Final Report

University of Virginia's College at Wise New Library

Image Courtesy of Cannon Design

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Structural Option
Advisor: Heather Sustersic
9 April 2014

University of Virginia's College at Wise - New Library

Wise, VA

General Information

Full Height: 119'
Number of Stories: 6
Size: 68,000 GSF
Cost: $43 Million
Date of Construction: Aug 2012 – Aug 2015
Project Delivery Method: Design-Bid-Build

Project Team

Owner:	UVA at Wise
Architect:	Cannon Design
Structural:	Cannon Design
MEP:	Thompson and Litton
Lighting:	Lafleur Associates
Construction:	Quesenberrys, Inc.
Civil:	Thompson and Litton
Landscape:	Hill Studio
AV/Acoustics:	Shen Milsom Wilke
Foodservice:	Culinary Advisors

Project Sponsor: CANNONDESIGN

Architecture

The goal of the façade design was to give the impression that the older existing buildings' architecture was based on the New Library's. This was achieved through use of materials such as brick and stone commonly found on the surrounding buildings.

Construction

Limited site area due to existing campus buildings impacted the construction by requiring offset staging and storage areas, along with the construction of a 500 foot service road.

Structural Systems

Foundation:	Slab on grade with column piers, footings and foundation walls
Framing:	Steel frame, composite wide flange steel members, and normal weight composite deck flooring
Lateral:	9 Reinforced concrete shear walls
Soil Retention:	Temporary Leave-In-Place Soil Retention System, which includes the use of soil nails and shotcrete covering.

Mechanical

VAV system with a roof mounted chilled-water AHU and 145.9 ton chiller providing 41,300 CFM, and an economizer and an a heat recovery unit

Electrical/Lighting

Five 480/277 3-phase panel boards
Nine 280/120 3-phase panel boards

Wall switch and low voltage occupancy sensors used for lighting control

Executive Summary

The New Library at the University of Virginia's College at Wise, located in Wise, Virginia, will serve as a main link between the upper and lower campus areas, which are currently divided by a steep 60 foot hill. The new 6 story, 68,000 ft^2, library will be integrated into the hillside, and will provide students with an easier and safer path across campus. Construction on the New Library began in August 2012 and will be completed in August 2015.

The following report contains information on the analysis and redesign of the structural system for the New Library. A structural overview of the existing steel structural system is included in the first portion of the report, while the majority of the report is comprised of the structural redesign along with additional analyses completed during the semester.

The primary structural redesign was completed using a conventionally reinforced two-way concrete flat slab. Deflection issues in the longer span bays were addressed as part of this redesign. There was also an interest to investigate the feasibility of a post-tensioned concrete floor slab, which was completed as a secondary redesign. RAM Concept was used to aid in the design of the floor systems, and the program output was verified by hand.

Since there was an increase in seismic loads due to the increased weight of the structure, the existing lateral system was analyzed to verify that it would still be adequate under the increased loads. ETABS was used to aid in the analysis of the lateral system.

Due to the decision to integrate the building into the existing hillside, water infiltration of the structure was a major concern. To ensure that the foundation wall drainage system was adequate, an analysis and design of the drainage system was completed as part of the first breadth study, along with a study of the water proofing for the foundation walls and basement slab.

As part of the decision as to whether a concrete structural system was a feasible option, a cost and schedule analysis was completed for the second breadth study. Through this study it was determined that the concrete system did offer a significant savings in cost, and would also offer a slight decrease in project duration.

After completing the redesign of the structure it was determined that a concrete structural system was a feasible option for the structure of the New Library. The functionality of the system in terms of floor-to-ceiling heights and column sizes was similar to that of the steel system and in some cases showed improvement. The concrete system was also able to offer a significant cost savings, and would result in decrease in project duration of a little over a week as long as adequate laborers were available.

Acknowledgments

I would like to thank the following people for helping me make this year such a success:

- The engineers at Cannon Design for giving me the opportunity to use the New Library at the University of Virginia's College at Wise for my thesis, especially Rachel Chicchi who was my contact in obtaining all of the plans and information I needed.

- The engineers at SK&A, especially Walid Choueiri without whose help I never would have learned RAM Concept as well as I did, and Hakan Onel who provided me with some much needed ETABS advice.

- My parents, for always encouraging me to do whatever I wanted to in life, and my fiancé Travis, for always being there for me and always being right.

- The AE faculty, especially my advisor Professor Sustersic for giving me her time, advice, and guidance whenever I needed it.

- All of my classmates and friends, especially:

  Natasha Beck

  Angela Mincemoyer

  Sikandar Porter-gill

  Kristin Sliwinski

  Alyssa Stangl

  You guys have helped me keep my sanity and have given me some of the best memories over the last 5 years.

- My Lord and Savior Jesus Christ for always giving me the strength and guidance to overcome all obstacles.

Table of Contents

General Description of Building

The New Library at the University of Virginia's College at Wise will be located directly between the existing lower and upper parts of the campus, as seen in **Figure 1**. The new 68,000 ft^2 building will be 6 stories tall and will cost approximately $43 million.

Figure 1: Site Location (Courtesy of Cannon Design)

Currently, there is a steep 60 foot hill dividing the UVA Wise campus. This had a large impact on the building's overall design. The New Library will be integrated into the hillside, shown in **Figure 2** and **Figure 3**, and will serve as a significant physical and architectural link between the two parts of campus. A long winding staircase is built into the existing hillside, and provides limited access for students. Students will be able to access the building from the first, second, third, fourth, and fifth levels and a 24 hour access zone will allow students to travel across campus more easily and safely after normal operating hours.

Figure 2: New Library (Courtesy of Cannon Design)

Structurally, the design includes a temporary retaining wall system and foundation walls which extend up to 68 feet below grade on the eastern corner of the building.

The University Architect wanted the New Library to bring a sense of cohesion to the existing buildings on campus. The design team was required to create a visual effect in which it would appear as if the surrounding buildings had been designed based on the New Library, thus creating an architectural link between the new building and the existing buildings. Architectural materials such as brick, stone, and cast stone, were chosen for the library's façade, as these are common to the existing buildings on campus. Along with numerous books and reference materials, the library will offer several other amenities to students including study rooms, conference rooms, smart workstations, and a café.

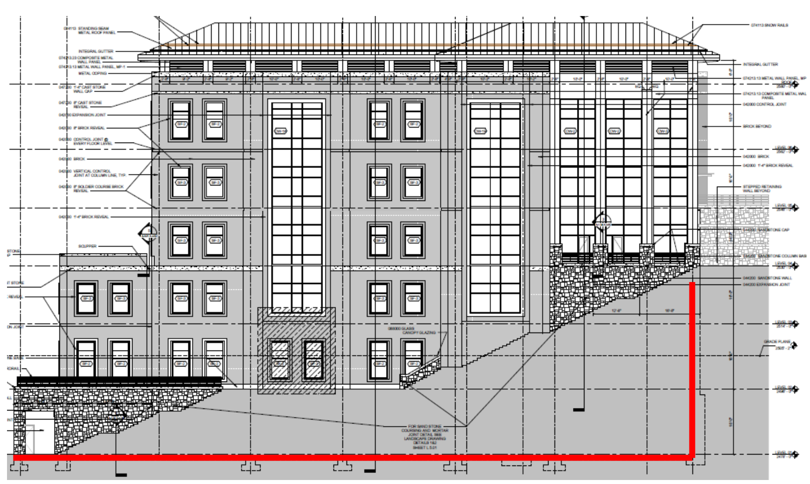


Figure 3: South Elevation Showing Building's Depth into Hillside (Sheet A-3.01)

Structural Overview of Existing System

Brief Description of the Existing Structural System

The New Library at the University of Virginia's College at Wise sits on a foundation system that consists of column piers, spread and strip footings, and foundation walls. Each floor of the six story building is framed using a composite system consisting of composite steel wide flange members and composite decking. Concrete shear walls make up the building's lateral system, along with several foundation walls that aid in resisting lateral soil loads. The upper roof system is comprised of pre-engineered cold formed metal trusses and a separate lateral system consisting of cold formed shear walls. The following section explains these components in more detail.

Building Materials

Structural building materials used in the New Library's design, along with their specifications, are listed below in **Table 1** and **Table 2**.

Structural Steel		
Member	Grade	Fy (ksi)
Wide Flange Shapes and WT Sections	ASTM A992	50
Channels and Angles	ASTM A36	36
Pipe	ASTM A53, Grade B	30
Hollow Structural Sections	ASTM A500, Grade B	46
Base Plates	ASTM A36	36
All Other Steel Members	ASTM A36	36
High Strength Bolts, Nuts, and Washers	ASTM A-325 or A4-490 (Min. ¾" ϕ)	

Table 1: Structural Steel Materials Specifications

Concrete and Reinforcing		
Use	Strength (psi)	Weight (pcf)
Footings	3000	150
Building Foundation Walls	5000	150
Slabs-On-Grade	3000	150
Slabs-On-Steel Deck	3000	150
All Other Concrete	4000	150
Use	Grade	
Typical Bars	ASTM A-615, Grade 60	
Welded Bars	ASTM A-706, Grade 60	
Welded Wire Fabric	ASTM A-185	

Table 2: Concrete and Reinforcing Specifications

Foundation System

S&ME, Inc. performed a geotechnical exploration of the proposed site for the New Library in January 2012. They recommended that the main library structure be supported on spread foundations bearing on bedrock with 8 kip per square foot (ksf) allowable bearing pressure. Due to the high bearing pressure, there was no need for soil improvements. It was also determined that the retaining walls need to be capable of resisting an equivalent fluid pressure of 47 pounds per cubic foot (pcf). See the lateral soil loads section for more details.

The final design for the building's foundation followed the recommendations provided in the geotechnical report. The New Library will be supported on a shallow foundation which will consist of individual spread footings and continuous strip footings, both of which will bear on bedrock.

The individual spread footings are located under the steel columns. At interior columns, the spread footings are located directly at the base of the column (see **Figure 4**). At exterior columns the spread footings are located at the base of the column piers(see **Figure 5**).In both of these cases, the connection is most likely pinned due to the use of the minimum number of required anchor bolts (4), and the fact that no moment frames are used in the structure.

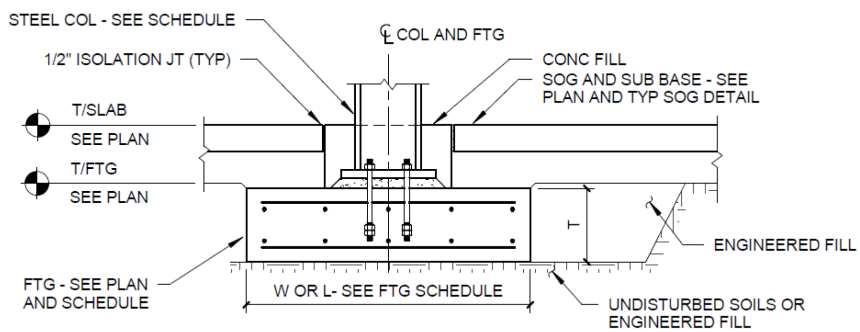


Figure 4: Typical Column Footing without Pier (Sheet S-3.01, Detail 2)

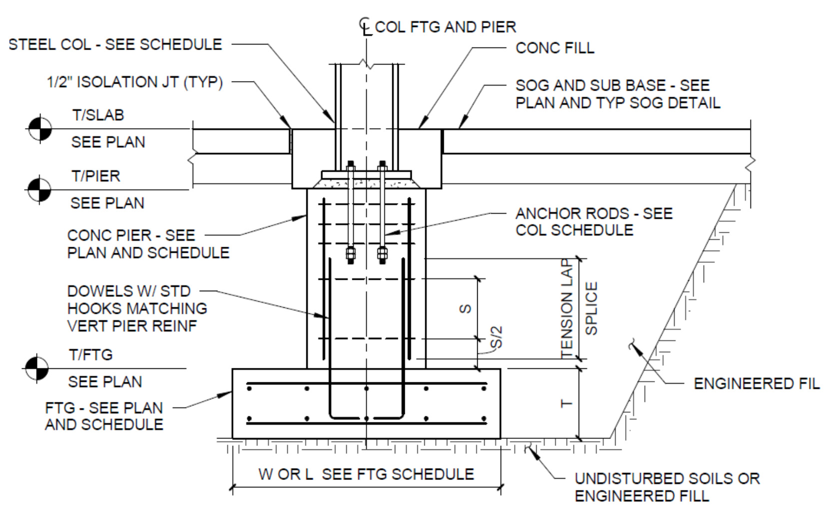


Figure 5: Typical Colum Footing with Pier (S-3.01, Detail 1)

Continuous strip footings are located under the perimeter foundations walls. Many of the footings are stepped in order to limit the amount of excavation required.

One of the biggest challenges with the project was designing a way to resist the lateral soil forces on the building's structure. After discussing several options, the team chose to use a temporary leave-in-place soil retention system (which includes the use of soil nails and shotcrete covering). This system was determined to be the most cost effective and efficient solution. The temporary system allows the soil to be excavated down to the bearing grade and the shotcrete then doubles as one side of the formwork for the foundation walls, thus decreasing the cost of formwork for the project.

It is expected that the rock anchors will deteriorate over time. Thus, the foundation walls are designed to resist the full soil load once the superstructure is complete. This was done by designing the foundation walls with a fixed-base condition, providing sufficient rebar to resist flexure, eccentric footings, and lateral support at upper floor levels. The foundation wall and this design concept can be seen in **Figures 6** and **Figure 7**.

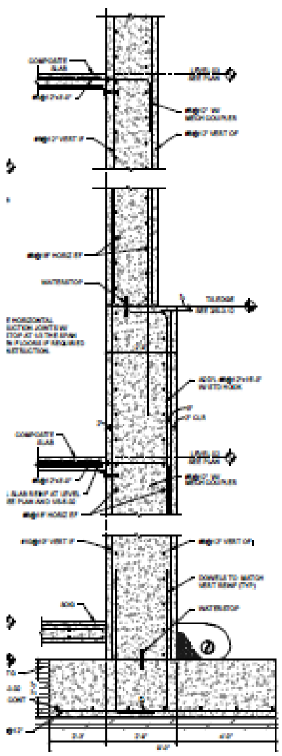


Figure 6: Foundation Wall(S-3.11, Detail 1)

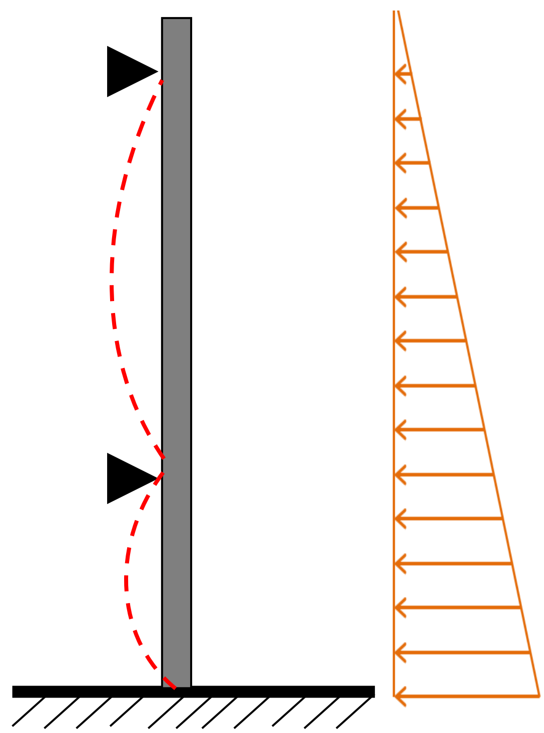


Figure 7: Foundation Wall with Design Concepts

Slab Thicknesses

Two different slab-on-grade thicknesses are used in the building. A 5” slab-on-grade, reinforced with 6x6-W2.9xW2.9 welded-wire-fabric, is located at Levels 1 and 2. On level 1, these slabs are located in the 24-hour access zone, which is an area of moderate student traffic. On Level 2, there is also a small section in the south corner of the building that is on grade and utilizes a 5” S.O.G. An 8” slab-on-grade, reinforced with #5@18” each-way on both the top and the bottom is located on Level 1.It is supporting areas of high density storage where specialty compact shelving will be located. **Figure 8** and **Figure 9** show the extents of each slab thickness on Levels 1 and 2.

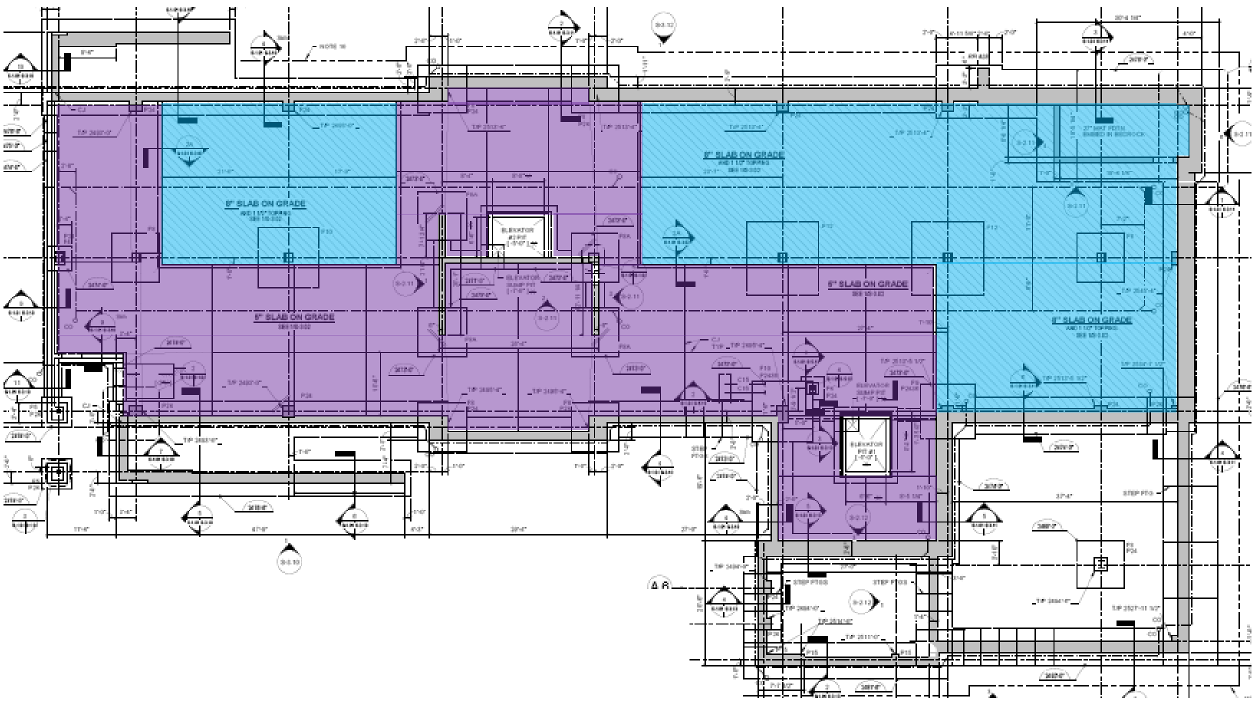


Figure 8: Level 1 Slab-On-Grade Thicknesses (S-1.01)

5”
8”

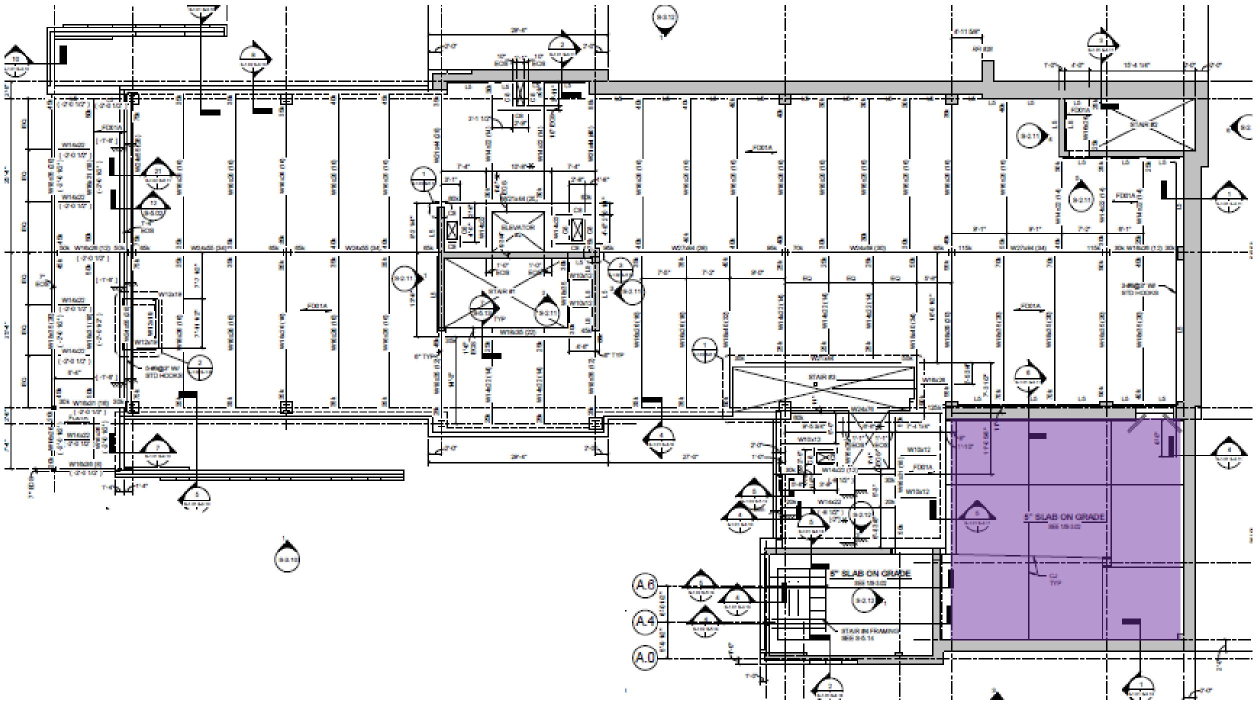


Figure 9: Level 2 Slab-On-Grade Thicknesses (S-1.02)

5”

Floor System

The New Library's floor system is a composite steel system comprised of 4 ½" normal weight concrete reinforced with6x6-W2.9xW2.9 welded-wire-fabric on 2" 18 gage steel deck (6 ½" total thickness). The 4 ½" topping provides the required 2 hour fire rating without the additional cost of spray-on fire proofing. The deck typically runs perpendicular to wide flange steel members, and in cases where the deck runs parallel to the members, #4 x 4'-0" rebar is placed at 18" on center to decrease cracking due to tensile forces in the concrete slab. Composite action is achieved by transfer of the load from the slab to the members by ¾" diameter x 3 ½" long shear studs.

Typical Bay: Floor

Multiple sized bays are used in the New Library. The typical beam span is 25'-4" and typical bay sizes range from 25'-4" to 31'-0". Typical members used to frame Level 2 up through Level 6 are primarily W16x26 beams. Smaller beams, such as W14x22, are used in areas around the stairwells and larger beams, such as W18x35, are used in areas supporting general collections along with areas of high student traffic. Typical interior girders supporting these beams are W25x55 and spandrel girders vary in size depending on location. **Figure 10** below shows a 27'-4" bay with W16x26 beams.

