% sand. A 2" drainage layer was determined to take any water that was not absorbed by the soil. Once the excess water was drained away, it could be collected and used for alternative uses.

A cost and schedule comparison was conducted for the OWCS and TWCS designs. It was determined that the OWCS would cost about $170.08 per s.f. and could be constructed in 375 days. The TWCS was found to be $162.78 per s.f. and could be completed in 262 days. This was compared to the original design of $182.96 per s.f. and 267 days, and found that the Two Way Concrete Slab was cheaper and could be constructed in the same time frame.

Acknowledgements

The author wishes to send thanks to the following professionals, architectural engineering faculty, and individuals for their assistance and generosity throughout the year with this thesis project.

Cates Engineering

Mike Stansbury

Tim Kowalcyk

Architectural Engineering

Dr. Richard A. Behr

Professor M. Kevin Parfitt

A special thanks to family, friends, and classmates because the accomplishments over the past five years could not have been possible without their support and friendship.

Introduction

Located in Anne Arundel County, Maryland, the Residences is a new construction apartment and retail building which is part of the Arundel Preserve Town Center Phase I Project (Figure 1). The Residences is a five to six story, 300,000 s.f., residential apartment building with 6,000 s.f. retail space surrounding a 5 story precast parking garage. This apartment building houses 242 upscale residential units consisting of studio, one and two bedroom layouts, and two level units. Along with the residential units, the building also includes a terrace level that contains a clubhouse, health center, and an outside pool. Construction of The Residences began in the fall of 2009 and should be completed in the beginning of 2011. It is owned and managed by the Somerset Construction Company and was designed by KTGY, Vienna, VA.

The structure of The Residences is comprised of the Hambro floor system, which uses a steel bar joist that supports a concrete slab (Figure 2). The floor systems are supported by a 6" light gage metal studs bearing and shear walls located throughout the building. A more in-depth structural analysis and details will follow in this report.

Figure 1: Site plan: Light Brown area represents the building. Gray area represents the parking garage. (Construction documents by Cates Engineering).

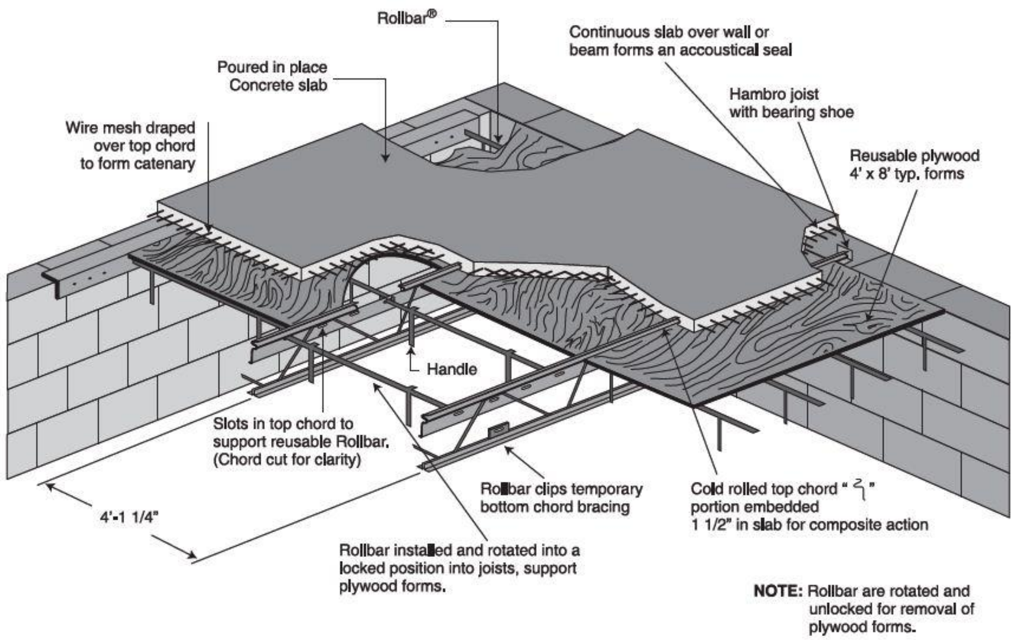


Figure 2: Hambro floor joist system. (Hambro Joist Company).

Structural System

Foundation System

According to the geotechnical report, the building rests on silt-clay facies[1] which is identified as clay, silt, and subordinate fine to medium grain muddy sand. The groundwater table is a minimum of 24 feet below existing grade, which is well below the foundation of the building. From the report, it is determined that the structures can be supported on shallow spread footings with an allowable bearing pressure of 5,000 pounds per square foot.

The building foundation system uses a 3'-0" wide strip footing with 3'-0"x3'-0" to 15'-0"x15'-0" column footing pads located mainly around the retail space and clubhouse area (Figure 3). The concrete slab on grade is 4" thick reinforced with 6 x 6 W1.4 xW1.4 welded wire fabric. All foundation concrete is to be 3,000 psi at 28 day strength.

[1] In geology, facies are bodies of rock with specified characteristics.

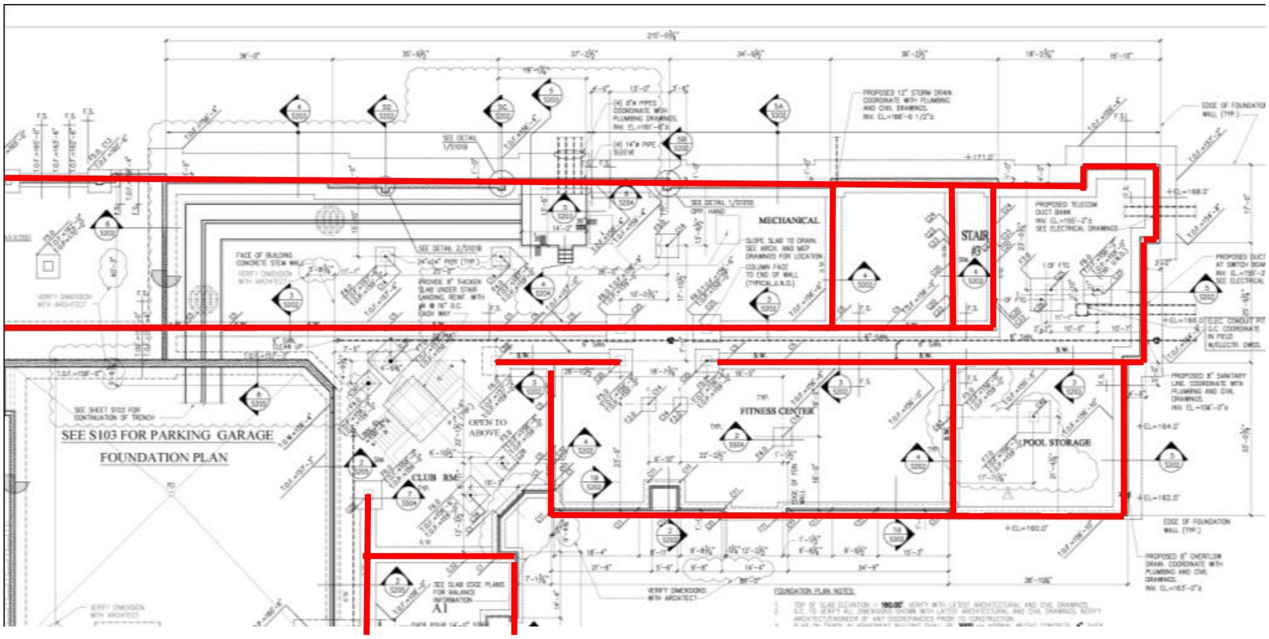


Figure 3: Foundation plan, part of the east wing. (Construction documents by Cates Engineering).