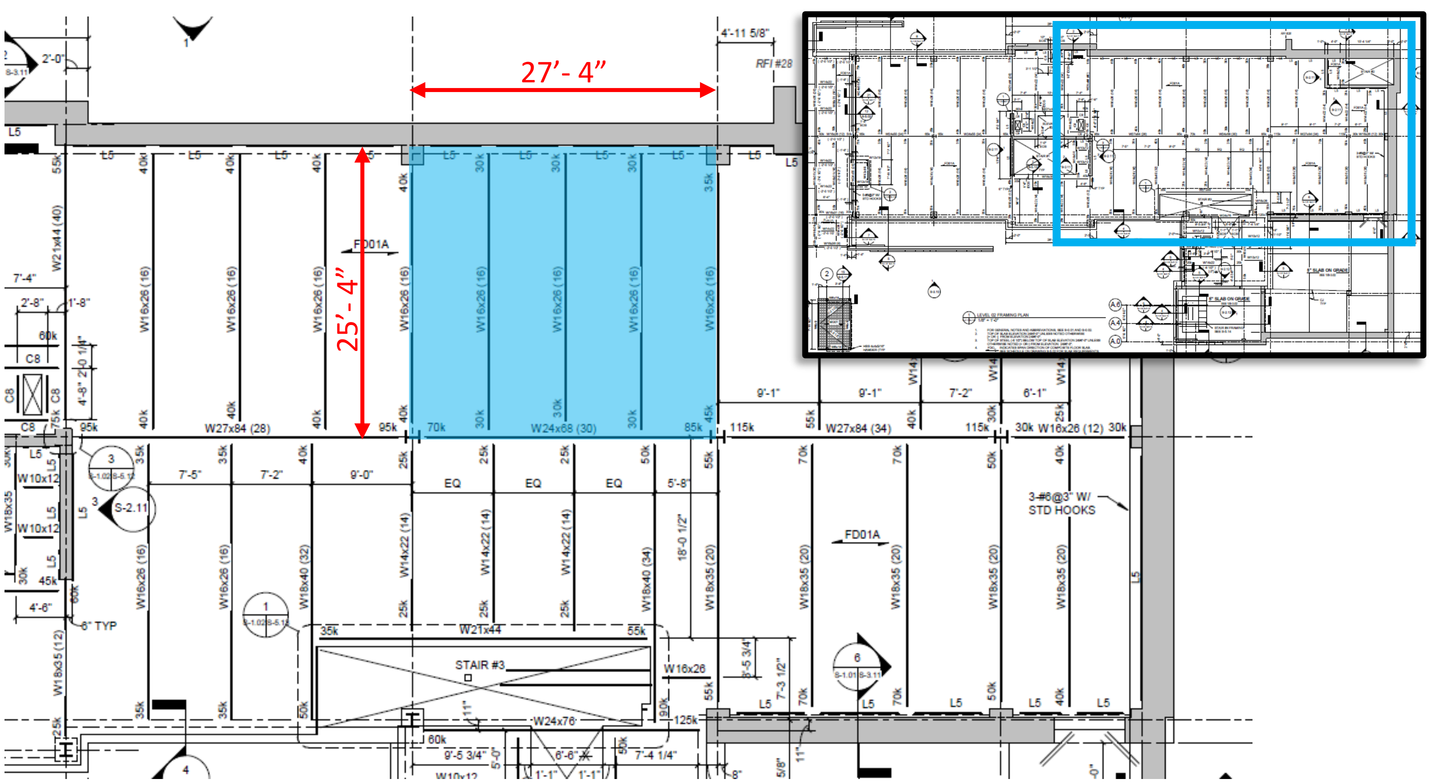


Figure 10: Level 2 Framing Plan Showing Typical Bay (Sheet S-1.02)

Framing System

All of the main structural columns in the New Library are wide flange steel members. Other columns found in the building are hollow structural steel, which are used in vestibules and in entrance areas. Most of the columns have a 12” depth and vary in weight; with the majority ranging between W12x45 and W12x65.The largest columns in the building are W12x170 and they extend between Level 1 and Level 3. The need for these larger columns is due to the increased tributary area, as compared to typical bays, and larger design loads from general collections on all upper floors. **Figure 11** shows the location of the W12x170 columns.

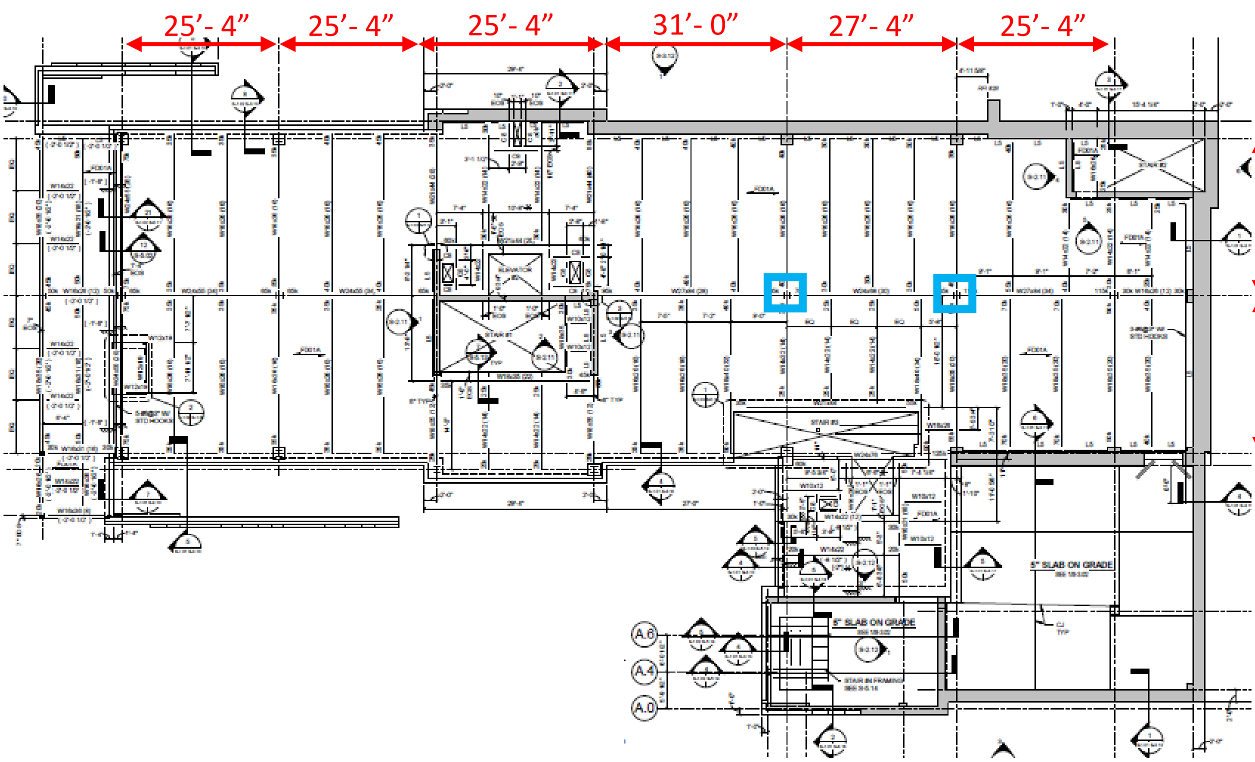


Figure 11: Level 2 Showing Location of W12x170 Columns (Sheet S-1.02)

Roof System

Two separate roof systems were used to complete the New Library. A lower roof covers the majority of the building between column lines 3-9 and A-E and supports an air handling unit and a chiller (mechanical well area). The framing is composite wide flange steel beams and a 6 ½” NWC slab. The upper roof is designed to mimic the existing campus buildings and also serves to conceal the building’s air handling unit and chiller.

Lower Roof

Bay sizes used in the lower roof framing of the New Library are the same as those used in the framing of the lower floors. The typical beam span is 25’-4” and typical bay sizes range from 25’ 4” to 31’-0”. Beams used to frame the lower roof are typically W18x35. This larger beam size is due to increased design loads based on the HVAC system. **Figure 12** below shows a 27’-4” bay with W18x35 beams.

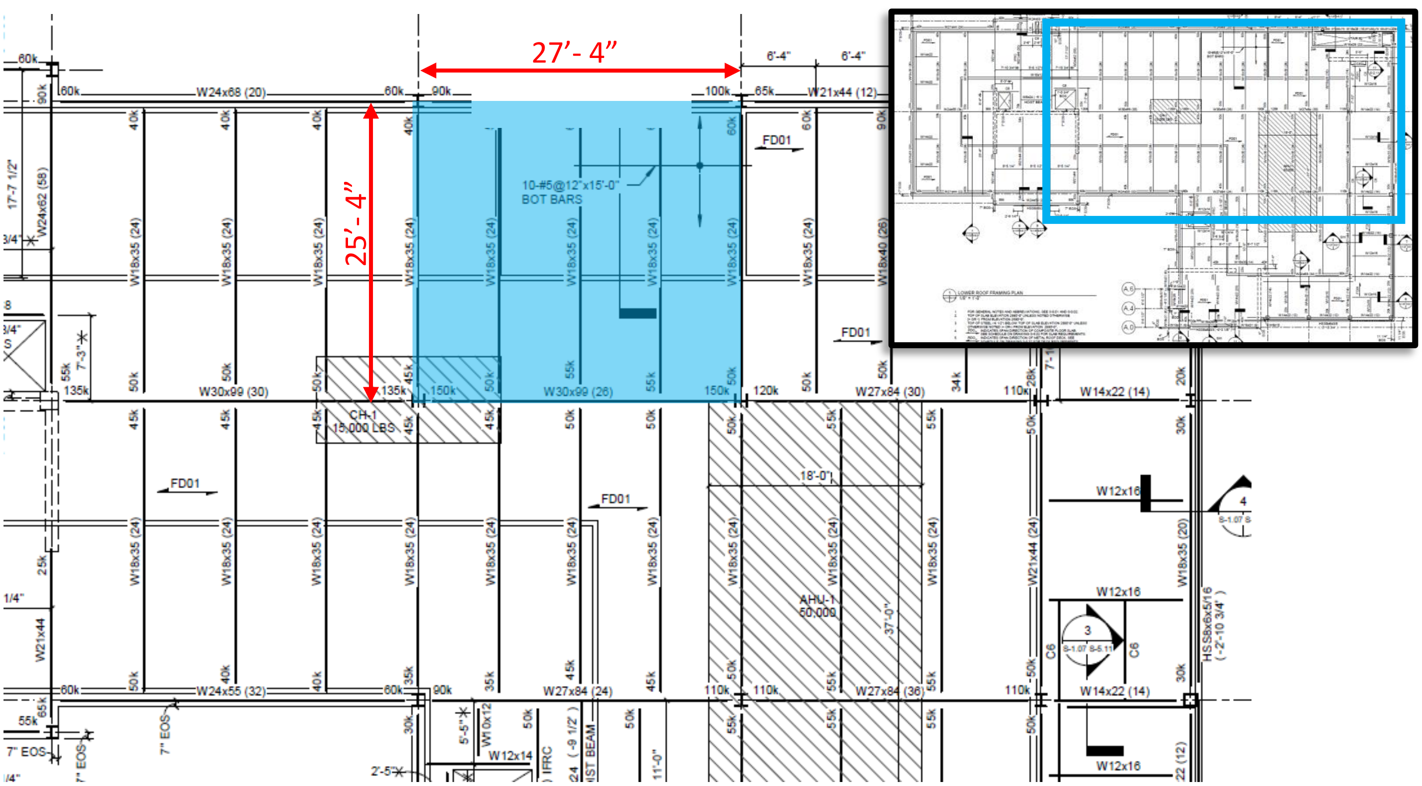


Figure 12: Lower Roof Framing Plan Showing Typical Bay (Sheet S-1.07)

Upper Roof

The upper roof is a raised false mansard consisting of pre-engineered cold formed metal trusses and cold formed shear walls. This layout can be seen below in **Figure 13**.These trusses are triangular in shape and approximately 9'-7"tall, are covered by 1 ½" type B roof deck, and sit on 6" load bearing CFMF studs. This can be seen below in **Figure 14**.

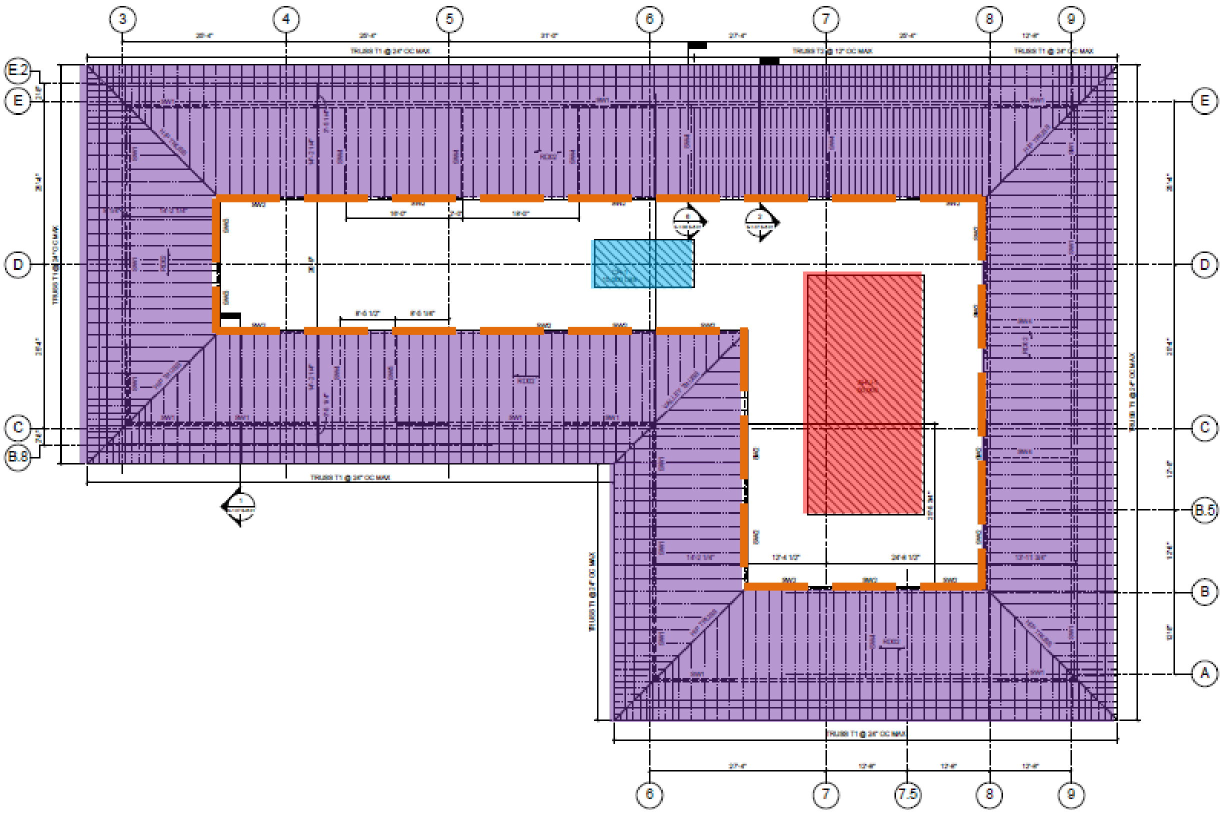


Figure 13: Upper Roof Framing Plan Showing Pre-engineered Trusses (Sheet S-1.08)

- Pre-engineered Light Gage Cold Formed Metal Trusses
- Mechanical Well Area
- Air Handling Unit
- Chiller

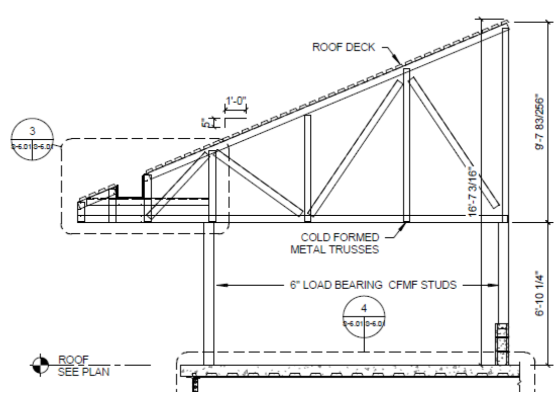


Figure 14: Cold Formed Metal Truss (S-6.01)

Lateral System

The lateral force resisting system for the New Library consists of ordinary reinforced concrete shear walls. There are nine 12” thick shear walls of varying length and height that make up this system. **Figure 15** shows the location of these shear walls and categorizes them based on their heights.

Each shear wall is reinforced with #5 rebar at a code maximum spacing of 18” each-way on each-face of the wall. This layout of reinforcing is typical with the exception of two walls that have condensed spacing in lower sections of the wall, especially in the horizontal direction. This condensed spacing is due to increased shear forces from soil loads.

Two of the walls located in the eastern corner of the building are introduced below grade as foundation walls. Levels 1 through 4 of this corner are located below grade at the location of the maximum retained soil. Once above-grade, soil loads no longer are the controlling load case and the walls are then designated as shear walls.

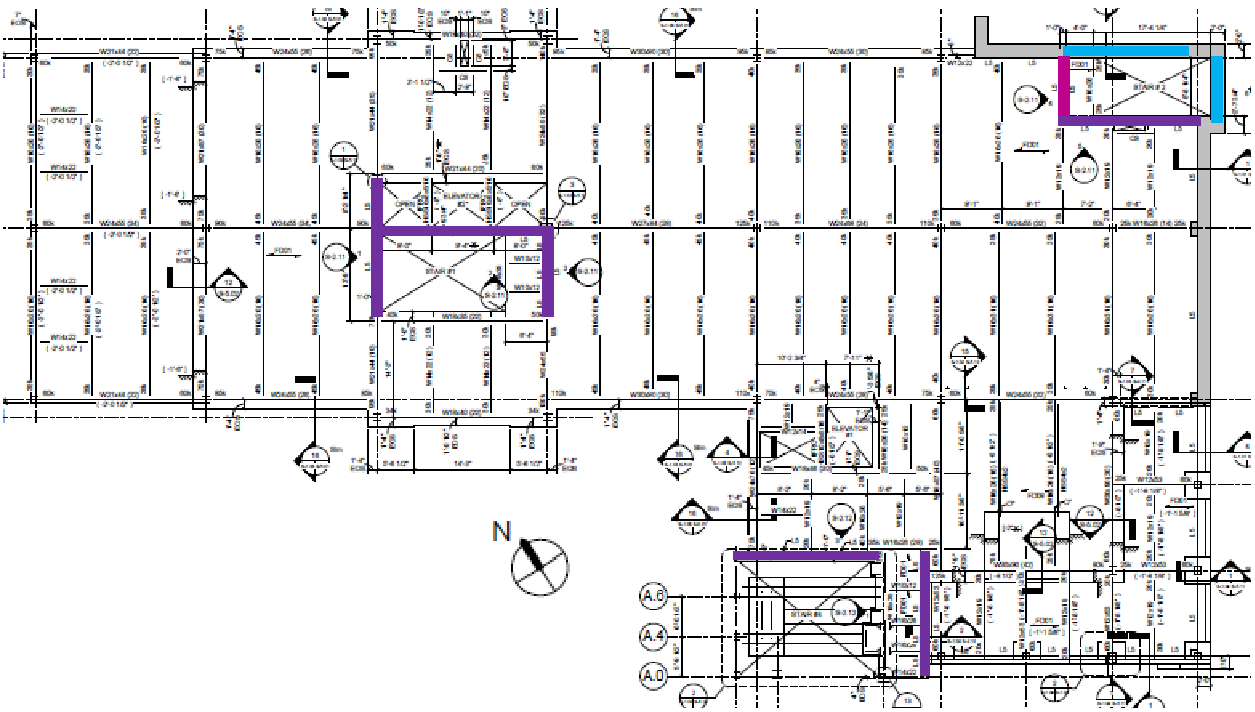


Figure 15: Floor Framing Plan Showing Shear Wall Locations (S-1.04)

Shear Walls From Level 1 to Roof Level
Shear Walls From Level 1 to Level 5
Shear Walls From Level 5 to Roof Level

Design Loads

The following section focuses on topics concerning the loads used in the original structural design of the New Library. These topics include national codes used for live and lateral loadings, the determination of the design loads used, and the load paths for different loading conditions.

National Code for Live Loads and Lateral Loadings

Load	National Code	Section
Live	ASCE 7-05 Chapter 6, and UVA Facility Design Guidelines	5
Lateral	ASCE 7-05 Chapter 12	8

Table 3: National Code Chapter and Section for Live and Lateral Load

Gravity Loads

Live Loads

Design live load values are listed on sheet S-0.01 of the structural drawings. The majority of these loads were determined using Chapter 4 of ASCE7-05, with the exception of the design roof loads. The loads not found in ASCE 7-05 are listed in **Table 4** with an explanation of how they were determined.

Load	Determination of the Load
Roof area below sloped roof	The area below the sloped roof will most likely never see a live load, so the design team chose to simply provide a small allowance.
Roof mechanical area	The design team chose to blanket the roof with a live load instead of using the specific dead loads for the mechanical units. To determine a reasonable allowance the team used the largest PSF unit at the time and increased the load by 25%.
Minimum Roof Live Load	UVA Facility Design Guidelines specifies a minimum design roof live load.

Table 4: Live Loads Not Found in ASCE7-05

Dead Loads

Design dead loads are listed on sheet S-0.01 of the structural drawings. These loads were based on material weights and industry standards used at Cannon Design.

Snow Loads

Design snow loads must follow the UVA facility Design Guidelines. These guidelines state that ground snow loads are to be determined by case studies and other Virginia Unified Statewide Building Code requirements. The USBC adopts chapters 2-35 of IBC 2009 which references ASCE 7-05.

Lateral Loads

Wind Loads

Design wind loads were determined using Section 6.5 of ASCE 7-05.Section 6.5, Method 2, which is the analytical procedure for determining design wind loads for buildings of all heights.

Seismic Loads

Design seismic loads were determined using section 12.8 of ASCE 7-05. Section 12.8 prescribes the Equivalent Lateral Force Procedure for determining seismic design loads.

Soil Loads

From the geotechnical report performed by S&ME, Inc. it was determined that the foundation walls should be designed for an at-rest equivalent fluid pressure of 47 pcf. The soil loads on the foundation walls are then dependent on the height of the wall. **Figure 16** shows this distributed force on the foundation wall.

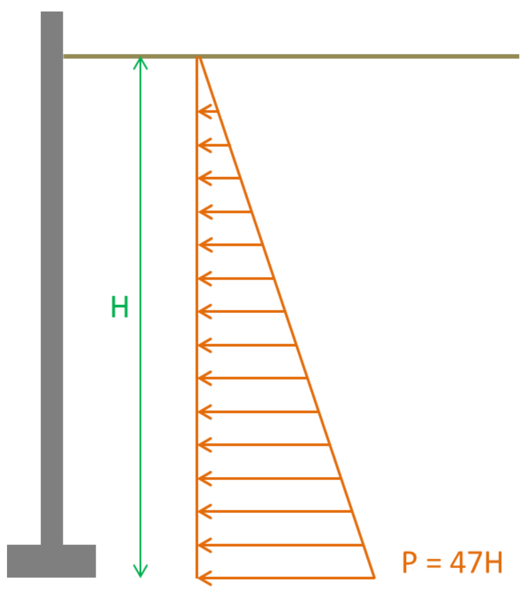


Figure 16: Equivalent Lateral Fluid Pressure

Design Codes and Standards

Below is a list of the design codes and standards used in the structural design of the New Library at the University of Virginia's College at Wise:

Codes Used in Original Design and Analysis

- **International Code Council**
  IBC 2009 (Chapters 2-35 Adopted by Virginia Uniform Statewide Building Code)
- **American Society of Civil Engineers**
  - ASCE 7-05: Minimum Design Loads for Buildings and Other Structures
- **American Concrete Institute**
  - ACI 318-08: Building Code Requirements for Structural Concrete
  - ACI 530-08: Building Code Requirements and Specifications for Masonry Structures
- **American Institute of Steel Construction**
  - AISC 360-05: Specifications for Structural Steel Buildings (Steel Construction Manual 13th Edition) - LRFD
- **University of Virginia Facilities Management and University Building Official**
  - Facility Design Guidelines

Below is a list of the design codes and standards used in the structural redesign of the New Library at the University of Virginia's College at Wise:

Codes Used in Redesign

- **International Code Council**
  IBC 2012 (Chapters 2-35 Adopted by Virginia Uniform Statewide Building Code)
- **American Society of Civil Engineers**
  - ASCE 7-10: Minimum Design Loads for Buildings and Other Structures
- **American Concrete Institute**
  - ACI 318-11: Building Code Requirements for Structural Concrete
  - ACI 530-11: Building Code Requirements and Specifications for Masonry Structures
- **University of Virginia Facilities Management and University Building Official**
  - Facility Design Guidelines

Proposal

Problem Statement

As previously discussed, the New Library utilizes a composite steel framing system, and the lateral system involves the use of ordinary reinforced concrete shear walls. Previous technical reports have shown that the existing gravity system and lateral system for the New Library are adequate to meet both strength and serviceability requirements.

Since no significant problems exist with the current steel structural system, a scenario has been created in which it is desired by the University to investigate the feasibility of a concrete framing system. As part of this investigation, the impact of the concrete system on the cost and construction schedule of the project should be compared to that of the existing steel system to allow the owner to make an informed decision. The architectural design of the New Library is based on the existing campus and surrounding buildings, so it is required that there is limited architectural impact with the system redesign.

Proposed Solution

The new structural system has been chosen to be a mild-steel reinforced two-way concrete slab with drop panels. This system will be analyzed using RAM Concept, which is one of the most efficient tools for designing concrete floor systems. It has already been determined that the existing shear walls are the most efficient lateral system and will be integrated with the new concrete system. ETABS will then be used to analyze the existing shear walls under anticipated increased seismic loads due to the increase in building mass. Columns will also be redesigned in concrete and will follow the existing column layout in order to minimize impact to the interior layout of the New Library.

The decision to use a two-way slab as the primary re-design was based on several factors. In Technical Report 3, a two-way flat slab was found to be the least expensive of the alternative concrete systems that were studied. Two-way systems with drop panels help to reduce the amount of negative reinforcement required at the columns and have become an industry standard. The bay sizes of the New Library are of a moderate span, approximately 25 feet, and are relatively square, which is ideal for a two-way slab system.

The largest bay in the New Library spans 31 feet. It is recognized that excessive deflections in this bay will be an area of concern with the alternate system. To address this issue alternative drop panel sizes and shallow beams will be investigated.

There is also an interest to investigate the option of a post-tensioned system as a secondary re-design. A schematic design of the full floor system will be designed using RAM Concept and will be compared to the two-way concrete system in order to determine if a complete PT floor option would be a practical option.

Breadth Topics

Construction Breadth

A comparative cost analysis will be performed in which the cost of the existing composite steel system will be compared to that of the redesigned two-way concrete system. This cost analysis will include materials, erection/formwork, and labor. Cost information for the existing steel system will be provided by Cannon Design, while cost information for the redesigned concrete system will be determined using RS Means.

A schedule analysis will also be performed in which the impact of this system change on the critical path and construction schedule will also be considered. The construction schedule and the critical path required for the concrete system will be compared to that of the existing steel system, which will be provided by Cannon Design.

Mechanical Breadth

Drainage at the base of the foundation/basement walls will be investigated. The location of the ground water levels in relation to the footings will be determined and the flow rate of the ground water and rain water will be calculated. It is assumed that a drainage system will be required. Thus, a drainage pipe and sump pump, if needed, will be sized based on the flow rate and code specifications.

Waterproofing for the foundation/basement walls and the effect of the water beneath the slab-on-grad will also be investigated.

MAE Requirement

Graduate level work will be incorporated into the structural system redesign. This will be done through the use of computer modeling. Material covered in AE530, Computer Modeling of Building Structures will be used extensively throughout the redesign. ETABS will be used in the analysis of the lateral system under ACSE7-10 wind and seismic loads and increased gravity loads due to the increased building weight. ETABS will be used to verify the shear walls for these increased loads. RAM Concept will be used in the design of the conventionally reinforced concrete slab and the post-tensioned concrete slab. This program will be learned through guided self-study for modeling of both conventionally reinforced concrete slabs and post-tensioned systems.

Structural Depths

Primary Depth: Two-Way Concrete System

Floor System Design

It was desired to investigate the possibility of a concrete gravity system for the New Library at the University of Virginia's College at Wise. It was determined that a two-way flat slab with drop panels would be the best choice for the system redesign. This system was chosen due to the decreased formwork and labor costs (as compared to the one-way system). The two-way system also is believed to work well for square spans of approximately 25 feet, which is similar to the majority of those found in the New Library.

For the design, RAM Concept was used as the primary modeling software. This program was chosen for its efficiency in designing concrete floor systems, along with its high recommendations from practicing engineers. Most of the floors in the New Library are similar in layout and required loading. As a typical floor, Level 5 was chosen to be redesigned. Level 5 is comprised primarily of areas for general collections, offices (including partitions), and reading rooms. The required design loadings for these areas per ASCE7-10 are 150psf, 80 psf, and 60 psf respectively. It was also decided that the areas designated as reading rooms were to be designed with a loading of 80 psf. This was done in order to match the design loading used by the original design team to allow for the most accurate comparison of the steel and concrete structural systems. All loads used in the design of the concrete system can be seen in Table A1 and Table A2 in Appendix A.

A base slab thickness was first chosen using the CRSI Manual. Based on the spans sizes and approximant factored superimposed floor loads a 10" slab was chosen as a base thickness. To determine the starting drop panel size ACI318-11 Section 13.2.5 was used. This section requires a minimum dimension of L/6 in each span direction, and a minimum total depth of 1.25h.

After adjusting the initial drop panel sizes to pass punching shear checks, the required size of several drop panels was extremely large. For example, the drop panel at column 8B was almost 14'-0" wide and projected 1'-1" below the slab. Based on this information, it was determined that the required drop panel sizes were unacceptable and an alternative design was needed. **Figure 17** shows the floor layout, and **Table 5** shows the required drop panel sizes and thicknesses after initial adjustments.

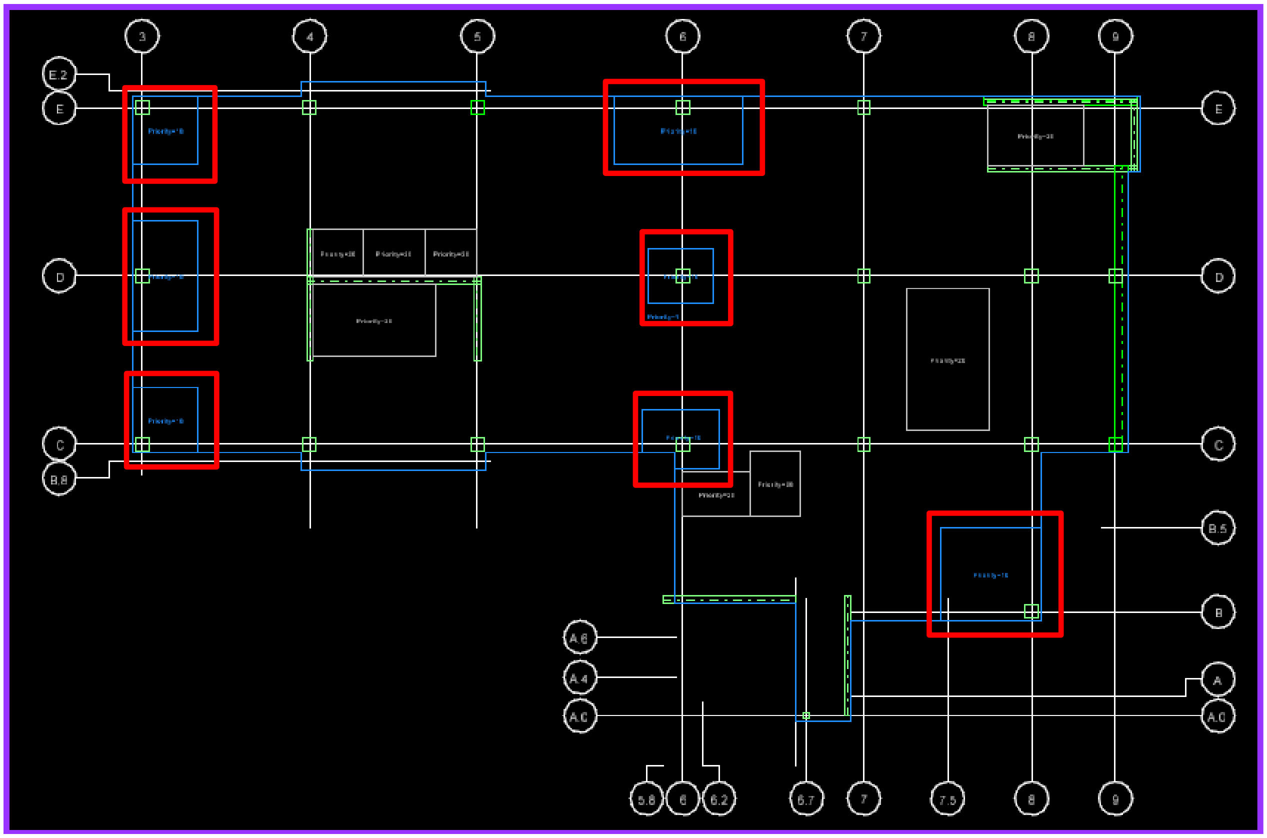


Figure 17: Flat Slab with Drop Panels – RAM Concept Plan

Column	-X (FT)	+X (FT)	-Y (FT)	+Y (FT)	Thickness (IN)	Required an Increase in Size
3E	1.33	8.44	8.44	1.33	6	Yes
3D	1.33	8.44	8.44	8.44	9	Yes
3C	1.33	8.44	1.33	8.44	9	Yes
6E	10.33	9.11	8.44	1.33	4	Yes
6D	5.17	4.56	4.22	4.22	6	No
6C	6.20	5.47	3.83	5.07	2.5	Yes
8B	13.67	1.33	1.33	12.67	13	Yes

Table 5: Required Drop Panel Sizes After Initial Adjustments

It was quickly noticed that punching shear, around the columns and drop panels, was going to control the floor design. Several trial floor designs were investigated including the addition of edge beams, small interior framing beams, and shear studrails in place of drop panels. Several of the main trial layouts can be seen in Figures B1 – B5 in Appendix B.

Drop Panels were originally chosen over shear studrails because studrails are fairly new to the engineering community and were originally proprietary. After some research, it was determined that shear studrails are now widely produced and are no longer an increased cost on building projects. One company, Decon, has been producing studrails since the 1970's and they state that their studrails actually "provide a lower overall in-place cost when compared to other existing punching shear control alternatives."

Shear stud rails seemed like the best option for punching shear, but a drop panel was ultimately used due to the increase in stiffness required to decrease long term deflections. See the deflection check section for more details on this choice and the decision to also include a shallow beam in the two bays adjacent to column 6D along column line D.

The final floor design was comprised of a 10” two-way slab with mild-steel reinforcement. There is a 7’-0” x 7’-0” drop panel at column 6D, and a shallow 14” deep, 10’-0” wide beam along column line D spanning between column line 5 and 7. Edge beams and interior beams around floor openings were also included. **Figure 18** shows this final floor layout, and **Figure 19** and **20** show the beam mark plan along with the beam schedule.

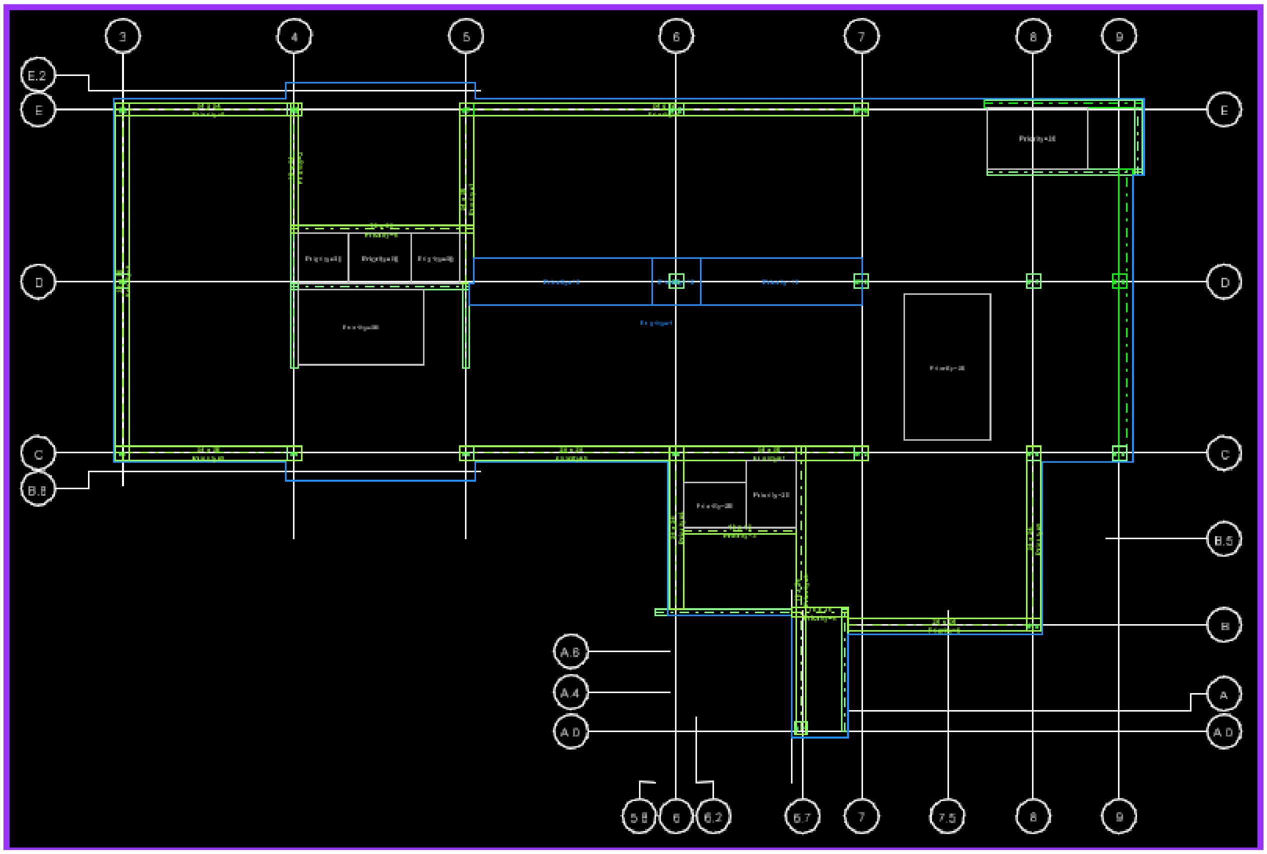


Figure 18: Final Floor Design – RAM Concept Plan

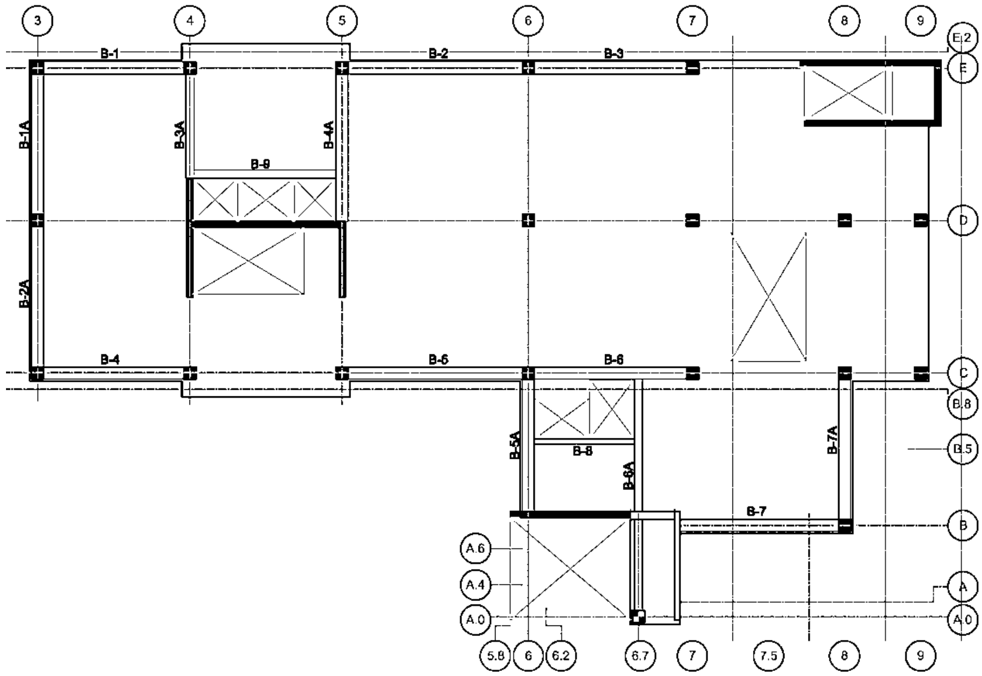


Figure 19: Beam Mark Plan

MARK	SIZE		REINFORCING					STIRRUPS			ADD'L HORIZ. REINFORCING EACH FACE
	WIDTH (IN)	DEPTH (IN)	BOTTOM	TOP				TYPE	SIZE	SPACING (IN)	
				L.E.	F.L		R.E.				
					LEFT	RIGHT					
B-1	24	24	7	4	2	2	5	2 LEG	#4	11.5	1
B-2	24	24	8	8	3	3	11	2 LEG	#4	11.5	2
B-3	24	24	6	11	2	2	6	2 LEG	#4	11.5	2
B-4	24	30	8	6	4	4	8	2 LEG	#4	9	1
B-5	24	24	9	11	3	3	11	2 LEG	#4	6.5	1
B-6	24	30	8	11	2	2	7	2 LEG	#4	6.5	1
B-7	10	12	2	6	1	1	2	2 LEG	#4	5.5	1
B-8	24	24	8	10	3	3	5	2 LEG	#4	5	1
B-9	16	24	4	0	4	4	0	2 LEG	#4	7.5	1
B-1A	24	30	8	9	3	3	4	2 LEG	#4	12	1
B-2A	24	30	8	4	3	3	9	2 LEG	#4	12	1
B-3A	16	24	5	11	1	1	5	2 LEG	#4	7	0
B-4A	24	30	12	12	2	2	6	2 LEG	#4	12	0
B-5A	24	24	6	0	2	2	8	2 LEG	#4	10.5	1
B-6A	16	24	9	0	9	9	0	-	-	-	0
B-7A	24	24	6	6	1	1	6	2 LEG	#4	10.5	0

Figure 20: Beam Schedule

Verification of Output

Although the slab design was completed using design software it was important to verify the output. In order to verify the results produced by RAM Concept several checks were performed.

FEA vs. EFM

The first verification was to check the design moments giving by RAM Concept. Due to the limitations of the direct design method, the equivalent frame method was used to calculate the design moments by hand. The moments along column line 3 were calculated and the results were then verified using SP Slab. These calculations along with the SP Slab output can be seen in Appendix C.1.

Once the hand calculations were finished, they were compared to RAM Concept. It was determined that it was not possible to directly match the results given by RAM Concept by using the equivalent frame method. According to the RAM Concept user manual, the program uses finite element analysis to calculate moments in the slab. Finite element analysis allows the program to more accurately predict the elastic behavior of a slab as compared to traditional frame/strip analysis (equivalent frame method). RAM Concept also provides a more accurate distribution of forces across the design strip.

Although the results cannot directly be matched, it is possible to verify results within a certain percentage. The total design moment across the spans will be comparable, even though the distribution of negative/positive moments to the joints varies.

Therefore, to verify the moments provided by RAM Concept, the total moments for both the span segment and the column strip were calculated, and the totals for each method were compared. **Table 6** below shows the percent different in the design moments. Based on the percent difference it was determined that RAM Concept's force distribution was reasonable.

Percent Different in Total Design Moments			
	Hand Calculations/SP Slab	RAM Concept	% Difference
Total Moment in Span A-B	650.13	712.75	9%
Total Moment in Span B-C	806.82	777.11	4%
Total Moment in Both Spans	1456.95	1489.86	2%

Table 6: Design Moment Comparison

Wide Beam (One-way) Shear/ Punching (Two-way) Shear

Column 6D is a critical column due to the large tributary area and high live loading. RAM Concept designed the required shear studrails for this column, but the program also designed shear stirrups. Typically one-way shear does not control in two-way slabs, and there shouldn't have been shear stirrups along with the shear studrails. To determine if one-way shear reinforcement was required hand calculations were performed.

Table 7 and **Table 8** below shows the results of the one-way shear and two-way shear hand calculations as compared to RAM Concept. The full hand calculations can be seen in Appendix C.2.

One-Way Shear			
	RAM Concept	Hand Calculations	% Difference
Max Shear Demand	143.1 K	143.1 K	0%
Max Capacity	302.6K	278.4 K	8%
*Hand Calcs take shears at d away from the face of support, RAM shears taken at face of support			

Table 7: One-Way Shear Comparison

Two-Way Shear			
	RAM Concept	Hand Calculations	% Difference
Max Shear Demand	284.6 K	280 K	1.6%
Max Capacity	189.7 K	189.9 K	0.1%

Table 8: Two-Way Shear Comparison

These results verify that RAM Concept's outputs are reasonable. They also show that the slab is adequate for one-way shear but not for two-way shear.

Checking Shear Stud Rails

Although shear stud rails were not used in the conventionally reinforced floor design they were used for the post-tensioned floor design. Therefore, verification of RAM Concept's output was complete and is included in Appendix C.3.

Deflection Checks

It was noted in the proposal that deflection problems were to be expected due to the 31'-0" and 27'-4" spans. Once the basic floor design was completed the deflections were checked. Table 9.5(b) from ACI318-11, shown in **Figure 21** below, was used to determine a maximum permissible deflection of L/480.

TABLE 9.5(b)—MAXIMUM PERMISSIBLE COMPUTED DEFLECTIONS

Type of member	Deflection to be considered	Deflection limitation
Flat roofs not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$\frac{\ell^{*}}{180}$
Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$\frac{\ell}{360}$
Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements (sum of the long-term deflection due to all sustained loads and the immediate deflection due to any additional live load)†	$\frac{\ell^{\ddagger}}{480}$
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections		$\frac{\ell^{\S}}{240}$

Figure 21: Table 9.5(b) from ACI318-11

RAM Concept produces deflection contours based on the initial deflections. The deflections of primary concern were the ones that included initial and long term deflections. These long term deflections are due to creep and shrinkage and consider the effects of cracking. To view these deflections a time-history analysis must be run. RAM Concept accounts for long term deflections a little differently than what we would like, so some adjustments need to be made. An explanation of these adjustments can be seen in Appendix D.

After an initial run, the maximum deflection in the slab was found to be L/297 (span 5D-6D). **Table 9** below shows the most critical spans and their corresponding deflections.

Span	Span Length (FT)	Deflection	L/480	Pass/Fail
5D - 6D	31	1.33	0.775	Fail
6D - 7D	27.33	1.02	0.683	Fail
5E - 6D	40	1.43	1.0	Fail
6E - 7D	37.33	1.24	0.933	Fail
5C- 6D	40	1.33	1.0	Fail

Table 9: Initial Deflection Check

Several trial designs were completed to determine the best solution to the deflection problems. Options included load averaging, compression reinforcement, drop panels, and shallow beams. These trial designs can also be seen in appendix D.

The final design incorporated a 7'x7' drop panel at column 6D and two shallow 14" beams. The final deflection checks can be seen in **Table 10**.

Span	Span Length (FT)	Deflection	L/480	Pass/Fail
5D - 6D	31	0.709	0.775	Pass
6D - 7D	27.33	0.511	0.683	Pass
5E - 6D	40	0.875	1.0	Pass
6E - 7D	37.33	0.817	0.933	Pass
5C- 6D	40	0.827	1.0	Pass

Table 10: Final Deflection Check

Edge Deflection Check

Deflection checks at the edge of the slab were also completed to ensure the design met the required L/600 to prevent damage to the masonry façade due to cracking.

The sustained deflection of the slab was compared to the maximum permissible deflections. Initial deflections will not affect the façade due to the fact that it will not be placed on the structure until after these initial deflections have occurred. **Table 11** below shows the deflection checks.