The floor system for the Residence is the Hambro floor joist system (Figure 2). The Hambro floor system uses a specially designed steel bar joist with a “S” shape top compression chord that serves three functions, a compression member in the non-composite joist during the construction stage, a chair for the welded wire fabric, and a continuous shear connection for the composite (cured concrete) stage. Detail information of the “S” shape top chord can be seen in Figure 4. The floor slab is a 3” thick 3,000 psi concrete with 6 x 6 W2.9 x W2.9 welded wire fabric. This particular floor thickness is chosen to give the system a 2 hour fire rated system. The slab is then supported by a 20” deep Hambro bar joist.

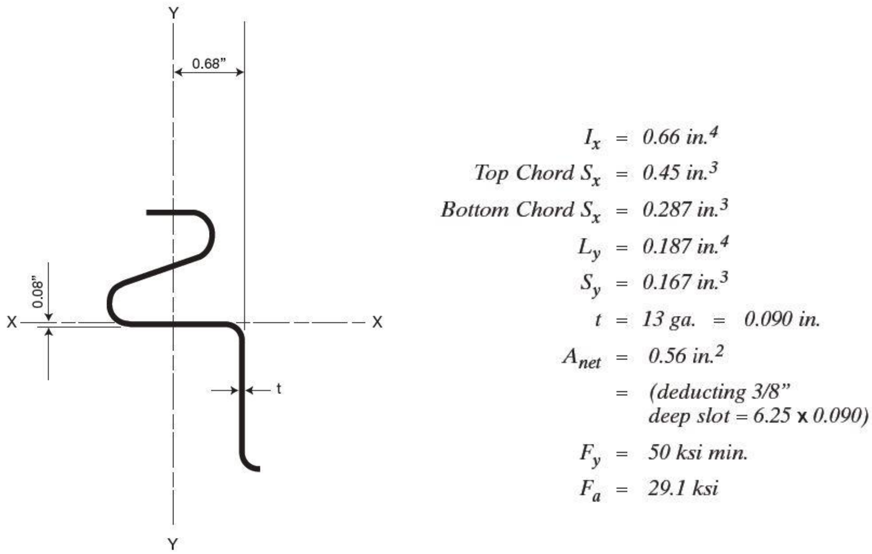
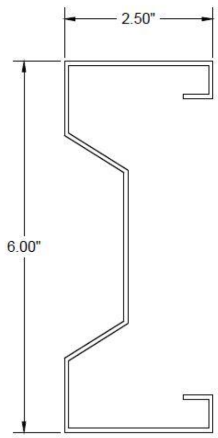


Figure 4: Top chord of the Hambro joist "S" chord with section properties.

Framing System

The design framing system in the Residences is light gage steel load bearing walls that are used to support the Hambro floor system and gravity loads in the building. The particular system uses the SigmaStud® load bearing light gage steel stud, a product of The Steel Network Company. The stud design is engineered to have a significant increase in load capacity when compared to the conventional “C” shaped studs. The Residences uses a 6” wide 18 gage stud with a flange length of 2.5”, as detailed in Figure 5. The exterior wall and interior corridor walls of the Residences are the primary bearing walls in the building. Figure 6 shows the location of the bearing walls in the building. Floor plans can be found in Appendix A.



$A=0.772\ in^2$

$I_x=4.183\ in^4$

$I_y=0.513\ im^4$

$F_y=50\ ksi$

$r_x=2.328\ in$

$r_y=0.815\ in$

$E=29,000\ ksi$

Figure 5: Section of light gage steel stud with section properties.

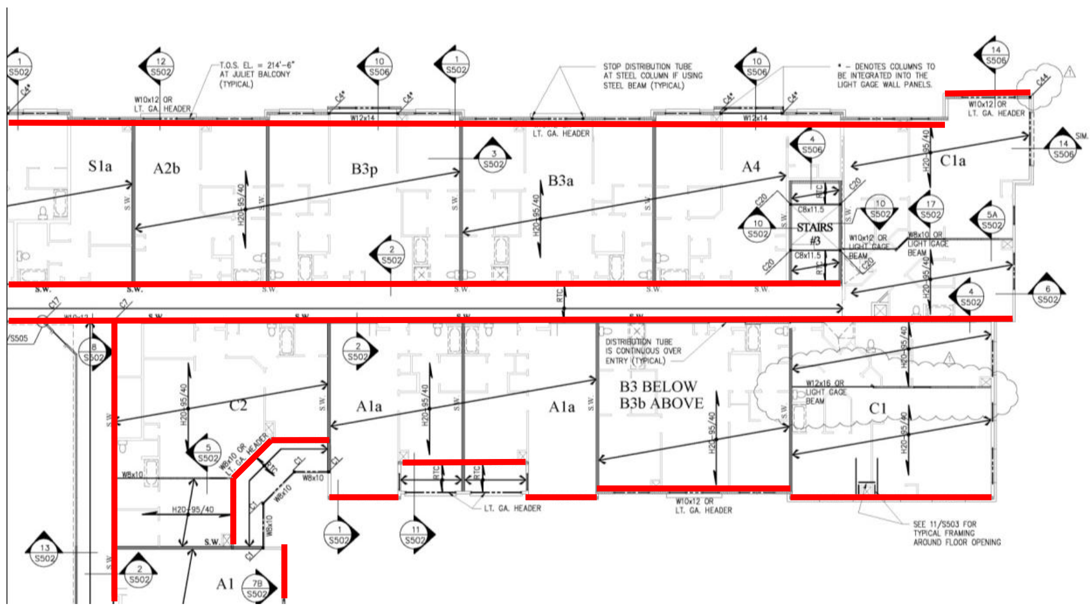


Figure 6: Location of bearing walls. (Construction documents by Cates Engineering).

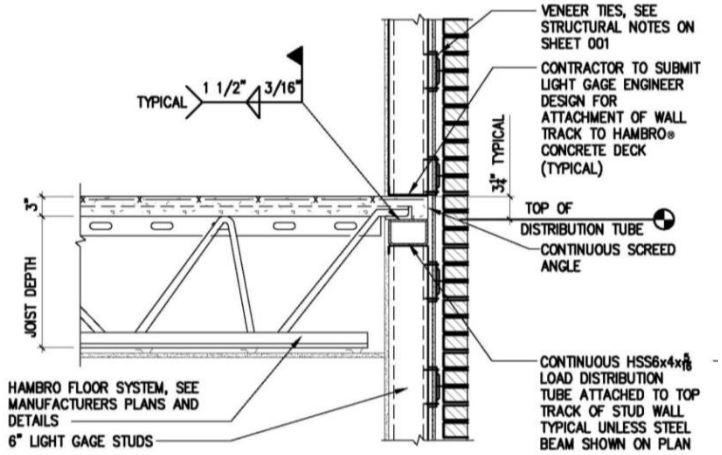


Figure 7: Exterior wall framing details. (Construction documents by Cates Engineering).

Lateral System

The lateral system in the Residences is a light gage shear wall system designed and engineered by The Steel Network Company. The system utilizes light gage 50 ksi steel hot dipped galvanized coated straps on both sides of the wall for shear resistance. A 6” wide flat strap is used in the lateral system of the Residences. (See Figure 8 for a typical framing detail). The shear walls are located all throughout the building (Figure 9), with most of the shear wall located in the corridor walls and the walls separating adjacent apartments.

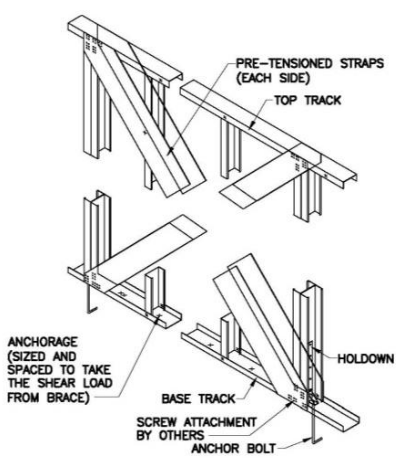


Figure 8: Lateral resistance system. (Construction documents by Cates Engineering).