Span	Span Length (FT)	Initial Deflections (in)	Final Deflections (in)	Sustained Deflections (in)	L/600 (in)	Pass/Fail
3C-D3	25.33	0.018	0.19	0.17	0.51	Pass
D3-E3	25.33	0.016	0.18	0.16	0.51	Pass
3E-4E	25.33	0.037	0.31	0.27	0.51	Pass
4E-5E	25.33	0.059	0.55	0.49	0.51	Pass
5E-6E	31	0.044	0.52	0.48	0.62	Pass
6E-7E	27.33	0.020	0.35	0.33	0.55	Pass
7E-8E	25.33	0.007	0.18	0.17	0.51	Pass
9E-9D	25.33	0.000	0.01	0.01	0.51	Pass
9D-9C	23.33	0.000	0.03	0.03	0.47	Pass
9C-8C	12.67	0.000	0.10	0.10	0.25	Pass
8B-7B	25.33	0.060	0.50	0.44	0.51	Pass
6C-5C	31	0.045	0.53	0.49	0.62	Pass
5C-4C	25.33	0.059	0.56	0.50	0.51	Pass
4C-3C	25.33	0.029	0.23	0.20	0.51	Pass

Table 11: Edge Deflection Checks

Reinforcement

The reinforcement for the floor slab is comprised of #5 bars running in both the latitude direction (E-W) and the longitude direction (N-S). **Figure 23** below shows a section through the floor slab showing the cover and location of the reinforcement in the floor slab (dimensions are given in inches). The location of this section can be seen in **Figure 22**.

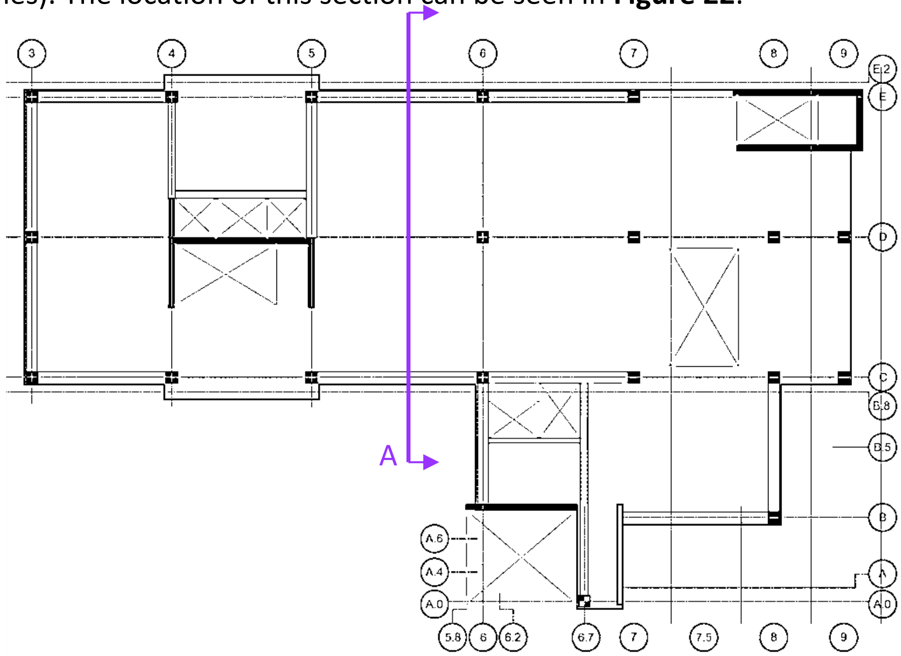


Figure 22: Plan View Showing the Location of Section 'A'

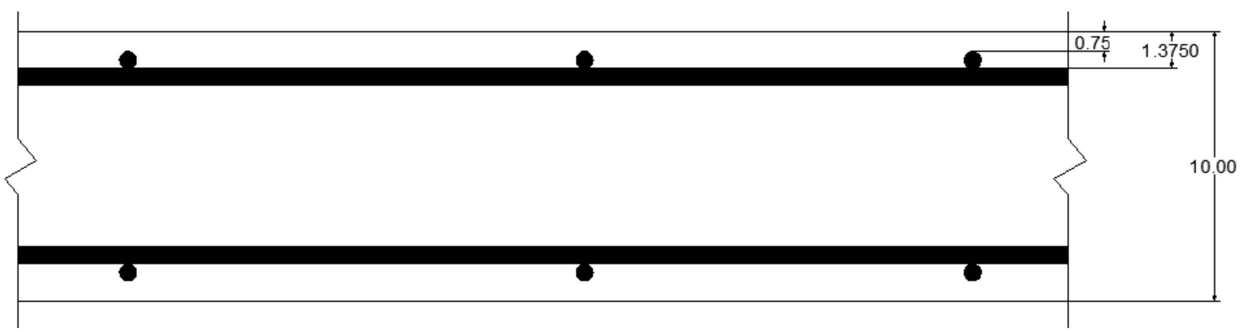


Figure 23: Section 'A' Showing Location and Cover of Slab Reinforcement

RAM Concept lays out the reinforcement based on the calculated requirements. This layout is not always the most economical due to the fact that it uses the minimum number of bars in each individual design strip instead of a traditional mat layout. To adjust the reinforcement layout, a top and bottom mat was chosen based on the minimum required reinforcement and the additional required reinforcement was calculated. **Figure 24** below shows the layout of the reinforcement along with the reinforcement schedule. The calculations for the additional reinforcement required can be seen in Table E1-E4 in Appendix E.

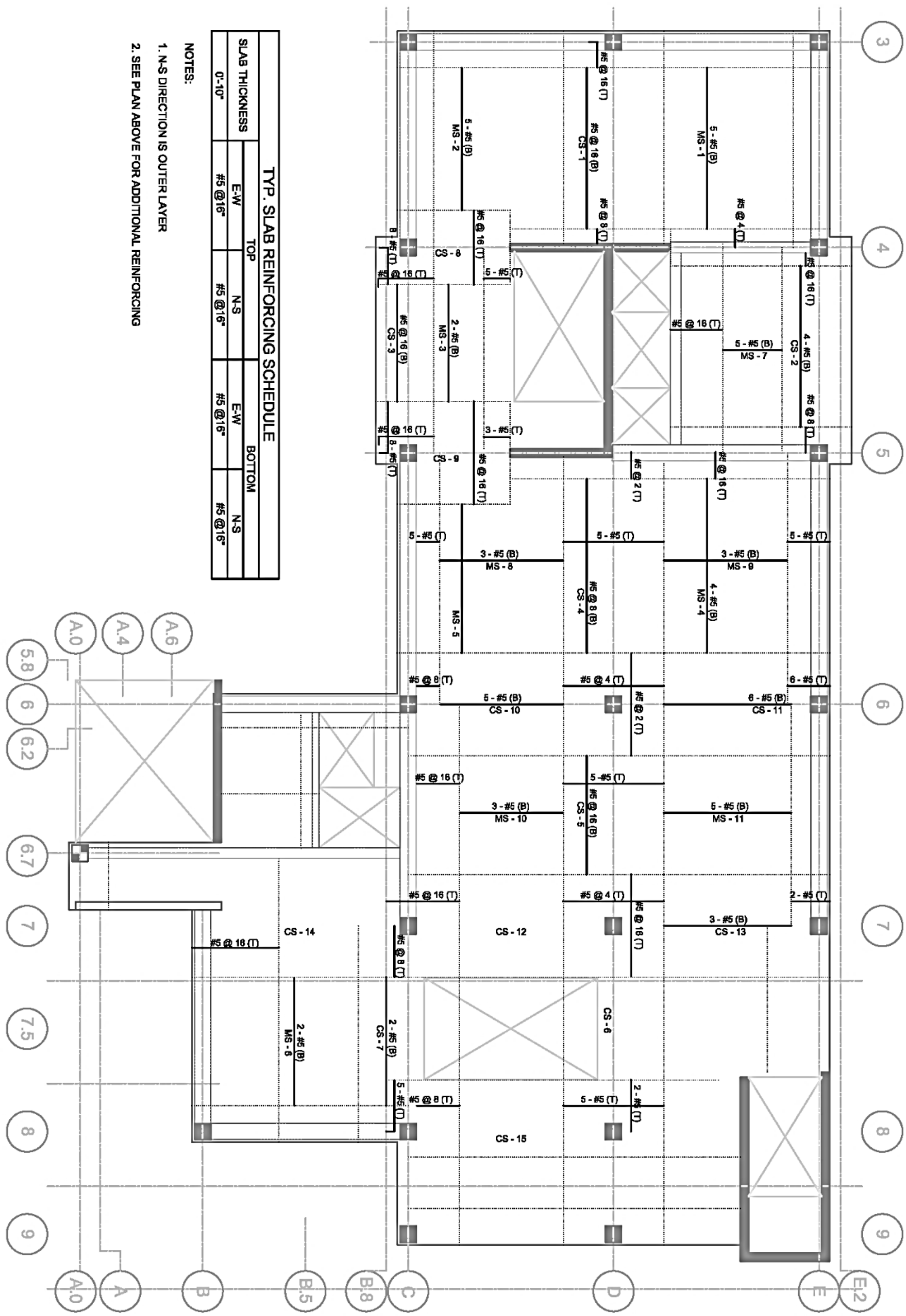


Figure 24: Reinforcement Layout

Column Design

The existing steel columns and the partition walls encasing them are approximately 24”x24”. It was desired to limit the size of the new concrete columns based on the existing interior design. Therefore, the beginning trial size was 24”x24”. It was expected that this size would be larger than required for axial loads.

After completion of the floor design the columns were designed, and the selected size was verified. Excel was used to calculate the axial load on each column at level 1, and unbalanced moments were taken from RAM Concept. Several critical columns were chosen and designed using SP Column. Figure F1 showing the axial load calculations, along with the SP Column output, can be seen in Appendix F.

The final column design for typical columns was 24”x24” with (8) #8 longitude bars and #3 transverse ties. Column 6C and 7E did required additional longitudinal reinforcement on lower levels, and column 6D also required an increase in size. **Table 12** below shows a summary of these results.

Column	Level	Size	Reinforcement
6D	1-3	28” x 28”	(16) #8’s
6C	1	24” x 24”	(12) #8’s
7C	1	24” x 24”	(10) #8’s

Table 12: Size and Reinforcement of Non-Typical Columns

The required axial and moment capacity was then compared to the available strength of the critical columns. It is recognized that the columns are over designed for capacity at upper levels, but this is due to the problems with punching shear. If the columns were made smaller the punching shear problems would be increased. When the columns are checked at level one, the ratio of capacity to required strength is much more economical. This ratio for the non-typical columns can be seen in Table 13 below, and additional column ratios can be seen in Table F1 in Appendix F.

Column	φPn/Pu
6D	1.02
6C	1.02
7C	1.01

Table 13: Column Capacities

Secondary Depth: Post-Tensioned System

There was an interest to investigate a post-tensioned option for the floor system in the New Library. Before beginning, design research was done to determine the pros and cons of this type of system. Both the information gained from research and schematic floor designs were used to decide whether the system was a feasible option.

Pros, Cons, and Concerns

There are several advantages with the PT system:

- Reduced slab depth and floor weight
- Longer spans achieved more economically
- Deflection and vibration control
- Improved constructability

One major concern with the PT system is shortening. Once the tendons are stressed they pull the edges of the slab in towards the center of the structure. This is expected with the PT system, but can be an issue when there is an unfavorable arrangement of shear walls in the structure. **Figure 25**, courtesy of "Post-Tensioned Concrete: Practical Applications" by the engineers at Holbert Apple, shows the preferred arrangement of shear walls vs. unfavorable arrangements of shear walls. It is best to have the shear walls located in the center of the structure. When they are near the edges the floor shortens and induces stresses into both the slab and the shear walls.

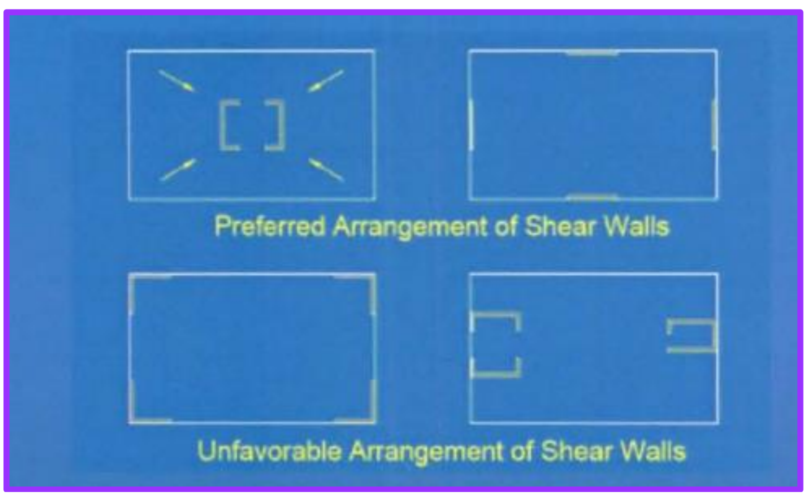


Figure 25: Arrangements of Shear Walls: Preferred vs. Unfavorable

Based on this information the location of the shear walls in the New Library would be unfavorable for the PT system. **Figure 26** below shows the location of the shear walls in the New Library.

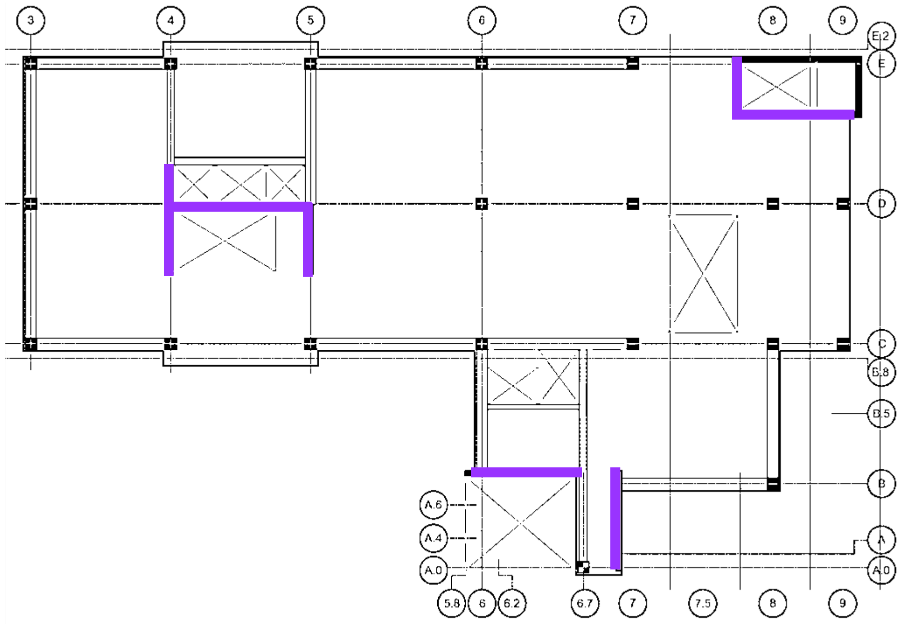


Figure 26: Location of Shear Walls in New Library

To avoid problems with shortening due to the location of the shear walls a pour strip would be used. This pour strip is a strip of the floor slab that is approximately 4' to 6' wide that is left open. An example of this can be seen in **Figure 27**. The pour strip remains open a minimum of 28 days (42 to 90 preferred) which gives time for the majority of shrinkage and creep to occur.

Figure 27: Example of a Pour Strip

Though possible, this method is often not favorable to construction workers due to the fact that is restricts site access to certain areas and can create a tripping hazard.

Shortening of the slab can also be an issue when there is foundation walls located next to the slab edge. Where this occurs the slab pulls away from the foundation walls, which also induce stresses into both the slab and the walls. To avoid this problem, slip joints must be added at the foundation walls. The slip joint will allow independent motion of the slab and the walls while allowing them to remain joined together.

PT systems are most beneficial in structures in which the tendons can span uninterrupted across several bays. In the New Library, there are often only two bays in the longitude direction, and although the latitude direction is several bays long, many of the bays are interrupted by the shear walls. This layout won't allow the tendons to be as efficient.

Initial Design

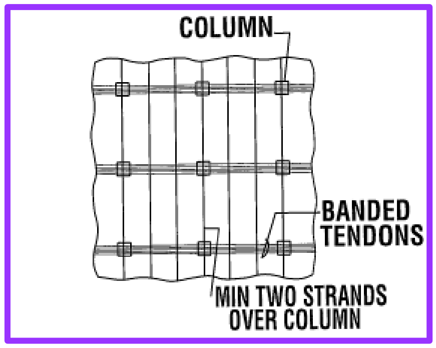


Figure 28: Banded-Distributed Tendon Layout (Courtesy of PTI Tech Note 8)

The PT system for the floor design was chosen to be an un-bonded system with a banded–distributed tendon layout. When using the banded-distributed tendon layout the tendons are banded together in one direction over column lines, while tendons in the other direction are uniformly distributed across the slab. This layout can be seen in **Figure 28**. The banded–distributed layout was chosen because it is a common layout used in industry, and is easier for constructability due to the fact that weaving of the tendons is only required in one direction rather than a basket weave in two directions. This layout also allows for the maximum permissible tendon drape since the banded and distributed tendons generally do not cross at their high or low points, except at the supports.

The initial slab thickness for the slab in the PT system was chosen to be 8”. This was chosen due to the fact that the slab thickness required for the conventionally reinforced slab was 10”, and one typical advantage of the PT systems is a decreased floor thickness.

Concrete strength of the system was also increased to an f’c of 5000psi, as compared to an f’c of 4000psi used in the conventionally reinforced design. One reason for this increase was because in industry, PT systems are almost always designed using 5000psi concrete. Another reason for this choice was to increase the max allowable concrete stress. ACI318-11 Section 18.3.3 states that prestressed two-way slab systems shall be designed as a Class U system with $f_t \leq 6\sqrt{f'c}$. This places an upper bound on the max tensile stress in the slab, and by increasing the f’c to 5000psi this upper bound is also increased.

The drop cap and shallow beams were also not added until it could be determined if the additional stiffness would be required.

Laying Out the Tendons

The tendons for the floor slab are ½" diameter tendons. Banded tendons were run in the latitude direction (long direction) and distributed tendons were run in the longitude direction (short direction). See **Figure 29** and **Figure 30** below for an overview.

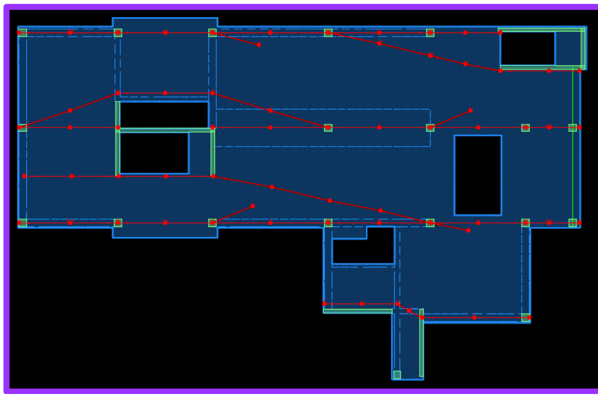


Figure 29: Tendons in the Latitude Direction

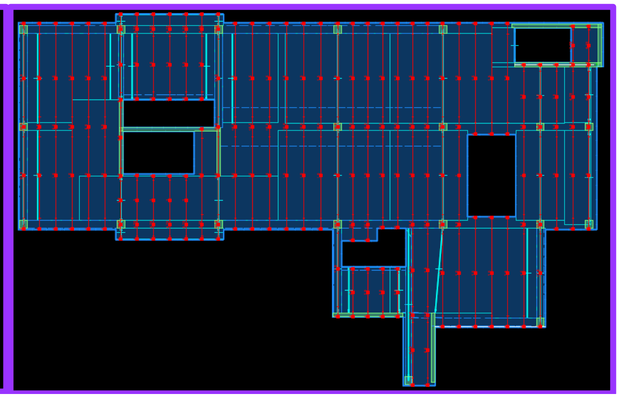


Figure 30: Tendons in the Longitude Direction

This layout was chosen to take advantage of the increased "d-value" in the longer direction. The banded tendons always go on the "outside" and the distributed tendons always go on the "inside", thus giving the banded tendons a larger "d-value". **Figure 31** below shows a section through the floor slab showing the cover and drape of the tendons at a column. The initial elevation of the tendons before balancing can be seen in Table G1 and Table G2 in Appendix G.1.

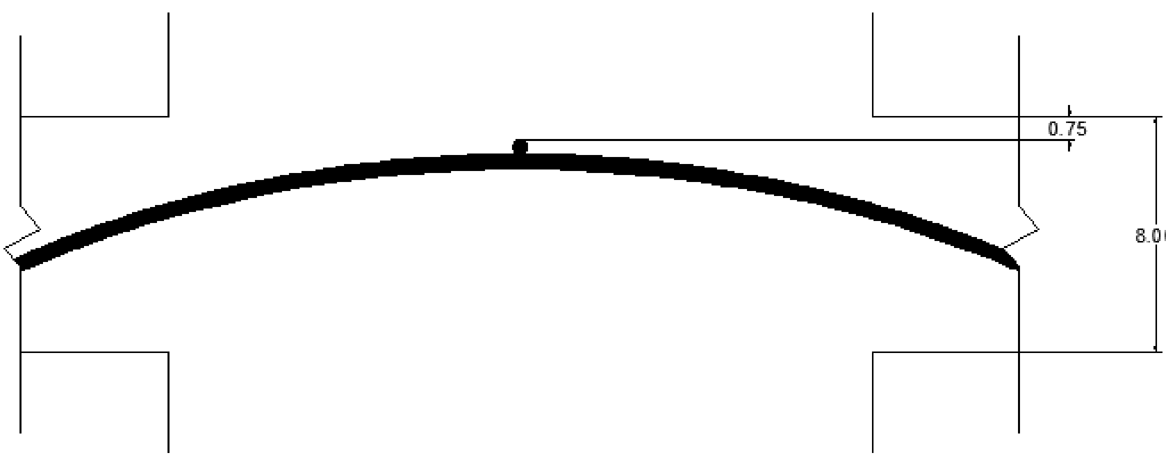


Figure 31: Section Showing Cover and Drape at Column

Choosing the Initial Number of Tendons

The initial number of tendons was based on the minimum precompression stress P/A (force/area). For an interior occupancy, an economical design with typical spans should be in the range of approximately 125psi – 175 psi.

For the banded direction the area used for this calculation was the design strip area. Once the force was determined, the number of tendons was calculated. It is an industry standard that in PT flat slab unbonded tendon construction, typically ½" diameter 7-wire tendons are used with an area of 0.153 in^2, an ultimate strength of 270ksi, and low relaxation steel (this is based on ACI318-11 Section 18.12.6). The average force, after all stress losses, is designed to end up at 27 kips/tendon at the worst. Therefore, to determine the initial number of tendons the calculated force was divided by 27 kips. A sample calculation of this can be seen below next to **Figure 32**, and the initial number of tendons for all of the design strips can be seen in Table G3 in Appendix G.2.

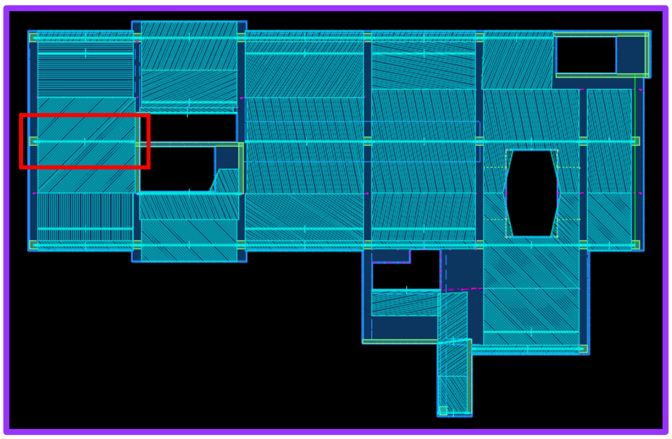


Figure 32: Plan View Showing the Location of Span 1-1

<u>Span 1-1</u>

$$A = (24.33')(12/1')(8") = 2429\ in^2$$

$$P = (125psi)(2429in^2) = 304kips$$

$$Tendons = \frac{304\ kips}{27\ kips/tendon} = 11\ Tendons$$

For the distributed tendon direction, a minimum of two tendons were run together. ACI318-11 Section 18.12.6 states that a minimum of two tendons must be provided in each direction over columns, so it is standard to run a minimum of two tendons together across the entire slab. For the distributed direction, the spacing between tendons was calculated using the minimum compression stress of 125psi. This calculation can be seen below.

$$125psi(8")(12"/1') = 12000\ lb/in$$

$$x = \frac{54000\ lb}{12000\ lb/ft} = 4.5\ ft$$

ACI318-11 Section 18.12.4 states that the maximum distributed direction tendon spacing is the minimum of 5 feet and eight times the slab thickness. For the floor system eight times the slab thickness is 5.33', so 4.5' is less than the maximums.