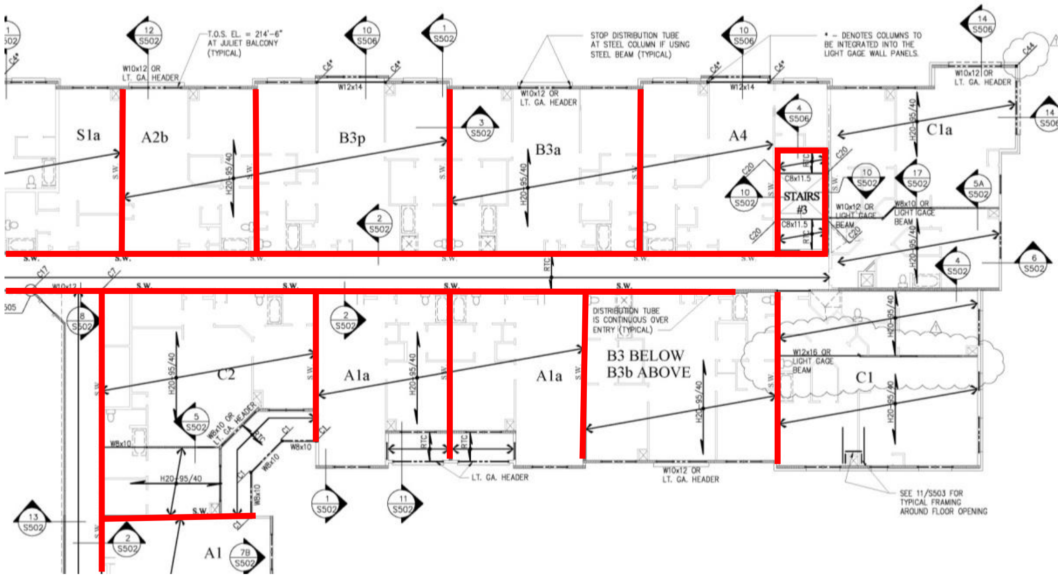


Figure 9: Location of the shear walls. (Construction documents by Cates Engineering).

Roof System

The roof system is the same, Hambro system, which is used for the floors throughout the building. The roof slab is 3" thick 3,000 psi concrete with 6 x 6 W2.9 x W2.9 welded wire fabric and is supported by a 20" deep Hambro joist.

Materials Used

Table 1: Materials Used For Thesis Design

Concrete		
Floor Slab	Normal Weight	f'c=4,000 psi
Roof Slab	Normal Weight	f'c=4,000 psi
Columns	Normal Weight	f'c=4,000 psi
Shear Wall	Normal Weight	F'c=4,000 psi
Reinforcement		
Deformed Bars	ASTM A-615	Grade 60
Welded Wire Fabric	ASTM A-185	

Codes and References

Design Codes

National Model Code:

2006 International Building Code

Design Codes:

Steel Construction Manual 13th Edition, AISC

American Iron and Steel Institute (AISI) 2008 Design of Cold Formed Steel Structural Members

American Concrete Institute (ACI) ACI 530-05, Building Code Requirements for Masonry Structures

American Concrete Institute (ACI) ACI 318-08, Building Code Requirements for Structural Concrete

Structural Standards:

American Society of Civil Engineers (ASCE), ASCE 7-05, Minimum Design Loads for Buildings and Other Structures

Thesis Codes

National Model Code:

2006 International Building Code

Design Codes:

American Concrete Institute (ACI) ACI 318-08, Building Code Requirements for Structural Concrete

Structural Standards:

American Society of Civil Engineers (ASCE), ASCE 7-05, Minimum Design Loads for Buildings and Other Structures

Load Analysis

Gravity Load

For this design, the use of the ASCE7-05 design live loads will be used. A roof live load of 100 psf was selected to allow the green roof to be accessible by the occupants. Design live load can be found in Table 2. Dead loads were found from a series of sources including, but not limited to, ASCE7-05 and manufacturer specifications. Design dead load can be found in Table 3.

Table 2: Design Live Loads

Location	Design (psf)	ASCE7-05 (psf)
Roof With Green Roof	100	20
Living	40	40
Corridors Exit stairs	100	100
Light Storage	125	125

Table 3: Design Dead Loads

Location	Design (psf)
Green Roof	72.5
Superimposed Dead Load	15
Self Wight (Concrete)	150 pcf

Snow Load

Due to the current location of this building being a snow region, snow loads are calculated in accordance to ASCE7-05 section 7. The high seismic region has no ground snow load. No snow load will be calculated for this region. The results of the load calculation can be seen in Table 4. Detail calculations and notes are included in Appendix B.

Table 4: Snow Loads

	Current Location
Ground Snow Load	Pg= 30 psf
Flat Roof Snow Load	Pf= 21 psf
Sloped Roof Snow Load	Ps= 21 psf

Wind Load

For this report, the wind load is analyzed for a smaller portion of the building to simplify the analysis of the lateral system. This can be done because of a building expansion joint that exist which can be seen in Figure 10. The calculation and values of the loads can be found is Table 5. The wind load was determined not to be the controlling lateral load. Detail calculations can be seen in Appendix C.

Table 5: Story Forces Due To Wind

	N-S Direction	E-W Direction
Gourd	11.8 kip	11.6 Kip
Second	13.4 Kip	13.2 kip
Third	15.4 Kip	15.1 kip
Fourth	15.0 Kip	14.7 kip
Fifth	21.6 kip	21.2 kip
Roof	8.1 kip	7.97 kip
Base Shear	123.6 kip	121.6 kip

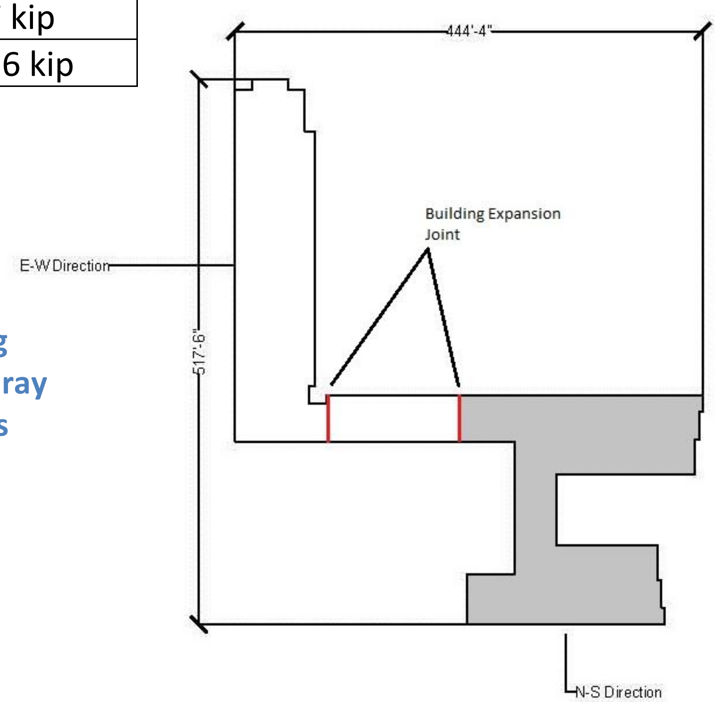


Figure 10: Building expansion joint. Gray region is what was redesigned.