Balancing the Tendons

The tendons were balanced both after the initial layout and after the final layout was completed. Too much uplift in a tendon can cause deflection reversals that may cause cracking in the slab. Therefore, balancing the load in adjacent tendon spans helps to prevent this from happening.

The balancing load is based on the weight of the design stip. The lower limit for the strip was 50% of the design strip weight, while the upper limit was 125% of the design strip weight. These percentages are based on industry standards.

RAM Concept calculated the current balancing load for the design strip. If the balancing load exceeded the upper limit, then the upper limit was input into the program as the desired load. RAM Concept then adjusted the tendon elevation to decrease the balancing load to the upper limit, and the elevation was then manually adjusted to the nearest ¼”. This occurred most often at locations in the slab where an exterior span was much shorter than an adjacent interior span.

If the balancing load was below the lower limit tendons needed to be added. Tendons were then only added after tendons had been added to pass for flexural requirements. This was done because most often this criterion will control the number of tendons rather than the lower limit.

Table G4 and Table G5 in Appendix G.3 shows the final balancing loads given by concept, whether they passed/failed, and the elevation adjustments.

Adjusting the Number of Tendons

As stated before, the upper bound on the maximum tensile stress in the slab is limited to $f_t \leq 6\sqrt{f'c}$. Therefore, this often controls the slab design. After the initial layout was run it was determined that this was the case, and several of the design spans were failing due to this limitation.

The economical upper limit for the maximum precompression force is 350psi. If more tendons than this are required to meet the stress limitations, another design solution should be considered. Based on this force, the max number of tendons for each design strip was calculated in the same way the initial number was calculated. The design strips that were failing were determined, and the number of tendons in these strips was increased. Table G6 in Appendix G.4 shows the initial number of tendons in the banded direction for each design strip, the max number of tendons, and the new required number of tendons. For the distributed tendons, it was determined that a maximum of 6 tendons could be used.

Even with the maximum number of tendons span 11-1 and 11-2 still failed. This was expected due to the problems with these bays in the design of the two-way conventionally reinforced slab. The first solution was the addition of a 12" deep, 10 ft wide shallow beam spanning between column line 5 and 7 along column line D (shown in **Figure 33**). This was chosen due to the fact that it was successful in the previous design. Once this beam was added, the number of tendons for span 11-1 and 11-2 were able to be reduced below the maximum limit.

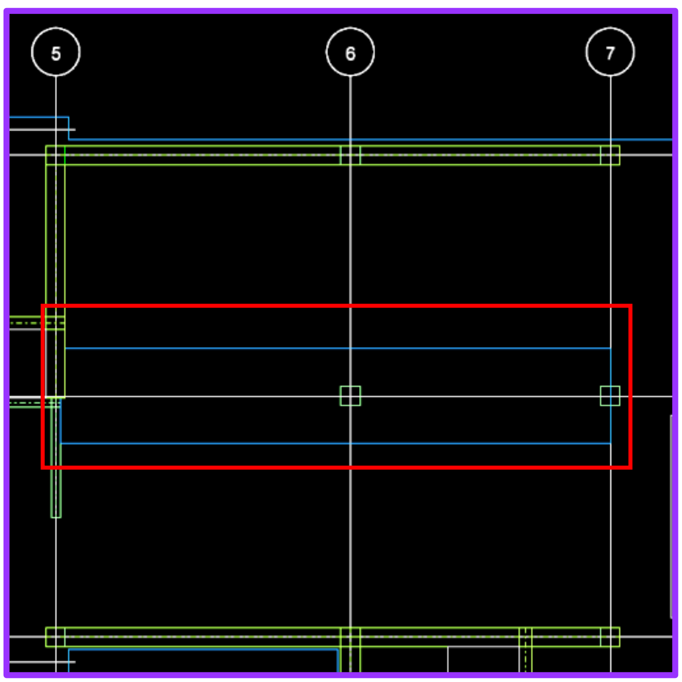


Figure 33: Shallow Beam along Column Line D

Checking Deflections

As stated before, ACI318-11 Section 18.3.3 states that prestressed two-way slab systems shall be designed as Class U systems. ACI318-11 Section 9.5.4.1 states that for Class U systems the deflections shall be calculated using gross section properties.

To account for creep and shrinkage without using cracked section properties, an additional load combination was input into RAM Concept.

2(Self-Dead) + 2(Balance) + 3 (Other Dead) + 1.6 (Live)

This combination accounts for service instantaneous deflections and long term deflections. A factor of one is applied to service instantaneous deflections which include deflections due to the superimposed dead load plus the live load. A factor of two is applied to the long term deflections which include deflections due to the dead load self-weight, the superimposed dead load, and 30% of the sustained live load. The factor of two is the long term deflection factor for duration of 5 years or more found in ACI318-11 Section 9.5.2.5.

ACI318-11 Section 9.5.4 also states that the deflections for all prestressed concrete flexural members must be compared to the maximum permissible deflection values in Table 9.5(b). Therefore, a limit of L/480 was also used for the PT deflection checks.

Five critical spans were checked for deflections. Based on the maximum deflection limit of L/480 it was determined that the system was adequate for deflection requirements. The deflection for each critical span is shown in **Table 14** below.

Span	Span Length (FT)	Deflection	L/600	Pass/Fail
5D - 6D	31	0.578	0.620	Pass
6D - 7D	27.33	0.413	0.547	Pass
5E - 6D	40	0.667	0.800	Pass
6E - 7D	37.33	0.587	0.747	Pass
5C- 6D	40	0.659	0.800	Pass

Table 14: Critical Deflections

Edge deflections were also checked against the maximum permissible deflection limit of L/600 and were determined to be adequate. These deflections can be seen in **Table 15** below.

Span	Span Length (FT)	Deflections (in)	L/600 (in)	Pass/Fail
3C-D3	25.33	0.14	0.51	Pass
D3-E3	25.33	0.13	0.51	Pass
3E-4E	25.33	0.21	0.51	Pass
4E-5E	25.33	0.47	0.51	Pass
5E-6E	31	0.32	0.62	Pass
6E-7E	27.33	0.21	0.55	Pass
7E-8E	25.33	0.21	0.51	Pass
9D-9C	23.33	0.03	0.47	Pass
9C-8C	12.67	0.03	0.25	Pass
8B-7B	25.33	0.23	0.51	Pass
6C-5C	31	0.37	0.62	Pass
5C-4C	25.33	0.50	0.51	Pass
4C-3C	25.33	0.25	0.51	Pass

Table 15: Edge Deflections

The Final Layout

Unlike the conventionally reinforced slab, a drop panel at the column was not needed to limit deflections in the post-tensioned system. Therefore, only the shallow beam was required at column 6D. Although no drop panel was required for deflections, the shallow beam was not thick enough to eliminate punching shear at the face of the column. Instead of the addition of a drop panel to eliminate the punching shear, shear studrails were used. Shear studrails not only can cost less, but are also beneficial from a construction stand point due to the fact that it is easier to lay the tendons through the studs rather than having to weave them through the stirrups.

The final tendon layout is shown in the two figures below. **Figure 34** shows the banded tendons and **Figure 35** shows the distributed tendons.

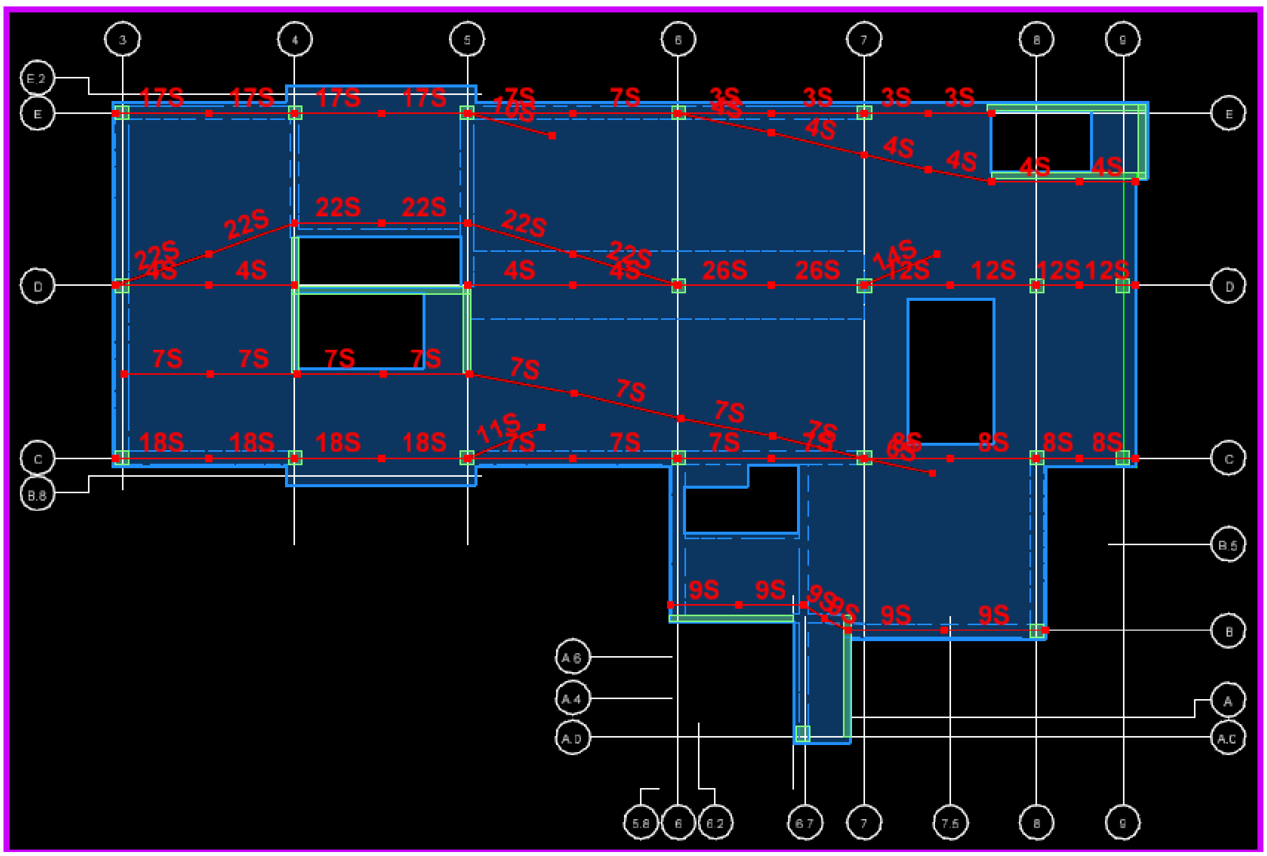


Figure 34: Final Layout of Latitude Tendons

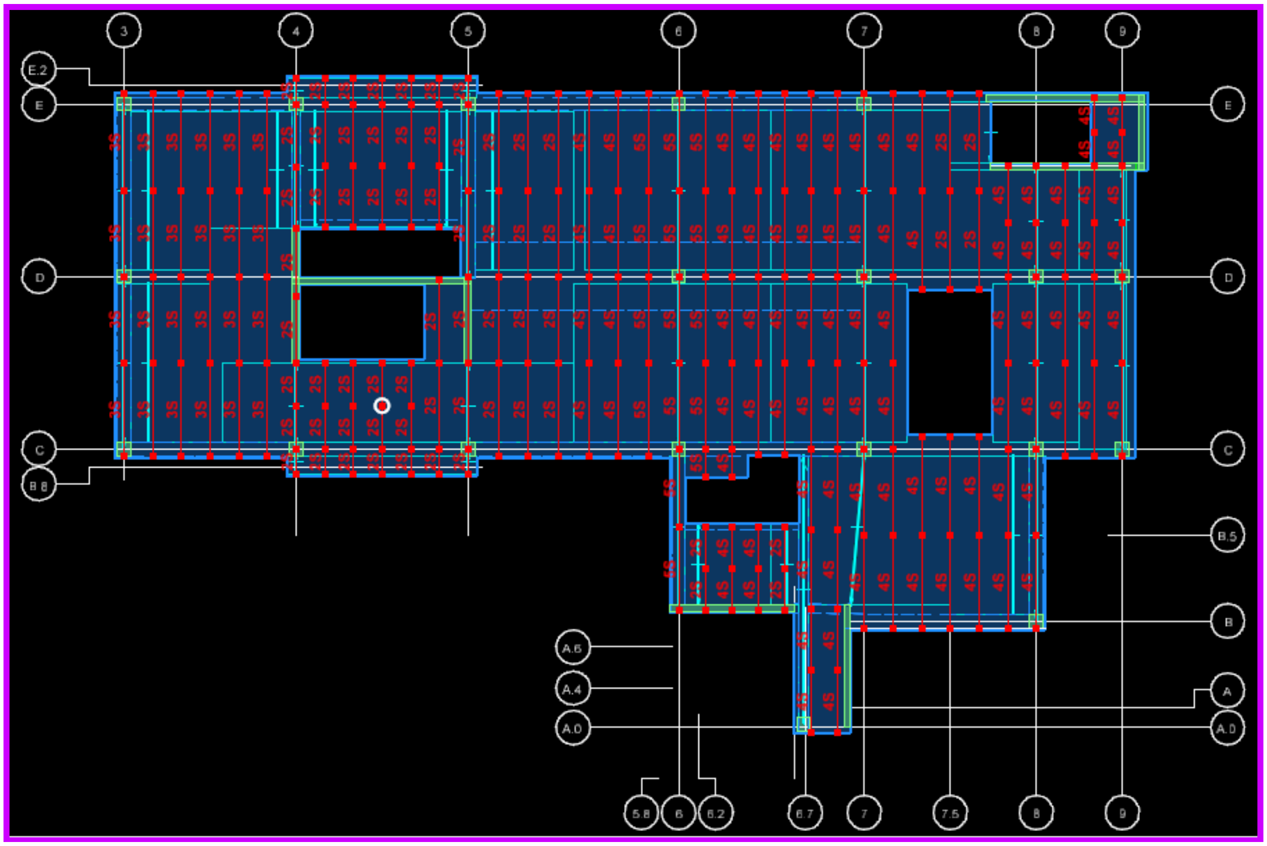


Figure 35: Final Layout of Latitude Tendons

Conclusion

By using a post-tensioned floor slab, the slab thickness was successfully reduced from 10”, used in the conventionally reinforced floor system, to 8”. This reduction in thickness would decrease the quantity of concrete required and increase the floor-to-ceiling heights.

Typically an increase in floor-to-ceiling height is beneficial, but in the New Library the floor-to-ceiling heights are not dictated by typical factors. Instead, the levels are based on the topography of the hillside, making the floor-to-floor heights 16’-0” to 18’-0”. Thus, a 2” increase in floor-to-ceiling heights in the New Library is not significantly beneficial.

Although the decrease in slab thickness would decrease the overall cost of the system, the additional costs and complications associated with the PT system would most likely outweigh the savings. These complications include the pour strips required due to the location of the shear walls, and the detailing of the slip joints required due to the foundation walls.

So, although is it possible to use a post-tensioned slab in the New Library, it was determined that is not recommended based on the minimal savings and additional complications.

Lateral Analysis

Once the structural system was changed from steel to concrete it was expected that the weight of the building would increase, thus increasing the seismic forces on the structure. It was decided that the lateral system should be re-analyzed under these increased forces to determine if adjustment needed to be made. Wind forces were also recalculated using ASCE7-10. The recalculated wind and seismic loads can be seen in Appendix H.

For Technical Report 4, a 3D ETABS model was created and the New Library's Lateral System was analyzed under wind and seismic loads calculated using ASCE7-05. Modeling decisions, verification of the ETABS model, and tables showing the building properties can be seen in Appendix I.

Overview

The lateral system used in the New Library is ordinary reinforced shear walls. The seven individual shear walls are shown in red in **Figure 36** below, while the diaphragms for each level are shown in gray. These seven shear walls can be located on the floor plan, Figure I1, provided in Appendix I where they are numbered 1-7. All of the shear walls are 12", with the exception of shear wall 1 and 2 which are 16" and 33" at the base level.

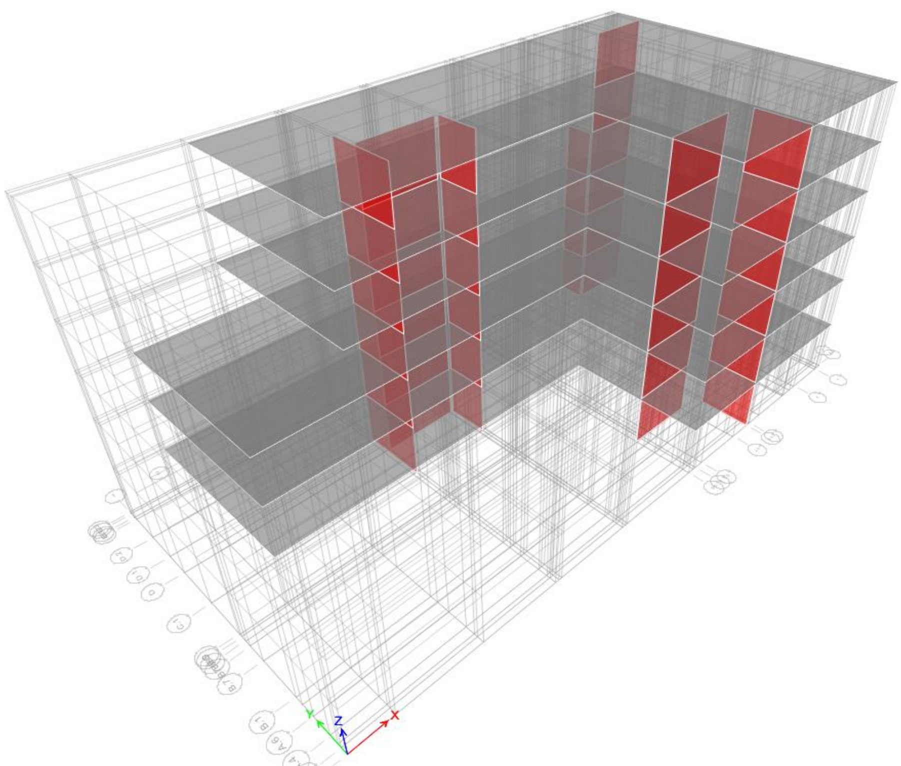


Figure 36: 3D View of ETABS Model

Wind Loads

Wind loads were recalculated using ASCE7-10. The four different wind cases to be applied to the building in order to account to quartering winds and torsion effects still applied. **Table 16-24** below show the resulting forces for each wind, along with **Figure 37 – 40** which show the corresponding images from ASCE7-10.

Note: ASCE7-10 requires a minimum wind pressure of *16 PSF* *in the windward direction.*

Case 1:

Wind Pressures (E-W Direction)					
Floor Height	Wall Length	Windward Pressure (PSF)	Leeward Pressure (PSF)	Trib Area (SF)	Force (K)
102	94.33	19.03	-9.49	849	24
84	94.33	18.07	-9.49	1604	44
68	94.33	17.11	-9.49	1509	40
52	94.33	16.00	-9.49	1509	38
36	94.33	16.00	-9.49	1604	41
18	94.33	16.00	-9.49	1698	43
			Base Shear=		231

Table 16: Case 1: Wind Pressures (E-W Direction)

Wind Pressures (N-S Direction)					
Floor Height	Wall Length	Windward Pressure (PSF)	Leeward Pressure (PSF)	Trib Area (SF)	Force (K)
102	121.67	18.78	-12.07	1095	34
84	121.67	17.83	-12.07	2068	62
68	121.67	16.88	-12.07	1947	56
52	147	16.00	-12.07	2352	66
36	147	16.00	-12.07	2499	70
18	147	16.00	-12.07	2646	74
			Base Shear=		362

Table 17: Case1: Wind Pressures (N-S Direction)

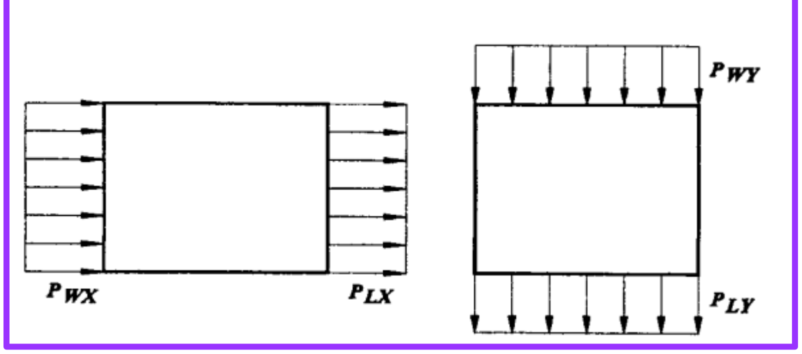


Figure 37: Wind Load Case 1

Case 2:

Wind Pressures (E-W Direction)								
Floor Height	Wall Length	Windward Pressure (PSF)	Leeward Pressure (PSF)	Trib Area (SF)	Force (K)	B_x (FT)	$(\pm)e_x$ (FT)	$(\pm)M_x$ (Ft-K)
102	94.33	16.00	-7.12	849	20	94.33	14.1	278
84	94.33	16.00	-7.12	1604	37	94.33	14.1	525
68	94.33	16.00	-7.12	1509	35	94.33	14.1	494
52	94.33	16.00	-7.12	1509	35	94.33	14.1	494
36	94.33	16.00	-7.12	1604	37	94.33	14.1	525
18	94.33	16.00	-7.12	1698	39	94.33	14.1	555
				Base Shear=	203			

Table 18: Case 2: Wind Pressures (E-W Direction)

Wind Pressures (N-S Direction)								
Floor Height	Wall Length	Windward Pressure (PSF)	Leeward Pressure (PSF)	Trib Area (SF)	Force (K)	B_y (FT)	$(\pm)e_y$ (FT)	$(\pm)M_y$ (Ft-K)
102	121.67	16.00	-9.05	1095	27	121.67	18.3	501
84	121.67	16.00	-9.05	2068	52	121.67	18.3	946
68	121.67	16.00	-9.05	1947	49	121.67	18.3	890
52	147	16.00	-9.05	2352	59	147	22.1	1299
36	147	16.00	-9.05	2499	63	147	22.1	1381
18	147	16.00	-9.05	2646	66	147	22.1	1462
				Base Shear=	316			

Table 19: Case 2: Wind Pressures (N-S Direction)

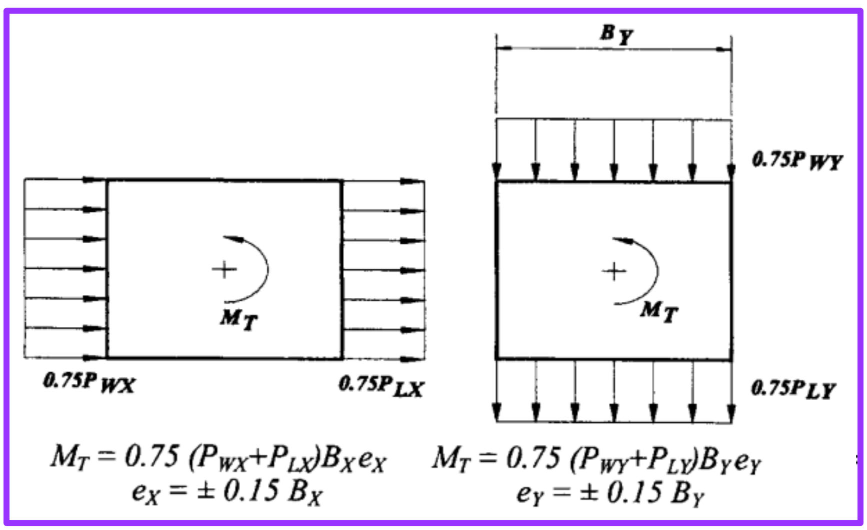


Figure 38: Wind Load Case 2

Case 3:

Note: Pressures in the X-Direction and Y-Direction are applied simultaneously.