Seismic Load

For this report, the seismic load is analyzed for a smaller portion of the building to simplify the analysis of the lateral system. This can be done because of a building expansion joint that exists which can be seen in Figure 10. The current location of the building is located in Anne Arundel County, Maryland, and a high scenic region was selected to be in south central California. The equivalent lateral force analysis was performed for the current location and because of the seismic design class of D for the high seismic region a modal response spectrum analysis had to be performed. Also, a modal response spectrum analysis was performed for the current location to check the values from the equivalent lateral force analysis. The calculation and values of the loads can be found is Tables 6-10 with detail information in Appendix D. Table 11 shows the maxing story drifts and total drift allows by code.

Table 6: Story Weights

Story Level	High	One Way Slab	Two Way Slab
Ground	11’	2995 kip	4633 kip
Second	22’	2995 kip	4633 kip
Third	33’	2995 kip	4633 kip
Fourth	44’	2995 kip	4633 kip
Fifth	55’	2995 kip	4633 kip
Roof	67’	4997 kip	6541 kip

Table 7: Seismic Load Current Location One Way Concrete Slab

Story Level	Lateral Force (kip)	Story Shear (kip)	Moments (Kip-ft)
Ground	32.5	818.8	357.9
Second	65.1	786.3	1431.7
Third	97.6	721.2	3221.3
Fourth	130.2	623.6	5726.8
Fifth	162.7	493.4	8948.1
Roof	330.7	330.7	22158.2
	818.80		41,844.13

Table 8: Seismic Load High Seismic Location One Way Concrete Slab

Story Level	Lateral Force (kip)	Story Shear (kip)	Moments (kip-ft)
Ground	131.7	3315.2	1449.2
Second	263.5	3183.5	5796.8
Third	395.2	2920.0	13042.8
Fourth	527.0	2524.7	23187.1
Fifth	658.7	1997.8	36229.9
Roof	1339.0	1339.0	89715.9
	3,315.22		169,421.70

Table 9: Seismic Load Current Location Two Way Concrete Slab

Story Level	Lateral Force (kip)	Story Shear (kip)	Moments (kip-ft)
Ground	51.6	1218.0	567.7
Second	103.2	1166.4	2270.9
Third	154.8	1063.1	5109.4
Fourth	206.4	908.3	9083.4
Fifth	258.1	701.9	14192.9
Roof	443.8	443.8	29735.6
	1,217.97		60,959.84

Table 10: Seismic Load High Seismic Location Two Way Concrete Slab

Story Level	Lateral Force (kip)	Story Shear (kip)	Moments (kip-ft)
Ground	209.0	4931.3	2298.5
Second	417.9	4722.3	9194.2
Third	626.9	4304.4	20686.9
Fourth	835.8	3677.5	36776.8
Fifth	1044.8	2841.7	57463.7
Roof	1796.9	1796.9	120392.7
	4,931.29		246,812.84

Table 11: Allowable Deflections

		Wind H/400		Seismic 0.02 H_{sx}	
	Story Height (ft)	Story Drift	Total Drift	Story Drift	Total Drift
Ground (1)	11	0.33”	0.33”	2.64”	2.64”
Second (2)	11	0.33”	0.66”	2.64”	5.28”
Third (3)	11	0.33”	0.99”	2.64”	7.92”
Fourth (4)	11	0.33”	1.32”	2.64”	10.56”
Fifth (5)	11	0.33”	1.65”	2.64”	13.20”
Roof (6)	12.67	0.38”	2.03”	3.04”	16.24”

Load Combination

Lateral load analysis is performed for this report and the load combinations that are provided by ASCE7-05 section2 that did not include lateral load forces is disregarded. It is also noted that the load combinations that includes the factor of .9D are used to calculate uplift forces for the later loads.

- 1.2D+1.6W+L+.5(Lr or S or R)
- 1.2D+1.0E+L+.2S
- .9D+1.6W+1.6H
- .9D+1.0E+1.6H

To determine the governing load case, it can be simplified to whether 1.6W+L is greater than 1.0E for the general loading conditions and whether 1.6W is greater than 1.0E for uplift. Since the seismic loads are much greater than the wind loads, it is safe to assumed that the 1.2D+1.0E+L+.2S and .9D+1.0E+1.6H are the controlling strength design for general loading and uplift respectively.

Proposal Thesis

Proposed Structural

The Residences is designed as a light gage metal studs bearing and shear walls which supports the Hambro floor system. In the analysis of the existing conditions of The Residences, it is found that the building did meet all structural codes and requirements. For the purposes of this thesis, The Residences will be re-evaluated using a one way and two way concrete floor system and different lateral systems.

The concrete system will be designed to support the gravity loads determined in the early technical reports. The existing building layout is used as a template to start the design process. Some variations may need to be implemented upon further analysis of the redesign. After the initial design is accomplished, the lateral loads will be determined and the lateral resisting systems will be designed.

The lateral loads will be compared between two locations: the current location of the building and a location in a high seismic region. Once the loads are determined, the lateral resisting systems will be designed. It is planned to perform research and design of seismic resistive systems to resist the lateral loads. A 3D model will be used to model the gravity and lateral system to aid in the design of the members and verify the accuracy of the design.

Breadth Options

In-Depth Cost and Schedule Impacts of Investigation

The first breadth study was chosen with its connection to the structural depth. The proposed changes to the floor system, superstructure, and lateral system will have an impact on the scheduling and cost of the building such as the scheduling changes that would involve the additional forming, placing, and shoring of the concrete. Also, the higher earthquake loads will have an impact on the cost of the building. Once the scheduling impact and cost changes are considered, the feasibility of redesigning The Residences as a concrete system will be evaluated.

Sustainability: Green Roof

To achieve a sustainable building, a green roof is going to be considered in place of the current rooftop. The design of the green roof is to consist of a study of the layers that make up the system and the flashing and membrane involved. Also, the green roof is to be designed with the intention of retaining water that can be used throughout the building. The drainage and flow of water to a central gray water collection tank is to be considered and designed. In addition, the green roof will be made accessible to the building’s occupants; thus, access to the green roof is to be designed. Finally, the loads from the green roof will be applied to the design of the gravity and lateral system.

Structural Design

Design Goals

The structural design goal of this project is to redesign The Residences to have a concrete super structure. The redesign will allow for a uniform structural system to be placed. Goals to be met throughout this project include:

- Compare the design of a One Way Concrete Slab and Two Way Concrete Slab
- Investigate the effects of having an increase of mass on the roof lever in high seismic region
- Not reduce the floor to ceiling height
- Minimizes architectural impact
- Use computer programs to aid in the design and analysis of the structural
- Evaluate the validity and ease of use of *Autodesk Robot Structural Analysis* program

Concrete Slab Design

One Way Concrete Slab

To minimize the architectural impact, the column layout for the One Way Concrete Slab was designed by overlaying the architectural floor plan and places the columns in a location that would not cause architectural changes. A column size of 14" x 14" was initially selected as a staring design and was later conformed to be able to support the loads; Figure 11 shows the location of the columns. To avoid changing the floor to ceiling height, the minimal beam thickness was selected in accordance to ACI 318-08 Table 9.5(a). The initial size of the beam was 20" and was later confirmed to be adequate to carry the loads. One design issue that was found was trying to keep enough space to allow for other disciplines to install equipment in the ceiling space. An initial slab thickness of 6" was calculated based on the span length and ACI 318-08 Table 9.5(a).

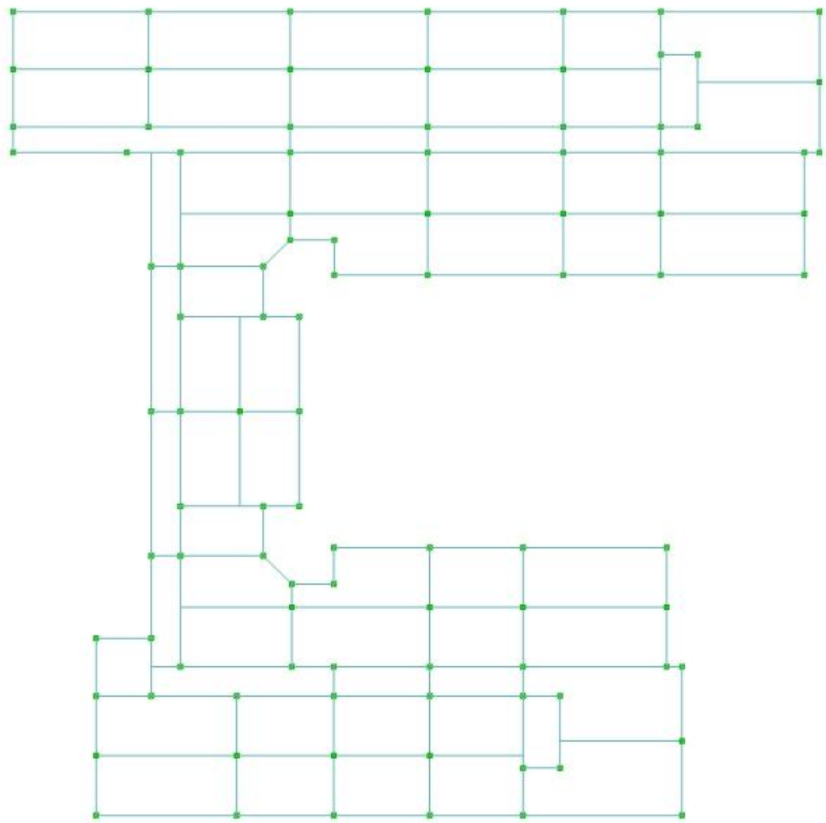


Figure 11: One Way Concrete Plan.

Once the initial design sizes of the member were selected, a 3D model was created using *Autodesk Robot Structural Analysis (ARSA)*. This program was chosen to compare its validity and ease of use to other structural analysis programs. The moment and shear values of key beam elements were determined from the 3D model and was used in hand calculations; example hand calculation can be seen in Appendix E. The hand calculations of the beam design were compared to ARSA and to *Structure Point Beam*. The values and design were found to be similar between the three. The column design was conducted in a similar fashion. The compress force and moment values were selected from a key column in the 3D model. The column was designed for three sections along its high. This was done to reduce the amount of rebar as the forces are reducing along the high. *Structure Point Column* was used to generate interaction diagrams for the column sections. This was compared to the ARSA output for column design. Figure 12 shows an interaction diagram of one of the three column sections. The two programs produced similar results for column design. The slab was calculated in the same way as a beam. A 1 ft. slice of the slab was considered in the calculation process. Further figures, diagrams, and calculations can be found in Appendix E.

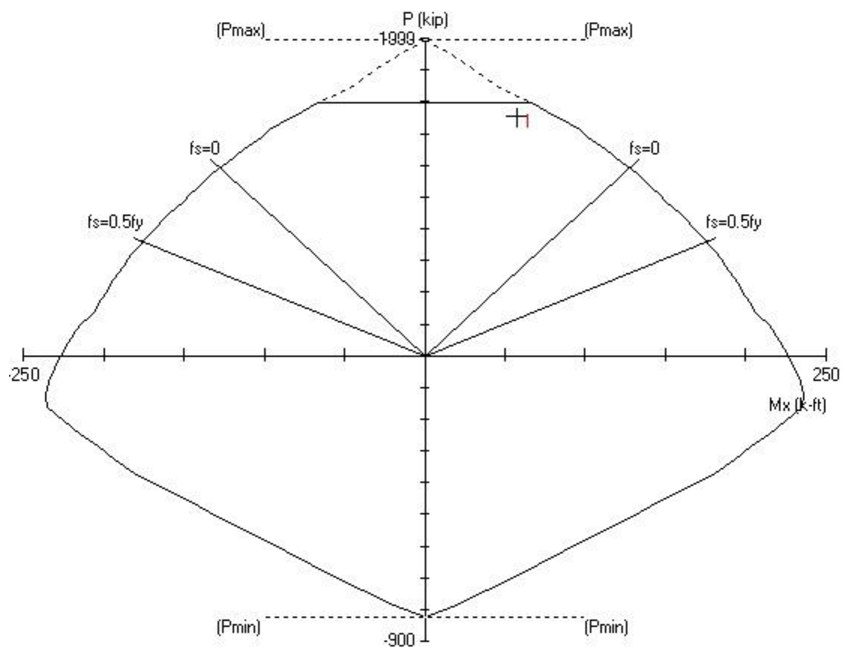


Figure 12: Column Interaction Diagram for the first two stories.

Two Way Concrete Slab

Column location was located by first selecting a square and regular bay size, and then it was compared to the architectural plans. The bay sizes were approximately 23' x 25'. It was noticed that to keep the square and regular bay size, minor architectural changes would need to be allowed. Figure 13 shows the location of the column. A column size of 18" x 18" was selected for the first two stories and 16" x 16" for all other stories and was later confirmed to be suitable to carry the load. The slab thickness was determined from ACI 318-08 Table 9.5(c), and the longest span length, the initial slab size, was found to be 10". Once the slab size and bay sizes were determined, the ACI 318-08 Direct Design method was used to determine the moments that the slab will support. The required reinforcement was determined by considering the slab as a beam with a thickness of 10" and width equal to the column or middle strip width. Next, one way shear and punching shear were calculated, and it was found that drop panels were needed to resist punching shear. Drop panels were selected instead of shear capitals for the added slab thickness reduction.

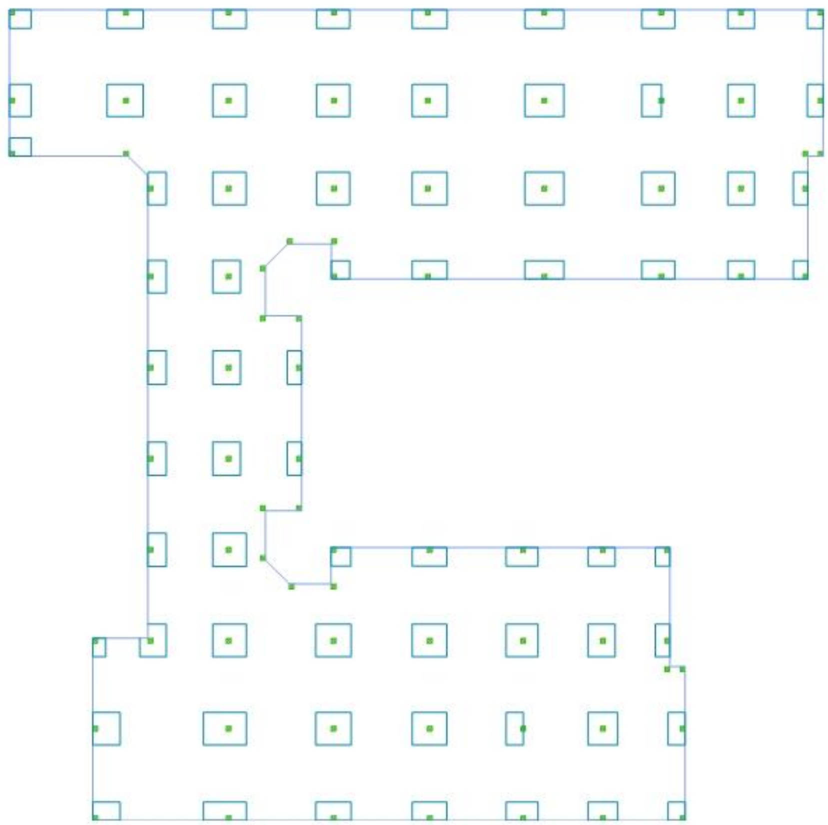


Figure 13: Two Way Concrete Plan.