Wind Pressures (E-W Direction)					
Floor Height	Wall Length	Windward Pressure(PSF)	Leeward Pressure (PSF)	Trib Area (SF)	Force (K)
102	94.33	16.00	-7.12	849	20
84	94.33	16.00	-7.12	1604	37
68	94.33	16.00	-7.12	1509	35
52	94.33	16.00	-7.12	1509	35
36	94.33	16.00	-7.12	1604	37
18	94.33	16.00	-7.12	1698	39
				Base Shear=	203

Table 20: Case 3: Wind Pressures (E-W Direction)

Wind Pressures (N-S Direction)					
Floor Height	Wall Length	Windward Pressure (PSF)	Leeward Pressure (PSF)	Trib Area (SF)	Force (K)
102	121.67	16.00	-9.05	1095	27
84	121.67	16.00	-9.05	2068	52
68	121.67	16.00	-9.05	1947	49
52	147	16.00	-9.05	2352	59
36	147	16.00	-9.05	2499	63
18	147	16.00	-9.05	2646	66
				Base Shear=	316

Table 21: Case 3: Wind Pressures (N-S Direction)

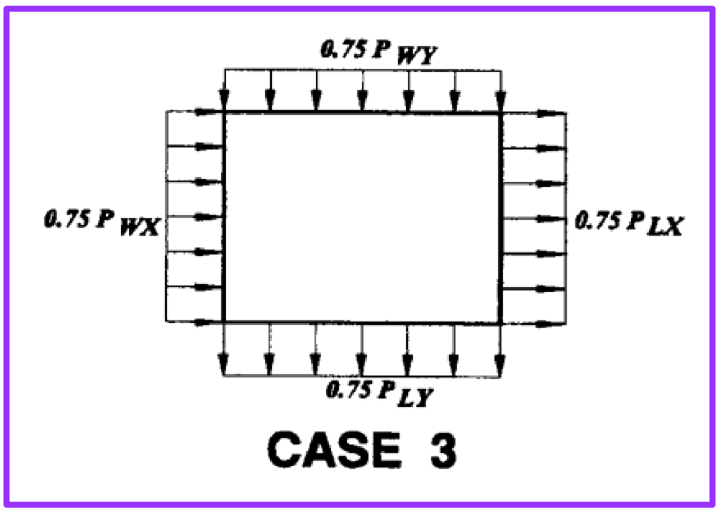


Figure 39: Wind Load Case 3

Case 4:

Note: Moments in the X-Direction and Y-Direction are applied at the same time.

Wind Pressures (E-W Direction)					
Floor Height	Wall Length	Windward Pressure (PSF)	Leeward Pressure (PSF)	Trib Area (SF)	Force (K)
102	94.33	16.00	-5.34	849	18
84	94.33	16.00	-5.34	1604	34
68	94.33	16.00	-5.34	1509	32
52	94.33	16.00	-5.34	1509	32
36	94.33	16.00	-5.34	1604	34
18	94.33	16.00	-5.34	1698	36
				Base Shear=	187

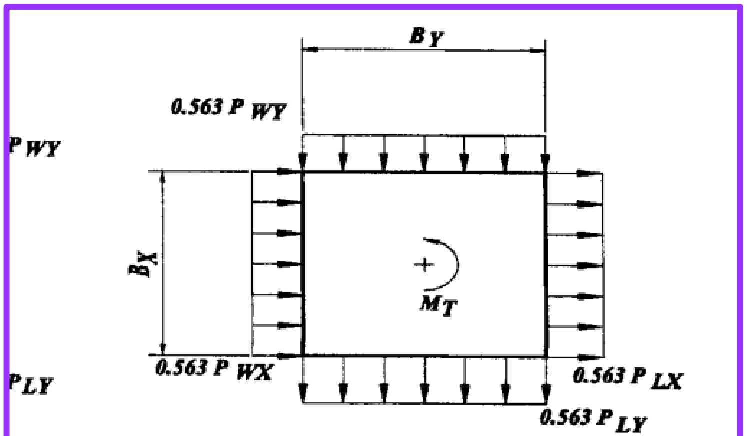
Table 22: Case 4: Wind Pressures (E-W Direction)

Wind Pressures (N-S Direction)					
Floor Height	Wall Length	Windward Pressure (PSF)	Leeward Pressure (PSF)	Trib Area (SF)	Force (K)
102	121.67	16.00	-6.80	1095	25
84	121.67	16.00	-6.80	2068	47
68	121.67	16.00	-6.80	1947	44
52	147	16.00	-6.80	2352	54
36	147	16.00	-6.80	2499	57
18	147	16.00	-6.80	2646	60
				Base Shear=	287

Table 23: Case 4: Wind Pressures (N-S Direction)

Wind Pressures (N-S Direction)								
Floor Height	Force - X (K)	Force - Y (K)	B_x (FT)	$(\pm)e_x$ (FT)	B_y (FT)	$(\pm)e_y$ (FT)	$(\pm)$ M of same sign	$(\pm)$ M of opposite sign
102	18	25	94.33	14.1	121.67	18.3	712	199
84	34	47	94.33	14.1	121.67	18.3	1345	376
68	32	44	94.33	14.1	121.67	18.3	1266	354
52	32	54	94.33	14.1	147	22.1	1638	726
36	34	57	94.33	14.1	147	22.1	1740	772
18	36	60	94.33	14.1	147	22.1	1843	817

Table 24: Case 4: Wind Pressures (N-S Direction)



$$M_T = 0.563\,(P_{WX}+P_{LX})B_X e_X + 0.563\,(P_{WY}+P_{LY})B_Y e_Y$$
$$e_X = \pm\,0.15\,B_X \qquad e_Y = \pm\,0.15\,B_Y$$

Figure 40: Wind Load Case 4

Seismic Forces

The increased building weight due to the new concrete structural system was calculated and the seismic forces calculated in Technical Report 4 were adjusted accordingly. **Table 25** and **Table 26** below show both the applied forces and moments.

Code Provisions to Note:

-ASCE7-10 Section 12.8.4.2 requires that when the diaphragm is not flexible, the design for seismic forces shall include the accidental torsional moment caused by the assumed displacement of the center of mass each way from its actual location by a distance equal to 5 percent of the dimension of the structure perpendicular to the direction of the applied forces.

-Due to the fact that the building is assigned to seismic category B, ACSE7-10 Section 12.5.2 states that seismic forces are permitted to be applied independently in each of two orthogonal directions and orthogonal interaction effects are permitted to be neglected.

-The torsion amplification factor Ax is also taken as 1.0 due to the building being assigned to seismic category B.

Calculation of Story Forces (E-W Direction)										
Level	Height (FT)	Total Weight (K)	$w_i h_i^k$ (K-FT)	C_{vx}	f_i (K)	V_i (K)	B_y (Ft)	5% B_y (FT)	A_x (K)	M_z (Ft-K)
Roof	102	2410	963012	0.210	129.0	129.0	94.3	4.7	1.0	609
6	84	3238	1077339	0.234	144.3	273.4	94.3	4.7	1.0	681
5	68	3305	871800	0.190	116.8	390.2	94.3	4.7	1.0	551
4	52	3647	722020	0.157	96.7	486.9	94.3	4.7	1.0	456
3	36	4602	621328	0.135	83.2	570.2	94.3	4.7	1.0	393
2	18	5356	340687	0.074	45.6	615.8	94.3	4.7	1.0	215
	SUM:	22558	4596186	1.000	615.8					2904

Table 25: Seismic Forces E-W Direction

Calculation of Story Forces (N-S Direction)										
Level	Height (FT)	Total Weight (K)	$w_i h_i^k$ (K-FT)	C_{vx}	f_i (K)	V_i (K)	B_x (Ft)	5% B_x (FT)	A_x (K)	M_z (Ft-K)
Roof	102	2410	963012	0.210	69.4	69.4	147.0	7.4	1.0	510
6	84	3238	1077339	0.234	77.7	147.1	147.0	7.4	1.0	571
5	68	3305	871800	0.190	62.8	209.9	147.0	7.4	1.0	462
4	52	3647	722020	0.157	52.0	262.0	121.7	6.1	1.0	317
3	36	4602	621328	0.135	44.8	306.7	121.7	6.1	1.0	272
2	18	5356	340687	0.074	24.6	331.3	121.7	6.1	1.0	149
	SUM:	22558	3717431	1.000	331.3					2281

Table 26: Seismic Forces N-S Direction

Lateral Earth Pressures

The Library at the University of Wise Virginia's College at Wise has a unique feature in which it is integrated into the existing 60 foot hillside. For this technical report the impact of the soil loads on the structures lateral system were considered using an equivalent fluid pressure of 47 pcf provided in the geotechnical report. **Tables 27 -29** below show the applied soil loads.

It is also important to note that in this report the soil loads were strictly used as an applied lateral load. They do not serve a role in aiding the building it terms of drift control, and were not considered to be causing drift for this analysis.

East Elevation:

Lateral Soil Forces(K)				
Level	Column Line A-C	Column Line C-C1	Column Line C1-D1	Column Line D-E2
5	0	0	0	42
4	62	54	105	338
3	541	224	279	729
2	1179	427	485	1188

Table 27: Seismic Forces E-W Direction

North Elevation:

Lateral Soil Forces(K)					
Level	Column Line 1-3	Column Line 3-5	Column Line 5-7	Column Line 7-8	Column Line 8-9.2
5	0	0	0	152	25
4	0	0	0	496	201
3	0	0	111	900	446
2	72	386	888	152	705

Table 28: Seismic Forces E-W Direction

South Elevation:

Lateral Soil Forces(K)					
Level	Column Line 1-3	Column Line 3-6	Column Line 6-7	Column Line 7-8	Column Line 8-9.2
5	0	0	0	0	0
4	0	0	0	0	25
3	0	0	52	172	113
2	0	155	416	557	127

Table 29: Seismic Forces E-W Direction

Member Spot Checks for Strength

As part of the lateral analysis of the New Library the shear walls that were spot checked in Technical Report 4 were re-checked under the increased wind and seismic loads. **Figure 41** and **Figure 42** below show the shear diagrams for shear wall 2 and 6.

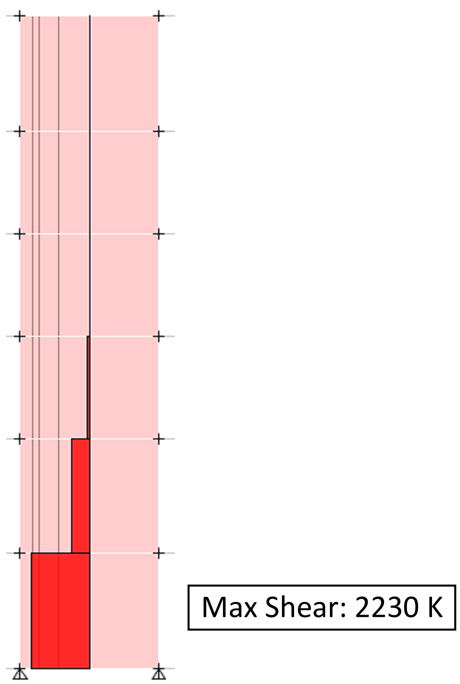


Figure 41: Moment Diagram for Shear Wall 2

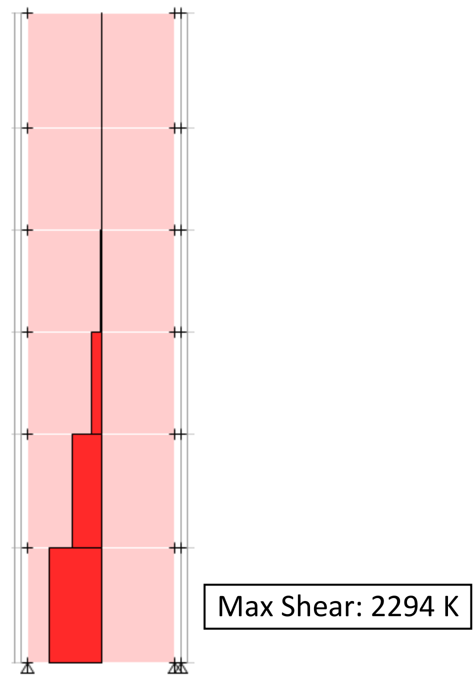


Figure 42: Moment Diagram for Shear Wall 6

The largest shear forces in both the x-direction and y-direction were still caused by soil loads. Thus, soil loads dictated the controlling load combination for the analysis of the shear walls. It was determined that load combination 7 from IBC 2012 was still the controlling combination due to the increased seismic forces being larger than the increased wind forces:

$$0.9D + 1.0E + 1.6H$$

The following pages show the hand calculations for determining the controlling load combination along with the member spot checks.

Hand Check Shear Wall 2

Determine Controlling Load Case

(2) $1.2D + 1.0E + L + 0.2S + 1.6H$

(6) $0.9D + 1.0W + 1.6H$

(7) $0.9D + 1.0E + 1.6H$

$\Rightarrow$ Load case (7) will control b/c soil is the controlling lateral forces and seismic is the second largest lateral forces

Determining Dead Load

Roof = $[(146\text{ psf})(335\text{ ft}^2)]/1000 = 48.9^k$
Level 6 = $[(162\text{ psf})(129.2\text{ ft}^2) + (21.667)(1)(18)(150)]/1000 = 79.4^k$
Level 5 = $[(162\text{ psf})(129.2\text{ ft}^2) + (21.667)(1)(16)(150)]/1000 = 72.9^k$
Level 4 = $[(162\text{ psf})(129.2\text{ ft}^2) + (21.667)(1)(16)(150)]/1000 = 72.9^k$
Level 3 = $[(162\text{ psf})(129.2\text{ ft}^2) + (21.667)(1)(16)(150)]/1000 = 72.9^k$
Level 2 = $[(162\text{ psf})(129.2\text{ ft}^2) + (21.667)(32/12)(18)(150)]/1000 = 181.8^k$

Total $P = 528.8^k$

Determine Seismic Loads

$E = E_h - E_v$ (ASCE 7-10 12.4.2)

$E_h = Q_E \rho \Rightarrow \rho = 1.0$ for SDC B

$= 249$ (Seismic x-Direction (-M))

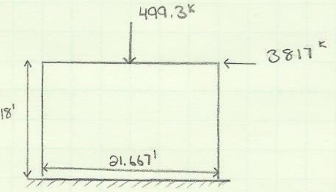
$E_v = 0.2 S_{DS} D$

$= 0.2(0.221)(528.8)$

$= 23.4^k$

Loads

$P_u = 0.9(528.8) + (23.4) = 499.3^k$
$V_u = 1.0(249) + 1.6(2230) = 3817^k$



$M = 1.0(20,291) + 1.6(49227) = 99,060'^k$

Check Strength of Shear Wall 2

$\phi V_n \geq V_u = 3817^k$

$V_n = V_c + V_s$

Determine Vc

$V_c = 3.3\lambda\sqrt{f'c}\,hd + (N_u d / 4 l_w)$ (ACI 318-11 Eq 11-27)

$\lambda = 1.0$

$f'c = 4000\,psi$

$h = 33''$

$d = 0.8 l_w = 208''$

$N_u = 528.8^k$

$= [3.3(1)\sqrt{4000}(33)(208) + (528.8(208)/4(260))]/1000$

$= 1432.7^k$

$V_c = \left[0.6\lambda\sqrt{f'c} + \dfrac{l_w(1.25\lambda\sqrt{f'c} + 0.2\,N_u/l_w h)}{\dfrac{M_u}{V_u} - \dfrac{l_w}{2}}\right] hd$ (eq 11-28)

* check to see if equation applies

$\dfrac{M_u}{V_u} - \dfrac{l_w}{2} = \dfrac{99060}{3817}(12) - \dfrac{260}{2} = 181.4'' > 0'' \Rightarrow$ Equation Applies

$= \left[0.6(1)\sqrt{4000} + \dfrac{260(1.25(1)\sqrt{4000} + 0.2(528.8/(260)(33)))}{181.4}\right](33)(208)/1000$

$= 1038.4^k$

$V_c = \min \begin{vmatrix} 1432.7 \\ 1038.4 \end{vmatrix} \Rightarrow 1038.4^k$

$\phi V_c = 0.75(1038.4) = 778.8 < 3817^k$

$\Rightarrow$ $\boxed{\text{Wall w/o shear reinforcement not adequate for shear strength.}}$

Determine V_s

$$V_s = \frac{A_v f_y d}{s}$$

* Reinforcement = #8 @ 18" each face

$$A_v = \left(\frac{18'(12)}{18''} + 1\right)(0.79)(2) = 20.54$$

$f_y = 60\text{ ksi}$
$d = 260''$
$S = 18''$

$$= \frac{(20.54)(60)(260)}{18}$$

$$= 17801^k$$

Determine if strength is adequate

$$\phi V_n = 0.75(1038.4 + 17801) = 13863.4$$

$$\Rightarrow 13863.4 > 3817^k \checkmark$$

$\Rightarrow$ Shear wall w/ shear reinforcement adequate for shear strength under increased seismic loads.

Hand Check Shear Wall 6

Determine Controlling Load Case

(7) $0.9D + 1.0E + 1.6H$

Determine Dead Loads

Roof $= [(146psf)(296.37ft^2)/1000] = 43.3^K$
Level 6 $= [(162psf)(18.02ft^2) + (23.33)(1)(18)(150)]/1000 = 65.9^K$
Level 5 $= [(162psf)(18.02ft^2) + (23.33)(1)(16)(150)]/1000 = 58.9^K$
Level 4 $= [(162psf)(18.02ft^2) + (23.33)(1)(16)(150)]/1000 = 58.9^K$
Level 3 $= [(162psf)(18.02ft^2) + (23.33)(1)(16)(150)]/1000 = 58.9^K$
Level 2 $= [(162psf)(18.02ft^2) + (23.33)(1)(18)(150)]/1000 = 65.9^K$

Total $P = 351.8^K$

Determine Seismic Loads

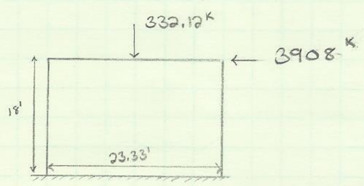
$E = E_h - E_v$

$E_h = Q_E \rho$
$= 238$

$E_v = 0.2 S_{DS} D$
$= 0.2(0.221)(351.8)$
$= 15.5^K$

Loads

$P_u = 0.9(351.8) + (15.5) = 332.12^K$
$V_u = 1.0(238) + 1.6(2294) = 3908^K$



$M = 1.0(5811.8) + 1.6(49227) = 84575^{\prime K}$

Check Strength of Shear Wall 6

$\phi V_n > V_u = 3908^k$

$V_n = V_c + V_s$

Determine V_c

$V_c = 3.3\lambda\sqrt{f'c}\,hd + (N_u d / 4 l_w)$

$\lambda = 1.0$

$f'c = 4000psi$

$h = 12"$

$d = 224"$

$N_u = 351.8$

$= [3.3(1)\sqrt{4000}(12)(224) + (332.12(224)/4(280))]/1000$

$= 561.1^k$

$$V_c = \left[0.6\lambda\sqrt{f'c} + \frac{l_w(1.25\lambda\sqrt{f'c} + 0.2N_u/l_w h)}{\frac{M_u}{V_u} - \frac{l_w}{2}}\right]hd$$

* Check to see if eqn applies

$\frac{M_u}{V_u} - \frac{l_w}{2} = \frac{84575}{3908}(12) - \frac{280}{2} = 119.7 > 0$ Equation Applies

$$= \left[0.6(1)\sqrt{4000} + \frac{280(1.25(1)\sqrt{4000} + 0.2(352)/(280)(12))}{119.7}\right](12)(224)/1000$$

$= 599^k$

$V_{c,min} = \begin{vmatrix} 561.1 \\ 599^k \end{vmatrix} = 561.1^k$

$\phi V_c = (0.75)(561.1) = 421^k < 3908^k$

$\Rightarrow$ wall w/o shear reinforcement not adequate for shear strength

Determine Vs

$$V_s = \frac{A_v f_y d}{s}$$

Reinforcement = #5 @ 18"

$$A_v = \left(\frac{18'(12)}{18} + 1\right)(0.31)(2) = 8.06$$

$f_y = 60\text{ ksi}$
$d = 224\text{ in}$

$$= \frac{(8.06)(60)(224)}{18}$$
$$= 6018^k$$

Determine if strength is adequate

$$\phi V_n = 0.75(561 + 6018) = 4934$$

$$\Rightarrow 4934^k > 3908^k \checkmark$$

$\Rightarrow$ $\boxed{\text{Shear wall w/ reinforcement adequate for shear strength under increased Seismic Loads.}}$

Drift Checks

Drift Due to Wind

The maximum drift of the structure under service wind loads was checked based on the industry accepted value of H/400. The overall height of the structure is 102 feet resulting in an allowable drift of 3.06 in. **Table 30** below shows the maximum drift due to each load case produced by the ETABS model and its comparison to the standard. It is important to note that to determine the service loads the original ultimate wind loads were divided by a factor of 1.6.

Wind Load Cases			
Load Case	Maximum Drift (in)	Allowable Drift (in)	Pass/Fail
Wind Case 1 X-Direction	0.679	3.06	PASS
Wind Case 1 Y-Direction	2.321	3.06	PASS
Wind Case 2 X-Direction (+M)	0.853	3.06	PASS
Wind Case 2 X-Direction (-M)	0.315	3.06	PASS
Wind Case 2 Y-Direction (+M)	2.656	3.06	PASS
Wind Case 2 Y-Direction (-M)	1.869	3.06	PASS
Wind Case 3	2.528	3.06	PASS
Wind Case 4 (+Moments in Same Direction)	2.735	3.06	PASS
Wind Case 4 (-Moments in Same Direction)	1.756	3.06	PASS
Wind Case 4 (+Moments in Opposite Direction)	2.607	3.06	PASS
Wind Case 4 (-Moments in Opposite Direction)	1.995	3.06	PASS

Table 30: Structure Drifts Due to Wind Loads

Discussion of Results:

The lateral system passes for all of the applied wind loading cases, and is adequate to resist the wind loads based on the serviceability criteria of H/400.

Drift Due to Seismic

A check of the maximum drift of the structure under service seismic loads was checked based on the ACSE7-10 Table 12.12-1. The building's occupancy category is category III which gives an allowable story drift of 1.5% of the story height below the given level.

Table 31 – 34 below compare the actual drift percentage given by the ETABS model to the 1.5% allowable drift including the application of the story drift amplification factor that can be found in ASCE7-10 Section 12.8.6. A C_d of 4 (ordinary reinforced shear walls) and an I_e of 1.25 (Risk Category III) were used in the calculation of the drift amplification factor.