Once again, when the initial design sizes of the member were selected, a 3D model was created using *Autodesk Robot Structural Analysis (ARSA).* The column design was conducted in a similar fashion to the one way concrete slab design. A key column in the 3D model was selected and force was determined. The column was design for three sections along its high. This was done again to reduce the amount of rebar as the forces are reducing along the high. *Structure Point Column* was used again to generate interaction diagrams for the column sections. Figure 14 shows an interaction diagram. *Structure Point Slab* was used to compare the values from the hand calculations and ARSA.

The hand calculation values were slightly higher than those from Structure Point Slab but were within acceptable limits. Further figures, diagrams, and calculation can be found in Appendix F.

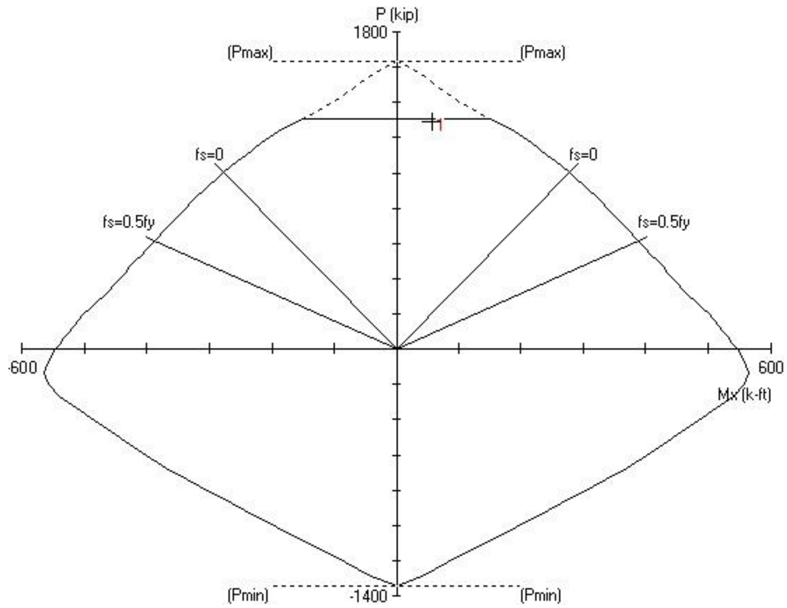


Figure 14: Column interaction diagram for the first two stories.

Shear Wall Design

Once the analysis of two seismic regions, the current building location seismic design category B, and a high seismic region seismic design category D were completed, the results were used to design a code lever ordinary reinforced concrete shear wall. Figures 15 and 16 show the location of the shear walls in the Two Way Concrete Slab design and the One Way Concrete Slab design. The ordinary reinforced concrete shear walls were not allowed to be designed for seismic design category D. Therefore, a special reinforced concrete shear wall was to be designed. ACI 318-08 has no requirements for shear walls in buildings assigned to SDC A, B, or C. For these buildings, ACI considers the requirements given in chapter 1 through 18 and chapter 22 to be adequate.

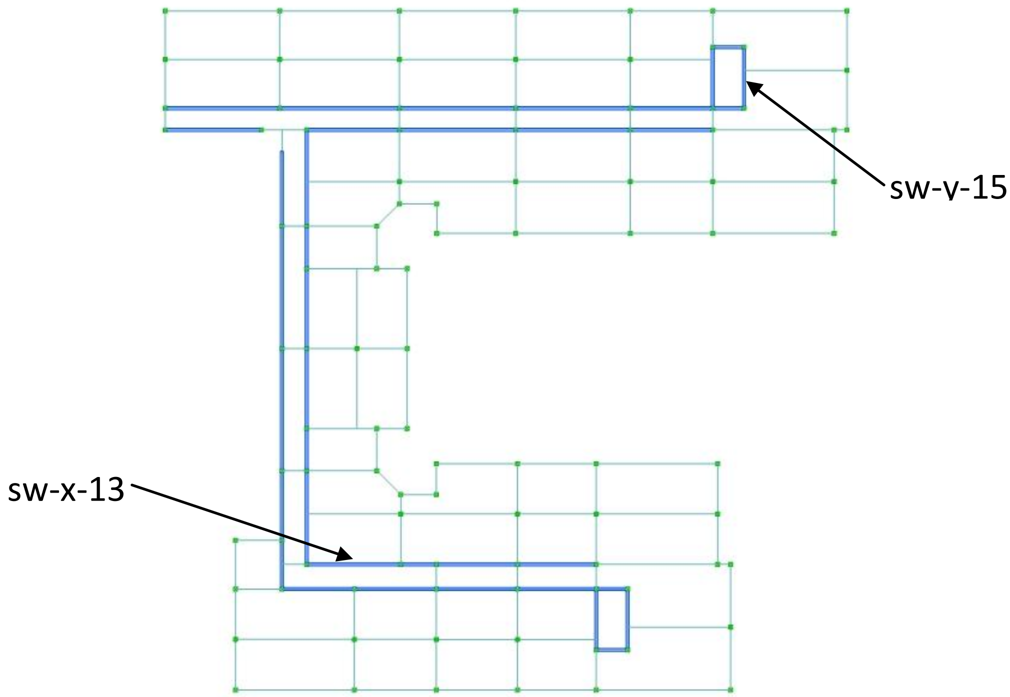


Figure 15: One way concrete shear wall location.

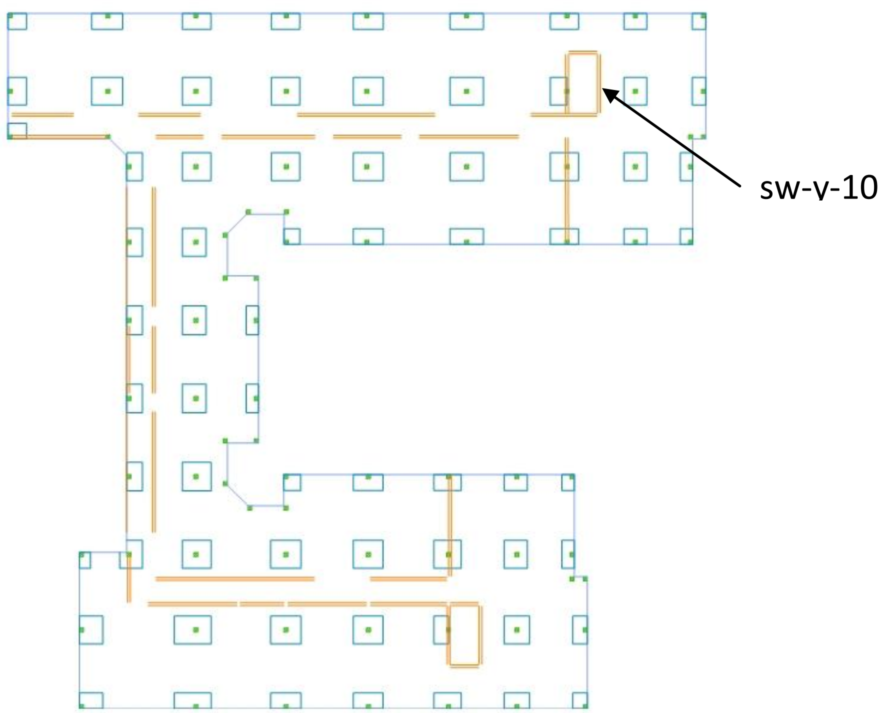


Figure 16: Two way concrete shear wall location.

The design procedure for these shear walls are a two part process. First, an axial load-moment interaction diagram was conducted on the given dimensions and concrete strength. Figure 17 shows an interaction diagram of a shear wall.

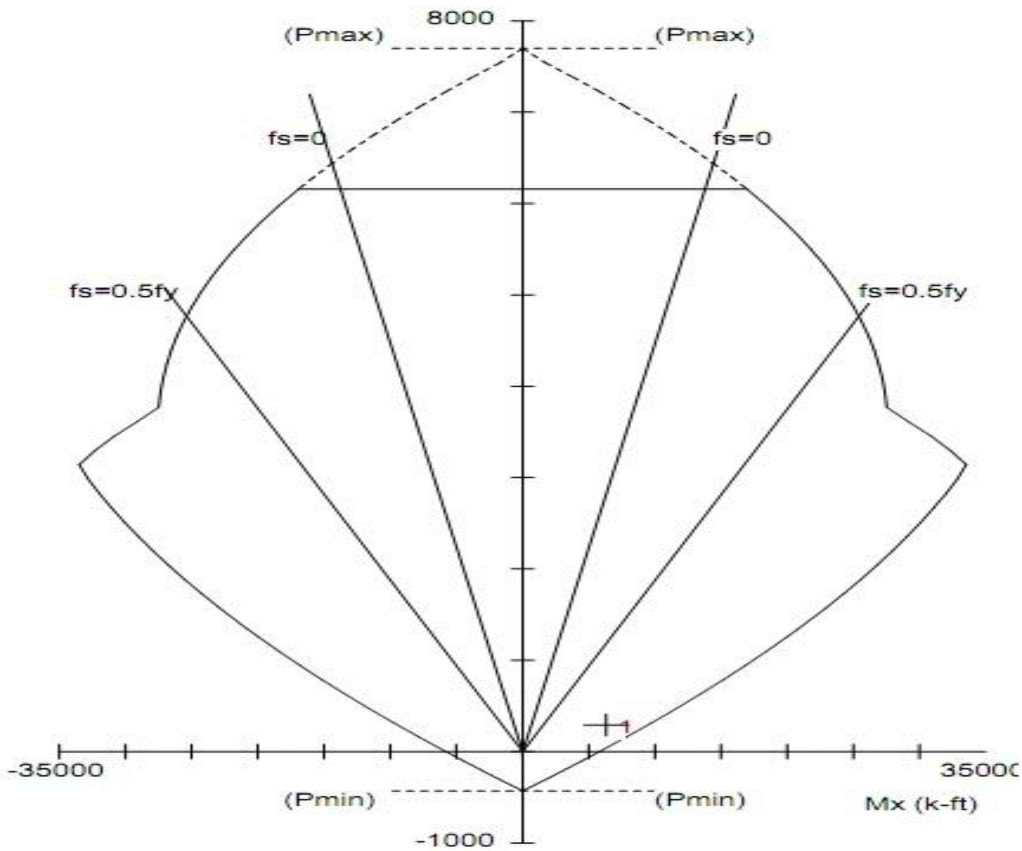


Figure 17: Interaction diagram of shear wall, one way x direction wall number 13.

The second part of the design was the selection of reinforcement that satisfies the design requirement under the loads and moments equal to or larger than the factored loads and moments. For shear walls in SDC other than A, B, or C, a more involved design procedure was required. Once the design loads and moments were calculated, the wall was designed for shear, combined axial load, and bending moment. An axial load-moment interaction diagram was also created for each shear wall. Next, the determination of boundary elements requirement using the displacement based methods was calculated for each shear wall. If it was found that boundary elements were required, then boundary elements were designed to the code. Computer models of each shear wall were created to determine the deflection and stress in the wall. Figure 18 shows an example of the model. Table 12 shows the calculated story drifts ratios and total story drift. Further figures, diagrams, and calculation can be found in Appendix G

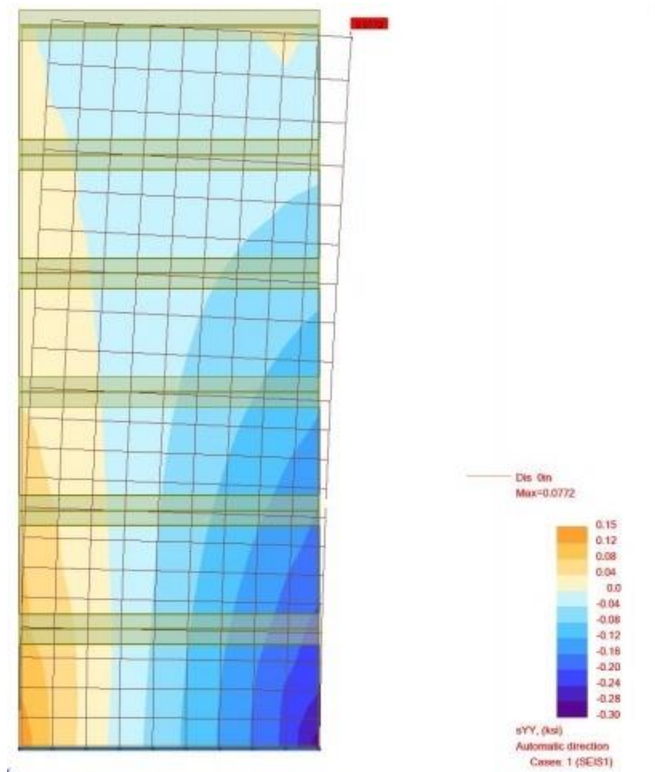


Figure 18: Maximum deflection and stress in one way x direction wall number 13

Table 12: Story Drifts.

One Way	Current location			
Story		Story Drift (in)	Drift Ratio	Total Drift (in)
1	0.033	0.150	0.11%	0.150
2	0.057	0.255	0.19%	0.405
3	0.076	0.340	0.26%	0.745
4	0.092	0.414	0.31%	1.160
5	0.105	0.471	0.36%	1.630
6	0.122	0.548	0.38%	2.178

One Way	High Seismic			
Story		Story Drift (in)	Drift Ratio	Total Drift (in)
1	0.141	0.705	0.53%	0.705
2	0.239	1.195	0.90%	1.900
3	0.319	1.596	1.21%	3.496
4	0.389	1.945	1.47%	5.441
5	0.442	2.210	1.67%	7.650
6	0.514	2.571	1.79%	10.221

Two Way	Current location			
Story		Story Drift (in)	Drift Ratio	Total Drift (in)
1	0.135	0.607	0.46%	0.607
2	0.191	0.861	0.65%	1.468
3	0.233	1.048	0.79%	2.515
4	0.249	1.120	0.85%	3.635
5	0.271	1.219	0.92%	4.854
6	0.305	1.371	0.95%	6.224

Two Way	High Seismic			
Story		Story Drift (in)	Drift Ratio	Total Drift (in)
1	0.300	1.349	1.02%	1.349
2	0.425	1.913	1.45%	3.262
3	0.517	2.328	1.76%	5.590
4	0.553	2.489	1.89%	8.079
5	0.562	2.528	1.92%	10.607
6	0.635	2.856	1.98%	13.463

Sustainability: Green Roof Design

Design Goals

The main design goal for the green roof design was to understand the layers and properties of the layers that make up a green roof. Other goals for the green roof design included but not limited to:

- Retain and collect rain water runoff.
- Gray water collection system.
- Accessibility to building occupants.

Green Roof Design

The green roof design first started by understanding the layers than design the green roof layers apparently. The typical layers in a green roof are: Vegetation, Growing Media, Filter Fabric, Drainage, Insulation, and Water Proofing. Figure 19 shows a section of the green roof. Once all the layers were determined, it was decided that there were three important sections of the green roof: Vegetation, Growing Media, and Drainage. Climate data for the past five years was located for the current location. It was found that the vegetation would need to survive temperatures as high as 100 °F and as low as 8°F. Along with temperature, the green roof needed to absorb a maximum around 3.0 in. and minimum of 0.0 in. of rain per day. Appendix H has more climate data for the location. From this data, it was determined that a grass or ground cover vegetation should be able to resist the extreme temperature.