Seismic: X-Direction Loading (+Eccentricity)											
Story	Story Height	Story Drift X-Dir.	Story Drift w/ Amp. Factor	Story Drift (%)	Allow. Drift (%)	Pass/Fail	Story Drift Y-Dir.	Story Drift w/ Amp. Factor	Story Drift (%)	Allow. Drift (%)	Pass/Fail
Roof	18	0.00246	0.0079	0.79	1.5	PASS	0.00236	0.007549	0.75	1.5	PASS
6	16	0.00239	0.0076	0.76	1.5	PASS	0.00228	0.007283	0.73	1.5	PASS
5	16	0.00219	0.0070	0.70	1.5	PASS	0.00206	0.006602	0.66	1.5	PASS
4	16	0.00181	0.0058	0.58	1.5	PASS	0.00166	0.005312	0.53	1.5	PASS
3	18	0.00115	0.0037	0.37	1.5	PASS	0.00100	0.003190	0.32	1.5	PASS
2	18	0.00035	0.0011	0.11	1.5	PASS	0.00026	0.000832	0.08	1.5	PASS

Table 31: Story Drifts Due to Seismic Loads (X-Direction, +M)

Seismic: X-Direction Loading (-Eccentricity)											
Story	Story Height	Story Drift X-Dir.	Story Drift w/ Amp. Factor	Story Drift (%)	Allow. Drift (%)	Pass/Fail	Story Drift Y-Dir.	Story Drift w/ Amp. Factor	Story Drift (%)	Allow. Drift (%)	Pass/Fail
Roof	18	0.00159	0.0051	0.51	1.5	PASS	0.00121	0.00386	0.39	1.5	PASS
6	16	0.00154	0.0049	0.49	1.5	PASS	0.00116	0.00370	0.37	1.5	PASS
5	16	0.00141	0.0045	0.45	1.5	PASS	0.00104	0.00331	0.33	1.5	PASS
4	16	0.00116	0.0037	0.37	1.5	PASS	0.00081	0.00258	0.26	1.5	PASS
3	18	0.00073	0.0023	0.23	1.5	PASS	0.00044	0.00140	0.14	1.5	PASS
2	18	0.00025	0.0008	0.08	1.5	PASS	0.00011	0.00034	0.03	1.5	PASS

Table 32: Story Drifts Due to Seismic Loads (X-Direction, -M)

Seismic: Y-Direction Loading (+Eccentricity)											
Story	Story Height	Story Drift X-Dir.	Story Drift w/ Amp. Factor	Story Drift (%)	Allow. Drift (%)	Pass/Fail	Story Drift Y-Dir.	Story Drift w/ Amp. Factor	Story Drift (%)	Allow. Drift (%)	Pass/Fail
Roof	18	0.00171	0.0055	0.55	1.5	PASS	0.00400	0.0147	1.47	1.5	PASS
6	16	0.00165	0.0053	0.53	1.5	PASS	0.00399	0.0146	1.46	1.5	PASS
5	16	0.00151	0.0048	0.48	1.5	PASS	0.00388	0.0135	1.35	1.5	PASS
4	16	0.00124	0.0040	0.40	1.5	PASS	0.00355	0.0114	1.14	1.5	PASS
3	18	0.00077	0.0024	0.24	1.5	PASS	0.00239	0.0077	0.77	1.5	PASS
2	18	0.00022	0.0007	0.07	1.5	PASS	0.00085	0.0027	0.27	1.5	PASS

Table 33: Story Drifts Due to Seismic Loads (Y-Direction, +M)

Seismic: Y-Direction Loading (-Eccentricity)											
Story	Story Height	Story Drift X-Dir.	Story Drift w/ Amp. Factor	Story Drift (%)	Allow. Drift (%)	Pass/Fail	Story Drift Y-Dir.	Story Drift w/ Amp. Factor	Story Drift (%)	Allow. Drift (%)	Pass/Fail
Roof	18	0.00098	0.00314	0.31	1.5	PASS	0.00376	0.0120	1.20	1.5	PASS
6	16	0.00095	0.00304	0.30	1.5	PASS	0.00366	0.0117	1.17	1.5	PASS
5	16	0.00087	0.00277	0.28	1.5	PASS	0.00338	0.0108	1.08	1.5	PASS
4	16	0.00071	0.00226	0.23	1.5	PASS	0.00285	0.0091	0.91	1.5	PASS
3	18	0.00043	0.00136	0.14	1.5	PASS	0.00194	0.0062	0.62	1.5	PASS
2	18	0.00012	0.00037	0.04	1.5	PASS	0.00070	0.0022	0.22	1.5	PASS

Table 34: Story Drifts Due to Seismic Loads (Y-Direction, -M)

Discussion of Results:

The lateral system passes for all of the applied seismic loading cases for the criteria of an allowable story drift of 1.5%. The worst case drift was located at the roof level and was due to the loading in the y-direction. This was expected due to the fact the forces in the y-direction are acting perpendicular to the long direction of the building, thus seeing less resistance to drift.

Stability Coefficient Check

According to ASCE7-10 Section 12.8.7 P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these forces and moments, and the story drifts induced by these effects are not required to be considered where the stability coefficient (ϴ), as determined by equation 12.8-16, is equal to or less than 0.1.

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d} \text{ (Eqn. 12.8-16)}$$

The stability coefficient for each seismic load case and each level of the structure was calculated and compared to 0.1. It was determined that all of the stability coefficients were less than 0.1 and P-delta effects did not need to be considered. **Tables 35 - 38** below show the calculated stability coefficients.

Seismic: X-Direction Loading (+Eccentricity)												
h_x (in)	P_x (K)	V_x (K)	Story Drift Ratio X-Direction	Story Drift with Amplification Factor	Story Drift (in)	Stability Coefficient (ϴ)	Less Than 0.1	Story Drift Ratio Y-Direction	Story Drift with Amplification Factor	Story Drift (in)	Stability Coefficient (ϴ)	Less Than 0.1
216	1716	129	0.00246	0.0079	1.70	0.0328	PASS	0.00236	0.00755	1.63	0.031	PASS
192	3519	273	0.00239	0.0076	1.47	0.0308	PASS	0.00228	0.00728	1.40	0.029	PASS
192	5342	390	0.00219	0.0070	1.35	0.0300	PASS	0.00206	0.00660	1.27	0.028	PASS
192	7304	487	0.00181	0.0058	1.11	0.0271	PASS	0.00166	0.00531	1.02	0.025	PASS
216	9475	570	0.00115	0.0037	0.79	0.0190	PASS	0.00100	0.00319	0.69	0.017	PASS
216	11680	616	0.00035	0.0011	0.24	0.0067	PASS	0.00026	0.00083	0.18	0.005	PASS

Table 35: Stability Coefficients Due to Seismic Loads (X-Direction, +M)

Seismic: X-Direction Loading (-Eccentricity)												
h_x	P_x (K)	V_x (K)	Story Drift Ratio X-Direction	Story Drift with Amplification Factor	Story Drift (in)	Stability Coefficient (ϴ)	Less Than 0.1	Story Drift Ratio Y-Direction	Story Drift with Amplification Factor	Story Drift (in)	Stability Coefficient (ϴ)	Less Than 0.1
216	1716	129	0.00159	0.0051	1.10	0.02	PASS	0.00121	0.0039	0.83	0.02	PASS
192	3519	273.3	0.00154	0.0049	0.94	0.02	PASS	0.00116	0.0037	0.71	0.01	PASS
192	5342	390.1	0.00141	0.0045	0.87	0.02	PASS	0.00104	0.0033	0.64	0.01	PASS
192	7304	486.8	0.00116	0.0037	0.71	0.02	PASS	0.00081	0.0026	0.49	0.01	PASS
216	9475	570	0.00073	0.0023	0.50	0.01	PASS	0.00044	0.0014	0.30	0.01	PASS
216	11680	615.6	0.00025	0.0008	0.17	0.00	PASS	0.00011	0.0003	0.07	0.00	PASS

Table 36: Stability Coefficients Due to Seismic Loads (X-Direction, -M)

Seismic: Y-Direction Loading (+Eccentricity)												
h_X	P_X (K)	V_X (K)	Story Drift Ratio X-Direction	Story Drift with Amplification Factor	Story Drift (in)	Stability Coefficient (Θ)	Less Than 0.1	Story Drift Ratio Y-Direction	Story Drift with Amplification Factor	Story Drift (in)	Stability Coefficient (Θ)	Less Than 0.1
216	1716	69	0.00171	0.0055	1.18	0.04	PASS	0.00400	0.0128	2.76	0.10	PASS
192	3519	147	0.00165	0.0053	1.02	0.04	PASS	0.00399	0.0128	2.45	0.10	PASS
192	5342	210	0.00151	0.0048	0.93	0.04	PASS	0.00388	0.0124	2.38	0.10	PASS
192	7304	262	0.00124	0.0040	0.76	0.03	PASS	0.00355	0.0114	2.18	0.10	PASS
216	9475	307	0.00077	0.0024	0.53	0.02	PASS	0.00239	0.0077	1.65	0.07	PASS
216	11680	331	0.00022	0.0007	0.15	0.01	PASS	0.00085	0.0027	0.59	0.03	PASS

Table 37: Stability Coefficients Due to Seismic Loads (Y-Direction, +M)

Seismic: Y-Direction Loading (-Eccentricity)												
h_X	P_X (K)	V_X (K)	Story Drift Ratio X-Direction	Story Drift with Amplification Factor	Story Drift (in)	Stability Coefficient (Θ)	Less Than 0.1	Story Drift Ratio Y-Direction	Story Drift with Amplification Factor	Story Drift (in)	Stability Coefficient (Θ)	Less Than 0.1
216	1716	69	0.00098	0.0031	0.68	0.02	PASS	0.00376	0.0120	2.60	0.09	PASS
192	3519	147	0.00095	0.0030	0.58	0.02	PASS	0.00366	0.0117	2.25	0.09	PASS
192	5342	210	0.00087	0.0028	0.53	0.02	PASS	0.00338	0.0108	2.08	0.09	PASS
192	7304	262	0.00071	0.0023	0.43	0.02	PASS	0.00285	0.0091	1.75	0.08	PASS
216	9475	307	0.00043	0.0014	0.29	0.01	PASS	0.00194	0.0062	1.34	0.06	PASS
216	11680	331	0.00012	0.0004	0.08	0.00	PASS	0.00070	0.0022	0.48	0.02	PASS

Table 38: Stability Coefficients Due to Seismic Loads (Y-Direction, -M)

Conclusion

The chosen lateral system for the New Library is ordinary reinforced concrete shear walls. It was determined that a lateral system redesign was unnecessary due to the fact that the new gravity system was designed in concrete and in several locations the shear walls aid in resisting lateral soil forces. Since the lateral system was not redesigned it was still important to verify that the members were adequate to resist the increased loads due to the new concrete gravity system. Initial wind and seismic loads were also recalculated using ASCE7-10.

After adjusting the loads, ETABS was used to distribute the forces to the shear walls and hand checks were completed for two of the walls. Based on the member spot checks, it was determined that the lateral system was adequate to resist the increased loads. Drift checks were also completed. The lateral system met serviceability requirements for max building drift due to wind loads and story drift due to seismic loads.

Based on this lateral analysis, the lateral system used in the New Library is adequate for both strength and serviceability under the increased loads.

MAE Coursework Integration

Coursework requirements for the MAE were integrated into several portions of this thesis. This was done through the use of computer modeling. Skills and knowledge of computer modeling were gained in AE 530, *Computer Modeling of Building Structures*, and through guided self-studies completed during the spring semester.

Both gravity system designs for the New Library were completed using RAM Concept, a structural analysis and design program used for elevated concrete slabs and mat foundations. RAM Concept was chosen due to the fact that it is one of the most efficient programs for designing concrete floor systems, and was highly recommended by professionals in industry. RAM Concept was learned through a guided self-study, and was used for modeling the conventionally reinforced concrete slab and post-tensioned slab. This self-study was completed through the completion of modeling tutorials provided on Bentley's website and in the RAM Concept user manual. Additional assistance was also provided by Heather Sustersic, my thesis advisor, and Walid Choueiri, Principle at SK&A Engineers.

The lateral system for the New Library was verified using the help of ETABS, a structural analysis program designed specifically for buildings. ETABS was one of the main modeling programs studied in AE530, and was used throughout the analysis of the lateral system in both the fall and spring semester. Wind and seismic loads were calculated by hand and then input into ETABS. ETABS then accurately distributed the forces to the members of the lateral force resisting system, comprised of ordinary reinforced shear walls. These forces were then used to verify the adequacy of the system.

As learned in AE 530, using a computer program as a "black box" with no knowledge of the topic can be dangerous and highly unethical. Therefore, results should always be checked to ensure that the program and the inputs are producing accurate results. The RAM Concept model was verified by hand and with the help of additional analysis programs such as SP Slab. These verifications can be seen in Appendix C. The ETABS model was verified using 2D hand analysis, and comprehension of modeling output. These verifications can be seen in Appendix I.

Breadth 1: Drainage System Study

One of the unique design features of the New Library at the University of Virginia's College at Wise is that it is to be integrated into a 60' hillside located in the middle of campus. This large grade difference raises a concern for water infiltration both at the base of the foundation walls and underneath the slab on grade. **Figure 43** shows an elevation view of the building and the depth of the hillside.

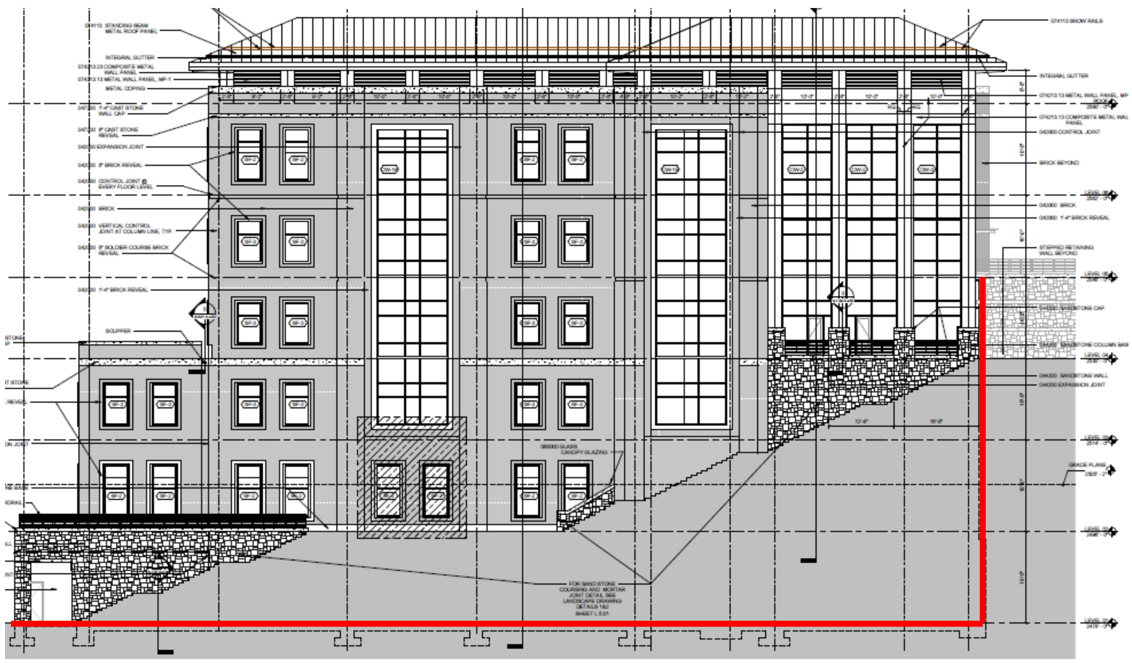


Figure 43: South Elevation Showing Depth of Footings (From Project Documents)

The current drainage system utilizes 8" diameter gravity drainage pipes at the base of the foundation walls and 4" diameter gravity drainage pipes located underneath the slab-on-grade. These drainage pipes empty into existing storm drain lines. It was desired that this drainage system be investigated and the size of the drainage pipe be verified. Water proofing for both the foundation walls and the slab were also designed.

Waterproofing the Wall

The drainage pipe is just one piece of the water proofing system that is required to guide the water away from the foundation wall. **Figure 44** below shows a schematic of the water proofing system that has been designed to direct the water to the drainage pipe.

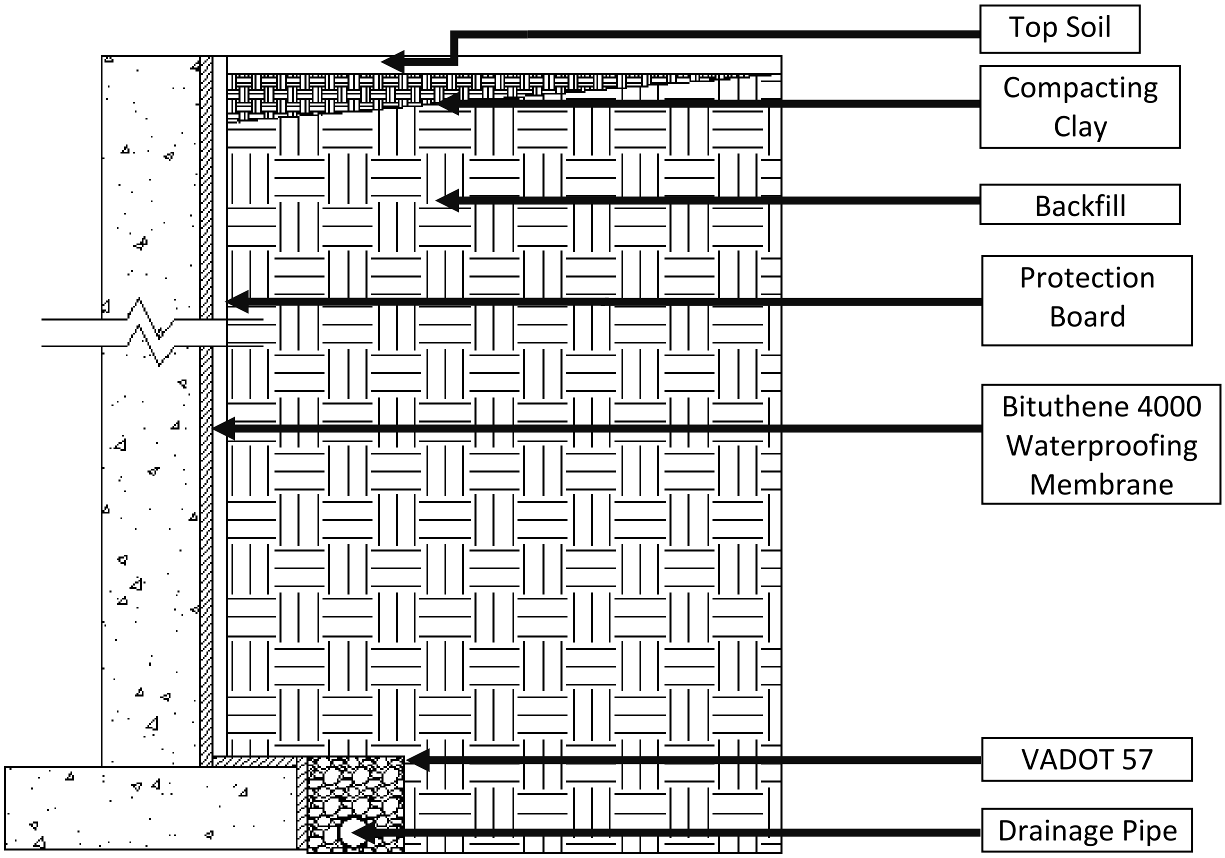


Figure 44: Water Proofing System

The waterproofing membrane chosen was the Bituthene System 4000. This waterproofing membrane is produced by WR Grace who has been producing construction materials for the international construction industry for more than half a century. WR Grace's Bituthene waterproofing membranes have been used in a number of international projects including the New Terminal at the Las Vegas McCarran International Airport.

The System 4000 was chosen specifically for its excellent adhesion to the wall and its ability to reduce inventory and handling costs. The adhesion is achieved through the use of the System 4000 Surface Conditioner. This conditioner is a "water-based, latex surface treatment which imparts an aggressive, high tack finish to the treated substrate. It is specifically formulated to bind site dust and concrete efflorescence, thereby providing a suitable surface for the Bituthene System 4000 Waterproofing Membrane"- Tech Sheet. This conditioner is packaged in each roll of membrane which is the reason the system reduces inventory and handling costs. For more information on this system see the Tech Sheet in Appendix J.1.

Water Path

Figure 45 shows the path the water will take as it passes through the soil and into the drainage pipe.

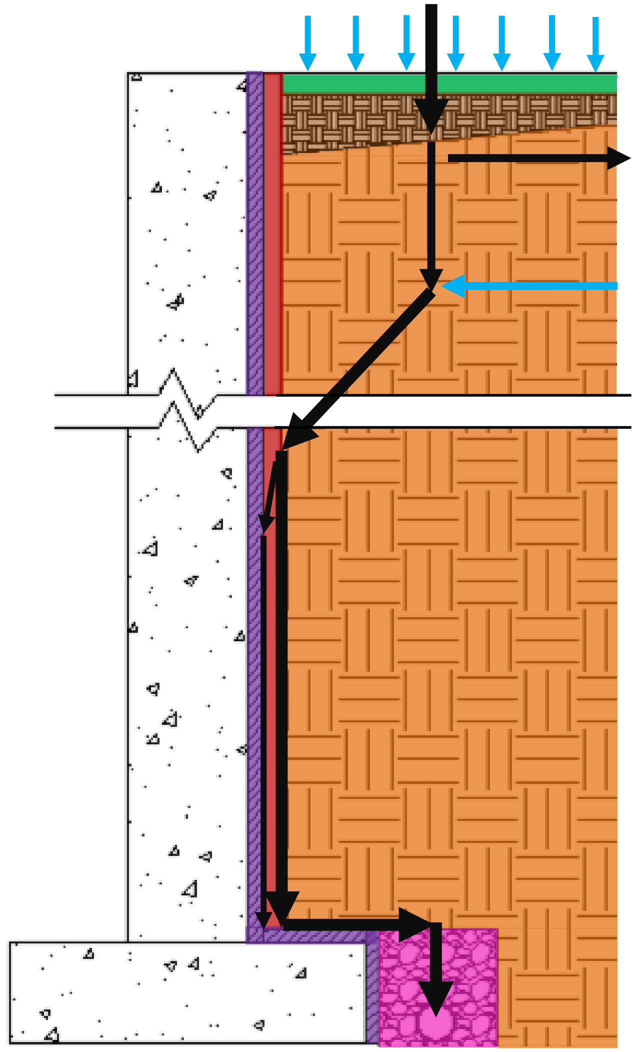


Figure 45: Water Path

1. Surface water enters though the **top soil**.

2. Water will then enter into a layer of **compacting clay**. This layer of clay is 10" – 12" thick at the wall and will thin out to just top soil approximately 12'-20' away from the building. The compact clay limits the amount of water that will pass into the backfill, and will direct the water away from the building.

3. Any water that penetrates the compact clay layer will enter into the **backfill** and will combine with existing ground water. The backfill is recommended by the geotechnical engineers on the project. This backfill should be comprised of full gradation soil with minimal fines. This will ensure voids in the fill to allow water to pass through.

4. Once in the backfill, the water goes into the **protection board**. This board is ½" thick and is comprised of a plastic layer and a geotextile membrane that faces away from the wall. The water enters the geotextile membrane and is directed down to the base of the wall.

   If the protection board fails, the water is stopped by the **Bituthene waterproofing membrane**. This material is a 1/16" thick waterproofing membrane that is ideal for waterproofing concrete structures below grade. This membrane will be applied to the wall and the footing. Once the water reaches this membrane it is directed down to the base of the wall.

5. At the base of the wall there is a drainage pipe that is set down in a 2'x2' trench of VADOT 57 stone, and this stone is then wrapped in a geotextile fabric. The geotextile fabric allows the water to pass through, but prevents the soil from passing through and clogging the drainage pipe. Once the water passes though the membrane it enters into the drainage pipe which will direct it to daylight. The size of the pipe is recommended by the geotechnical engineers.

Waterproofing the Slab

In order to prevent water infiltration through the slab a 2" thick concrete mud slab will be poured in place first. Mud slabs are beneficial on projects because they are much easier to deal with in wet weather conditions as compared to crushed stone or soil. They also provided a smooth, level surface for the floor slab.

WR Grace Preprufe 300R Plus membrane will be installed between the mud slab and the floor slab. This membrane is approximately ½" thick and is laid adhesive side up so it is ready to bond to the concrete floor slab. This forms a permanent, seamless seal against ground water. Preprufe 300R Plus is specifically designed to be used below slabs and its high tensile strength provides resistance against the stress of ground settlement. For more information on this membrane see the Tech Sheet in Appendix J.2.

Determining the Type of Drainage Required

Test borings for the site were completed by S&ME, Inc. to determine if water was present and at what elevation. If the elevation of the water is higher than the bottom of the footing then ground water will be a concern. If not, the drainage pipe will be strictly for the removal of surface water. At the time of boring, all bore holes were dry, but 48-72 hours later all of the holes except hole 5 (which was filled in) were found to have water in them. **Table 39** below shows the elevation of the water at the relevant boring locations along with the elevation of the bottom of the footing. A plan view of the locations of borings 3-8 can be seen in Figure J1 in Appendix J.3.