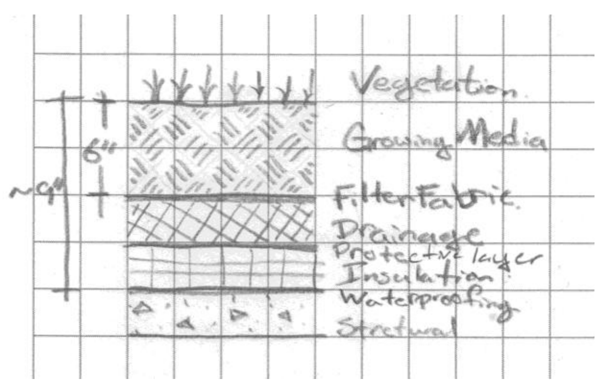


Figure 19: Layers of green roof.

An initial design of a 6 in. growing media was used. The larger than normal thickness was picked to encourage the roots to grow down away from the extremes of the surface environment. The growing media will contain 3 components: lightweight aggregate, organic material, and sand. With the building location in a humid region, the amount of organic material needs to be within 10% to 20% of the total weight. If too much organic matter is used, the volume of mix decreases due to decomposition and requires replacement due to the displacement of the media. Also, as the organic material breaks down, the fine filters out down to the filter fabric and decomposes further creating a slime which impedes the drainage causing the water to build up in the media. Table 13 shows the component content by volume.

Table 13: Components Content by Volume

Lightweight Aggregate	50%-70%
Organic Material	10%-20%
Sand	20%-30%

The drainage layer needs to be able to take all the water that is not absorbed by the vegetation and growing media. A 2” drainage layer will be adequate to take the rain water runoff. The water is to be collected in a collection take to be use for watering plants, cleaning lawn tools, and other uses. Figure 20 shows how the collection system would work.

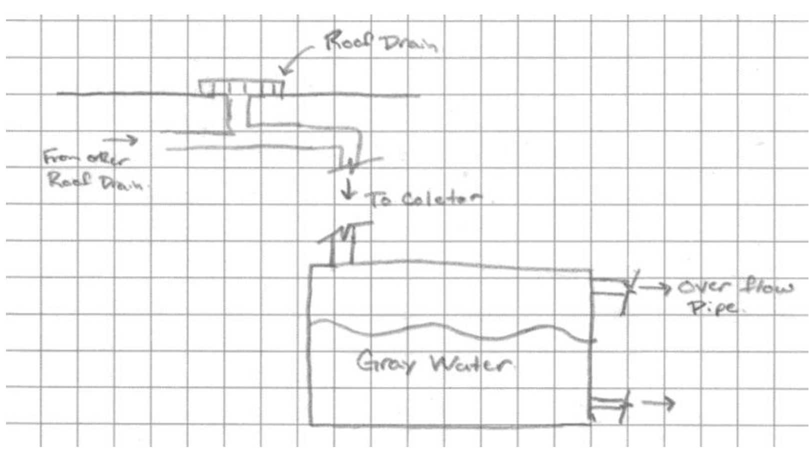


Figure 20: Gray water collection system.

In-Depth Cost and Schedule Impacts of Investigation

Design Goals

The main design goal for the cost and schedule is to compare the One Way Concrete Slab system to the Two Way Concrete Slab system. Other goals include:

- Compare the results to the original design
- Reduce the cost of the structure
- Reduce the schedule of the structure

Cost Analysis

A cost and schedule comparison of original structural load bearing walls to the two new designs, One Way Concrete Slab and Two Way Concrete Slab, was created using RS Means 2010 and retail values. The final contract cost for the project is $39 million and the structural cost is $10.5 million or $183.96 per square foot. Takeoffs for both structural systems were performed to compare the change in cost and the change in schedule. Construction began in 2010, which is why RS Means 2010 was chosen to perform the base cost and schedule for this thesis.

Table 12: Cost Summary

	One Way Slab	Two Way Slab	Original Design
Cost	$4.6 million	$4.4 million	$10.5 million**
Cost per SF	$170.08	$162.78	$183.96

**** Cost is of total building.**

Detailed structural takeoffs were performed for the design portion of the building for both designs. Concrete takeoffs and steel takeoffs were taken from the 3D model. More detail takeoffs of the structures can be found in Appendix I. A summary of the cost analysis is provided in Table 12. It was observed that the One Way Concrete Slab was more expensive than the Two Way Concrete Slab. Both systems were cheaper than the original system. For the One Way Concrete Slab design, it was determined that the structural system would cost approximately $170.08 per square foot, and the Two Way Concrete Slab would cost approximately $162.78 per square foot.

Schedule Analysis

From the takeoff performed for the cost study, the schedule of task was created. Using the recommended crew and the crew output data from RS Means, a detailed schedule breakdown was created using Microsoft Office Project. Microsoft Office Project was used to create a more accurate schedule to show how tasks can over lap during the construction process. A summary of the schedule comparison can be seen in Table 14.

Table 14: Schedule Summary

Schedule Summary	
	# Days
One Way Concrete Slab	375
Two Way Concrete Slab	262
Original Design	267

It was observed that the new concrete design could be constructed in a longer duration than the original design. Also, the Two Way Concrete Slab could be constructed in a longer duration than the One Way Concrete Slab. The One Way Concrete Slab design could be complete in 375 day, and the Two Way Concrete Slab would take 262 days. The original design was completed in 267 days. The difference in schedule days is approximately 5.4 months (108 days) compared to the One Way Concrete Slab, and 0.25 months (5 days) compared to the Two Way Concrete Slab. Detail schedule data can be found in Appendix J.

Conclusion

The structural redesign of The Residences showed that a Two Way Concrete Slab design was comparable to that of a One Way Concrete Slab design. The design process for both systems was straight forward. The slab thickness for the One Way Concrete Slab was determined to be 5" and was 10" for the Two Way Concrete Slab. The One Way Concrete Slab was able to be integrated with the existing architectural design with minor architectural impact. As for the Two Way Concrete Slab, to try and keep a square and regular bay, the system had more problems integrating with the existing architectural design. With keeping the floor to ceiling height as 24", as originally designed, the minimal depth of the beams that needed to be in the One Way Concrete Slab design reduced the space that could be utilized by other disciplines. Concrete shear walls were designed using the provisions and requirements from AIC 318-08. It was found that for the current location, ordinary reinforced concrete shear walls could be used. However, for the high seismic location, ordinary reinforced concrete shear walls could not be used. Therefore, special reinforced concrete shear wall were designed.

The use of Autodesk Robot Structural Analysis program was used throughout this thesis to analyze the redesign. When the program was compared to SAP, it was found that ARSA was similar in their basic elements but lacked the more advanced features that SAP had.

From the investigation of green roof designs, it was determined that most green roofs are comprised of three major layers: Vegetation, Growing Media, and Drainage. For the vegetation layer, it was determined that grass would be able to resist the temperatures and the impact from occupants walking on it. The growing media needed to be comprised of about 50% -70% lightweight aggregate, 10%-20% organic material, and 20%-30% sand. A 2" drainage layer was determined to take any water that was not absorbed by the soil. Once the excess water was drained away, it could be collected to be used for alternative uses.

A cost and schedule comparison was conducted for the One Way Concrete Slab and Two Way Concrete Slab designs. It was determined that the One Way

Concrete Slab would cost about $170.08 per s.f. and could be constructed in 375 days. The Two Way Concrete Slab was found to be $162.78 per s.f. and could be completed in 262 days. This was compared to the original design, $182.96 per s.f. and 267 days, and found that the Two Way Concrete Slab was cheaper and could be constructed in the same time frame.

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