Based on this information, ground water at location 6, 7, and 8 will be a concern. This was to be expected based on the fact that all three of these locations are on the east side of the building which will be built into the existing hillside.

Compare Depth of Footing to Water Level Measurements				
Boring Number	Location	Top of Footing	Bottom of Footing	Elevation of Water Level
B-1	Outside of building footprint - West side	-	-	2484.0
B-2	Outside of building footprint - West side	-	-	2463.5
B-3	Outside of building footprint - North-west side	2476.5	2474.83	2471.6
B-4	Inside of building footprint	2476.5	2474.83	2474.4
B-5	Inside of building footprint	2476.5	2474.83	-
B-6	Inside of building footprint	2474	2472.33	2494.3
B-7	Outside of building footprint - East side	2476.5	2474.25	2503.0
B-8	Outside of building footprint - East side	2476.5	2474.25	2511.0

Table 39: Depth of Footing vs. Water Level

It is also important to note that test boring 6 is located under the building. This means that it is possible that water may seep into the rock/soil under the slab-on-grade. Therefore, drainage pipes will also need to be located beneath the floor slab.

Determining the Flow Rate

The drainage pipe size depends on the amount of water in gallons per minute (gpm) that needs removed from the site. To determine this amount, both the water observed on the site and water due to expected rain fall was used.

Observed Site Water:

To determine the gpm of the site water the depth of the water observed was multiplied by the area of the bore hole (all holes were 3 ¼" in diameter), divided by the number of hours to fill to that level, and then converted to gpm. **Table 40** shows this calculation for test boring 6, 7, and 8. From this, the water level at B-8 gave the critical flow rate.

Flow Rate of Ground Water						
Boring Number	Depth (FT)	Area of Bore Hole (FT^2)	Depth * Area (FT^3)	Number of Hours	Flow Rate (FT^3/HR)	Flow Rate (gpm)
B-6	22	8.29	182.4	48	4	0.4738
B-7	46	8.29	381.4	72	5	0.6604
B-8	59	8.29	489.2	72	7	0.8470

Table 40: Flow Rate of Ground Water

Rain Water:

Appendix B of the International Plumbing Code 2012 gives average rainfall rates for Virginia. The average rainfall for Bristol, VA, which was the closest to Wise, VA, is listed as 2.7 in/hr or 0.028 gpm/SF.

An approximated tributary area from the building foundation wall to 10' away from the structure was used to calculate the total gpm from rainfall. This dimension was based on approximately half the distance from the foundation wall to the surrounding storm drain. This area around the perimeter of the building was approximately 2870 SF. This gives 80.4 gpm. If this rainfall is divided between two main pipes (one for each side of the building) then each pipe is expected to handle 40.2 gpm.

Including the rainwater and ground water the total gpm is approximately 41 gpm. This flow rate is relatively small, so a sump pump will not be used to remove the water. Instead gravity drainage pipes will be used to remove the water away from the site at a slope of 1%.

Determining the Size of the Pipe

Table 1102.5 of the IPC 2012 gives the allowable piping materials for drainage pipes (shown in Figure J.2 in Appendix J.4). Based on this information, perforated PVC piping was chosen as the piping for the drainage system.

To determine the required pipe size the nomograph for PVC pipe was used. This nomograph can be seen in Figure J.3 Appendix J.5.

Due to the flow being so minimal the minimum required pipe size is only 1 ¾", but IPC 2012 Section 1112.1 requires that the minimum drainage pipe size be at least 4". Therefore both the drainage pipes at the base of the foundation wall and under the slab-on-grade will be 4". This final pipe size shows that the existing drainage pipe design is adequate.

Final Pipe Design:

4" perforated PVC pipe at the base of the foundation walls
(2) 4" perforated PVC pipe beneath the slab-on-grade

All drainage pipes will be gravity drainage pipes with a 1% slope.
All drainage pipes will drain to storm drain lines.

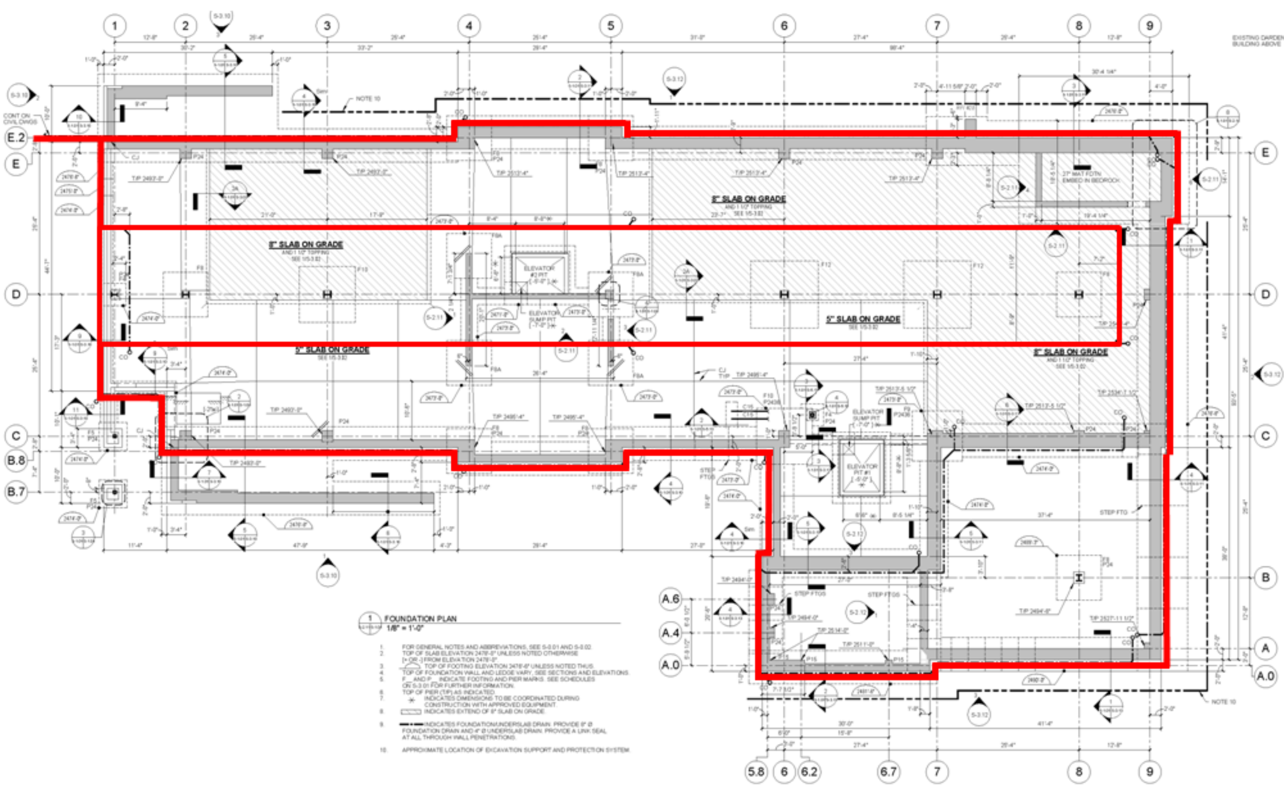


Figure 46: Location of Drainage Pipes

Breadth 2: Cost and Schedule Analysis

Cost Analysis

It was determined in Technical Report 3 that a concrete structural system should be less expensive than a steel structural system based on assembly estimates using RS Means. This breath will compare the cost of the two-way conventionally reinforced concrete system to the existing steel system through a detailed cost analysis to determine if this is accurate.

Steel Estimate

A detailed estimate of the steel system was provided by the engineers at Cannon Design. The items included for comparison in this estimate are broken down and shown in **Table 41**. The total cost of the steel structural system was about $1.5 million with the structural floor framing being the primary cost. The cost of the system was approximately 3% of the total project cost which was approximately $43 Million.

Item	Amount
Fiber Reinforcement	28,317
Normal Weight Fill	144125
Finish Elevated Slab	67,830
Cure and Protect Slab	10,755
Wide Flange Steel Column	208,893
Structural Floor Framing	742,673
Metal Floor Deck	178,797
Spray Fire Proofing	102,629
Total Cost	**$ 1,484,019**

Table 41: Steel System Estimate

Concrete Estimate

A detailed cost estimate of the concrete structural system was completed using RS Means. The system was broken down into five main categories which were formwork, structural concrete, finishing, placement, and reinforcement. The total cost of each category including waste, and the total cost including adjustment factors for time and location can be seen in **Table 42**. A breakdown of each detailed estimate along with the applicable waste and adjustment factors can be seen in Appendix K.

Item	Amount
Formwork	553622
Structural Concrete	273961
Finishing	42863
Placement	51167
Reinforcement	231115
Total Cost	**$ 1,268,200**

Table 42: Concrete System Estimate

The total amount of concrete and reinforcement was estimated based on the level 5 floor design completed using RAM Concept. The amount of concrete and reinforcing for the other floors in the structure was extrapolated based on the typical design and the percent increase required to complete the other floors.

Some special considerations taken into account for the concrete system estimate were the placement method, the elevated slab concrete mix, and the column forms. The placement of the concrete was assumed to be by pump rather than a crane and bucket. This assumption was made based on the recommendations that a 7-story building was of reasonable height for construction to be completed using a concrete pump.

To decrease the concrete system schedule, an accelerated concrete mix was specified to be used for the elevated slabs. In order to account for this in the cost analysis, the cost of the structural concrete for the elevated slabs was estimated to be $8 more than then cost per cubic yard given in RS Means.

Since all of the columns except two are 24” x 24” it was decided that the column forms would be rented instead of built. This decision was estimated to increase the system cost by approximately $2 per month, but would significantly decrease the project schedule.

Cost Comparison

After completing the cost analysis and comparison of both systems it was determined that the concrete structural system would be less expensive than the steel structural system. The cost of the steel system was about $24.50/SF as compared to the redesigned concrete system cost of $21.00/SF. The total savings in cost is approximately 15%.

Schedule Analysis

Along with a cost analysis of the existing composite steel system vs. the redesigned concrete system, a schedule analysis was also completed in order to provide a complete comparison of both systems. It was expected that the concrete system's total duration time would be longer than that of the composite steel system due to the typical increased time required for formwork.

Steel Schedule

The project schedule for the New Library was provided by Cannon Design. The structural steel portion of the project is projected to take approximately 119 days, and is expected to last from March 3, 2014 until August 15, 2014. The schedule is mainly comprised of steel erection, stair erection, and decking. **Figure 47** below shows the structural steel portion of the schedule. The tasks in green are the remaining work to be completed while the tasks in red are the critical remaining work to be completed.

STRUCTURAL STEEL		03-Mar-14 A	15-Aug-14		119
SS-05 1215	Structural Steel Imbeds - Fab & Del	03-Mar-14	10-Mar-14	0%	5
SS-05 1200	Structural Steel - Mobilize	02-Jun-14	03-Jun-14	0%	1
SS-05 1220	Steel Erection - Lvl 2	04-Jun-14	25-Jun-14	0%	15
SS-05 1202	Structural Steel - Fab & Del	11-Jun-14 A	04-Jun-14	2.08%	1
SS-05 3100	Decking - Lvl 2	20-Jun-14	25-Jun-14	0%	3
SS-05 1230	Steel Erection - Lvl 3	25-Jun-14	02-Jul-14	0%	5
SS-05 3160	Stair Erection Lvl1-Lvl2	25-Jun-14	04-Jul-14	0%	7
SS-05 3110	Decking - Lvl 3	27-Jun-14	02-Jul-14	0%	3
SS-05 1240	Steel Erection - Lvl 4	02-Jul-14	09-Jul-14	0%	5
SS-05 3120	Decking - Lvl 4	04-Jul-14	09-Jul-14	0%	3
SS-05 3170	Stair Erection Lvl2-Lvl3	04-Jul-14	15-Jul-14	0%	7
SS-05 1250	Steel Erection Lvl 5	09-Jul-14	16-Jul-14	0%	5
SS-05 3130	Decking - Lvl 5	11-Jul-14	16-Jul-14	0%	3
SS-05 3180	Stair Erection Lvl3-Lvl4	15-Jul-14	22-Jul-14	0%	5
SS-05 1260	Steel Erection - Lvl 6	16-Jul-14	23-Jul-14	0%	5
SS-05 3140	Decking - Lvl 6	18-Jul-14	23-Jul-14	0%	3
SS-05 3190	Stair Erection Lvl4-Lvl5	22-Jul-14	01-Aug-14	0%	8
SS-05 1270	Steel Erection - Lvl 7	23-Jul-14	30-Jul-14	0%	5
SS-05 3150	Decking - Lvl 7	25-Jul-14	30-Jul-14	0%	3
SS-05 3200	Stair Erection Lvl5 - Lvl6	01-Aug-14	08-Aug-14	0%	5
SS-05 3210	Stair Erection Lvl6 - Lvl7	08-Aug-14	15-Aug-14	0%	5

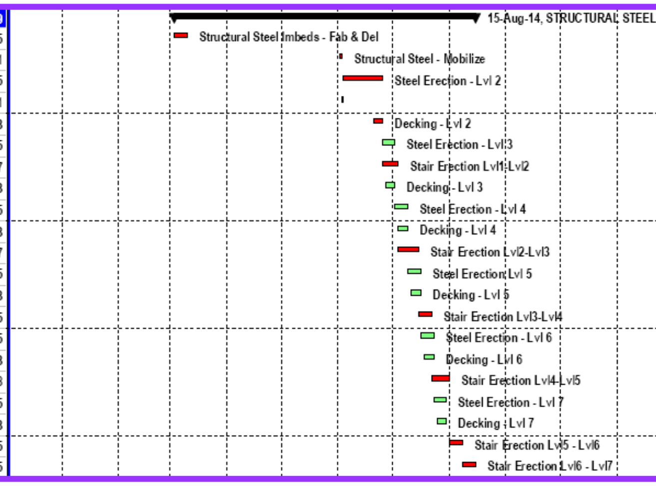


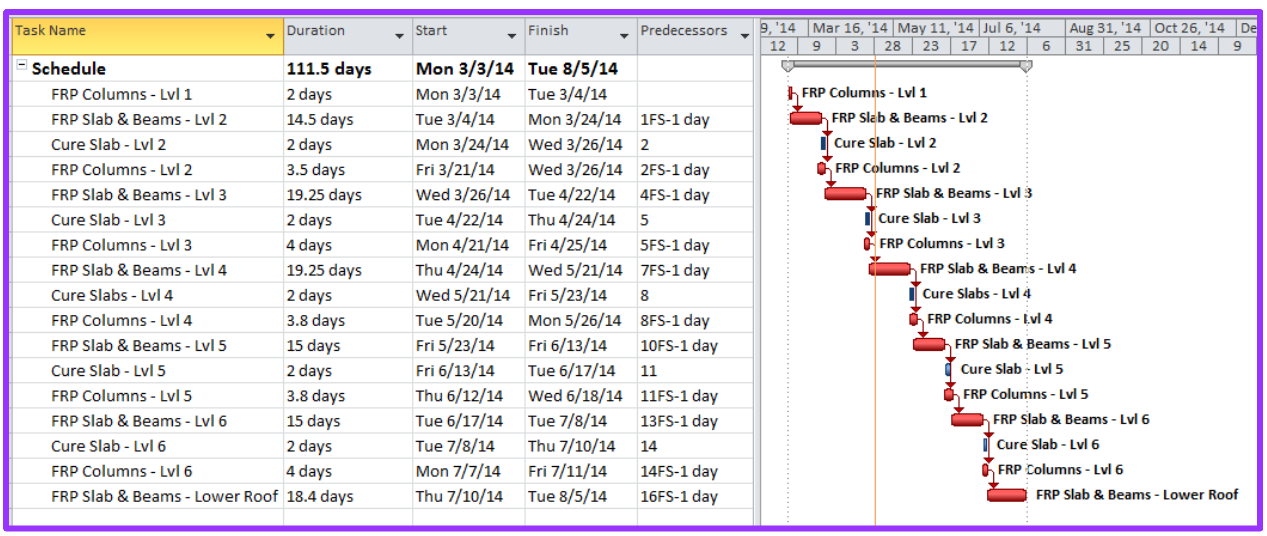
Figure 47: Structural Steel Schedule (Courtesy of Cannon Design)

Concrete Schedule

The construction of the concrete gravity system followed a similar pattern for each level of the building. First, the columns were completed by placing the reinforcement, setting the formwork, and then placing the concrete. Next, the beams and elevated slabs were completed following the same process, and shoring was used to support the elevated slabs until the columns reached adequate strength. Once the slabs were finished the next level of columns were started and this process was repeated for each level of the structure. The total time for the concrete structure to be completed was determined to be approximately 112 days.

Most of the daily output values for each activity were determined using RS Means. From these values the duration of each item was calculated, and can be seen in Appendix L. Most of the tasks were also based on the use of one crew, with the exception of formwork and slab finishing. The time required to complete the formwork for the elevated slabs was determined based on the use of three crews due to the extended amount of time required for formwork. For example, with one crew it would have taken 24 days to form one elevated slab. The time required to finish the elevated slabs was then determined based on the use of two crews. Time required for the formwork for the columns was also significantly less due to the fact that the column forms were rented, rather than built on site.

The schedule for the concrete system was created using Microsoft Project. **Figure 48** below shows the completed schedule. The critical path of the new structure was primarily impacted by the time required to form and finish the slabs, even with the increased number of crews. By using shoring and reshoring, the slabs could be formed before the columns were completed, and the use of an accelerated concrete mixture in the slab allowed a decrease in schedule time.



Task Name	Duration	Start	Finish	Predecessors
Schedule	**111.5 days**	**Mon 3/3/14**	**Tue 8/5/14**	
FRP Columns - Lvl 1	2 days	Mon 3/3/14	Tue 3/4/14	
FRP Slab & Beams - Lvl 2	14.5 days	Tue 3/4/14	Mon 3/24/14	1FS-1 day
Cure Slab - Lvl 2	2 days	Mon 3/24/14	Wed 3/26/14	2
FRP Columns - Lvl 2	3.5 days	Fri 3/21/14	Wed 3/26/14	2FS-1 day
FRP Slab & Beams - Lvl 3	19.25 days	Wed 3/26/14	Tue 4/22/14	4FS-1 day
Cure Slab - Lvl 3	2 days	Tue 4/22/14	Thu 4/24/14	5
FRP Columns - Lvl 3	4 days	Mon 4/21/14	Fri 4/25/14	5FS-1 day
FRP Slab & Beams - Lvl 4	19.25 days	Thu 4/24/14	Wed 5/21/14	7FS-1 day
Cure Slabs - Lvl 4	2 days	Wed 5/21/14	Fri 5/23/14	8
FRP Columns - Lvl 4	3.8 days	Tue 5/20/14	Mon 5/26/14	8FS-1 day
FRP Slab & Beams - Lvl 5	15 days	Fri 5/23/14	Fri 6/13/14	10FS-1 day
Cure Slab - Lvl 5	2 days	Fri 6/13/14	Tue 6/17/14	11
FRP Columns - Lvl 5	3.8 days	Thu 6/12/14	Wed 6/18/14	11FS-1 day
FRP Slab & Beams - Lvl 6	15 days	Tue 6/17/14	Tue 7/8/14	13FS-1 day
Cure Slab - Lvl 6	2 days	Tue 7/8/14	Thu 7/10/14	14
FRP Columns - Lvl 6	4 days	Mon 7/7/14	Fri 7/11/14	14FS-1 day
FRP Slab & Beams - Lower Roof	18.4 days	Thu 7/10/14	Tue 8/5/14	16FS-1 day

Figure 48: Structural Concrete Schedule Created using Microsoft Project

Schedule Comparison

The total construction time for the concrete system was approximately 112 days. This is 7 days shorter than the steel construction length, which is 119 days. Typically concrete systems require a longer construction length than steel system, but with increased crews and efficient scheduling the concrete system can actually require a shorter construction time.

One thing to keep in mind is the fact that concrete construction requires skilled labors to complete the work. The concrete system schedule heavily depends on the efficiency of the workers and the team in charge of the job site. This impact would be to be taken into consideration before making the decision to use a concrete structural system.

Construction of both the steel system and the concrete system would be completed in August. Considering the fact that the New Library is located in the middle of the campus at the University of Virginia's College at Wise, this means that construction of either structural system would be completed before students returned for fall semester.

System Comparison

The existing structural system for the New Library is composite steel, and the redesign was completed using reinforced concrete. It was desired to determine the feasibility of a concrete structural system as compared to the existing steel system.

One consideration in using a concrete system instead of a steel system was the allowable construction type of the New Library. The existing steel structural system was classified as Type IB. Using Table 503 from IBC 2012, it was determined that the New Library's construction type is required to be Type IA or IB based on its A-3 occupancy group and height of six stories. Table 601 from IBC 2012 gives the required fire-resistance ratings for building element in order to achieve a given construction type, and section 722 provides fire resistance ratings based on the thickness of structural elements and the cover to reinforcement. The primary structural members including the slab, beams, and columns all have a fire-resistance rating of 2 hrs. Based on this, the fire-resistance rating of the concrete structure is 2hrs and the construction type is Type 1B. The existing steel system was also classified as Type 1B, so there was no change in construction type.

The average depth of members of the existing steel floor system as compared to members of the redesigned concrete system can be seen in **Table 43** below.

Member	Steel	Concrete
Slab/Floor Depth (in)	6.5	10
Interior Beam Depth (in)	16	-
Interior Girder Depth (in)	24	24
Maximum Edge Beam Depth (in)	30	30

Table 43: Depth of Floor System

Based on the depth comparison, it can be seen that the floor-to-ceiling height with the concrete system will be approximately 6.5"-12" less than the existing floor system. Typically this is beneficial but in the New Library the floor-to-floor heights are dictated by the topography of the hillside rather than the structure, and the existing floor-to-floor heights are 16'-18'. Therefore, the reduction in depth is not as beneficial in the New Library.

As seen in the construction analysis potion of this report, the concrete structure offers a savings in cost of 15% as compared to the existing steel system. The concrete system also does not pose a negative impact on the construction schedule, and could actually decrease the schedule time by a little over one week.

Based on this information it was determined that it is feasible to use a concrete structural system in the New Library, especially from a cost savings stand point.

Conclusion

This report consisted of an analysis and redesign of the New Library at the University of Virginia’s College at Wise. During the fall semester, analyses of the existing composite steel gravity system and concrete shear wall lateral system were completed. It was determined that the original designs were adequate for both strength and serviceability criteria, so a scenario was created in which the feasibility of a concrete structural system was to be considered.

The primary structural redesign was completed using a conventionally reinforced two-way concrete flat slab. This system was chosen based on the typical cost savings of a two-way system as compared to other concrete floor systems. Also, the bay sizes in the New Library are relatively square which is beneficial in a two-way system. It was recognized that there would be deflection issues in the longer bays, so deflection solutions were investigated as part of the redesign.

There was also an interest to investigate the feasibility of a post-tensioned concrete floor slab, which was completed as a secondary redesign. From this it was determined that a post-tensioned slab would not be a good choice due to slab shortening complications with the shear walls and foundation walls. RAM Concept was used to aid in the design of the floor systems, and the program output was verified by hand.

The existing ordinary reinforced concrete shear walls were determined to be the most efficient lateral system for the New Library. Although the system was not redesigned, it was analyzed under increased seismic loads. Based on hand calculations and computer output from ETABS, it was determined that the existing system was adequate for both strength and serviceability.

As part of the first breadth study, an analysis and design of the foundation wall drainage system was completed, along with a study of the water proofing for the foundation walls and basement slab. This was done to ensure that there would be no water infiltration due to the building’s integration into the hillside.

For the second breadth study, a cost and schedule analysis was completed to help determine the feasibility of the concrete system. Through this study it was determined that the concrete system would offer a savings in cost and a decrease in the construction schedule.

It was determined that a two-way, conventionally reinforced concrete system would be a feasible option for the structural system as long as adequate laborers are available. The steel and concrete systems are similar in size and depth, with the concrete system offering a small increase in floor-to-ceiling height. The concrete system will offer a significant cost savings, and will also result in a slight decrease in project duration.

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Note: Various course notes were also used in the completion of this thesis report.