Spring 2023

McLaughlin-Walsh Residence Hall Expansion

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Department of Civil, Environmental, and Sustainable Engineering

I HEREBY RECOMMEND THAT THE THESIS PREPARED
UNDER MY SUPERVISION BY

Rebecca Huang, Shanelle Smith

ENTITLED

McLaughlin-Walsh Residence Hall Expansion

BE ACCEPTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR THE DEGREE OF

BACHELOR OF SCIENCE

IN

CIVIL ENGINEERING

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RESIDENCE HALL EXPANSION

by

Rebecca Huang
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SENIOR DESIGN PROJECT REPORT

submitted to
the Department of Civil, Environmental, and Sustainable Engineering

of

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Rebecca and Shanelle

RESIDENCE HALL EXPANSION

Rebecca Huang and Shanelle Smith

Department of Civil, Environmental, and Sustainable Engineering
Santa Clara University, Spring 2023

ABSTRACT

An expansion of McLaughlin-Walsh Residence at Santa Clara University was designed to provide additional housing accommodations to on-campus residents. The design included both the structural and geotechnical components. The proposed solution included a vertical expansion of the residence hall with the design of a podium to span over the existing structure to support an additional residential structure. The loads of the additional residential structure were estimated to complete the design of the podium structure. The podium was designed to support the estimated loads using reinforced concrete columns, steel beams, prefabricated trusses, and concrete on metal deck. The podium design was selected over other considered alternatives due to its ability to provide housing vertically, rather than expanding horizontally. Using the loads from the additional residential structure and the podium structure, the foundations were designed. Due to the large loads, reinforced concrete drilled shaft foundations were selected and designed. The podium and foundations were designed to accommodate three (3) residential floors above the existing residence hall, which would provide housing for an additional 200-250 students.

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01. INTRODUCTION

Background

In recent years, Santa Clara University (SCU) has experienced record-breaking enrollment and admissions numbers for incoming classes of first-year students according to press releases from 2022, 2021, and 2019. Despite this recent influx of students, new housing is not being built to accommodate both the incoming and current students. The lack of new housing solutions puts a strain on housing decisions made by students, which in turn affects students' finances and overall campus culture since students are being increasingly pushed to move off campus.

Problem Statement

Despite increases in the undergraduate enrollment and admission rate, new housing is not being built at SCU to properly accommodate students' needs. Therefore, more housing solutions must be implemented and made available to students.

Key Objectives for Project and Stakeholders

The key objective of this senior design project was to combat the current student housing issue by designing more student housing. The stakeholders for this project were the Santa Clara University Board of Trustees and the SCU student population, both incoming and current. The needs from the students were further investigated through surveys conducted for this project. Out of 80 students who were surveyed about current issues at SCU, 70% of students considered additional housing to be the most urgent need on-campus right now. While new housing solutions open up possibilities to admit more students to SCU, the objective of this project was to successfully accommodate students who are already enrolled and attending SCU. This way, the cycle of admitting more students, not having enough housing, building more housing, admitting more students, etc. can be broken and the issue can be resolved.

Project Approach

This project included both structural and geotechnical scopes of work. Tasks for each scope were as follows:

Structural

1. Determined the structural components of the additional housing structure
2. Calculated the loads from the additional housing structure
    a. Lateral loads
    b. Gravity loads
3. Designed the podium structure based on the loads
4. Proposed package with the finalized design and structural analysis
    a. CAD model

Geotechnical

1. Sought to understand existing subsurface conditions
2. Identified existing bearing capacity and settlement concerns
3. Evaluated allowable bearing pressure for new foundation
4. Selected foundation type
5. Performed settlement analysis of foundation
6. Proposed foundation design package

02. ALTERNATIVES ANALYSIS

The following section includes an alternative analysis of different solutions to address the housing problem at SCU. To complete this analysis, constraints and criteria are listed to provide a framework for comparing options. After the establishment of these constraints and criteria, alternative solutions are listed and ranked based on how well they meet the criteria listed.

Constraints

The constraints for this project were items that must be met in order for a solution to be considered for this project. The constraints for each solution were as follows:

1. Available Land Area: The designed solutions cannot exceed the lot area that is available on and around SCU.
2. Building Codes: All building codes, including the California Building Code 2022, California Green Building Codes, Santa Clara City Codes, and Codes of Ethics have to be met.
3. City Permission: All construction projects will require the approval of the City and also from the County and State if applicable.
4. Accommodation of At Least 100 More Students: All solutions must have the capacity to accommodate at least 100 additional students. A significant number of students need accommodations to reduce the housing problem.

Criteria

The criteria being used to compare solutions are listed below and are given a weight from one (1) to five (5), with five (5) being the most important criteria and one (1) being the least important. The criteria are as follows:

Active campus feasibility – 5

Each solution varied depending on its feasibility to be completed on an active campus. This criteria has technical, economical, and social/cultural impacts. Building any housing solution on an active campus takes technical knowledge regarding planning, scheduling, site logistics, etc.. This factor is important to consider given the extent of the team's technical knowledge up to this point. There is also a possibility that a given solution will displace students who are living on campus. In that scenario, the school should be responsible for compensating or accommodating displaced students over the course of construction. If students are displaced or if different sections of campus are blocked off, there could be a shift in campus culture. The impact of construction on the students' daily lives is very important, since the solution is meant to benefit students. Due to these reasons, this criteria has been ranked as a five (5) and is one of the most important considerations for a solution. This criteria is also connected to the project duration since the duration will influence the time of year the project will be completed, and further will determine how active campus would be during construction.

Land Use Efficiency – 4

The land use efficiency factor involves the consideration of whether the land is being used to its maximum capacity. Within the list of alternatives, solutions should use both the horizontal and vertical space to the fullest potential. Using the land efficiently will be beneficial both economically and environmentally. By using land efficiently, lot investments will decrease for the owner and less land will need to be developed, leading to a decreased environmental footprint. The land use efficiency criteria was given a weight of four (4) because it would be preferable if the project is using more vertical space rather than horizontal space; it is not the most important criteria the team considered.

Proximity to Campus – 4

The distance between the respective solution and the closest campus entrance is a factor with social, environmental and economic impacts. Therefore, this criteria was given a weight of four (4). The closer the project is located to campus, the more desirable it is in terms of users' preference. Students, being the users in the problem and project, will prefer a location of living that is close to campus. This conclusion was supported by the survey conducted within the student population for this project. Another consideration towards this factor is that students might potentially need methods of transportation if the housing location is further away from campus. Transportation other than walking will initiate increased costs for the users, as well as posing potential environmental impacts.

Sustainability – 3

The sustainability factor includes the number of environmental impacts the project will impose, as well as the amount of carbon footprint throughout the project duration. The environmental footprint will be analyzed through the type of project, the amount of land used, and the type of structure and materials used. Relative aspects, such as the construction duration and transportation of materials, were also taken into consideration while evaluating the sustainability portion of the alternative solutions. The sustainability criteria were given a weight of three (3) because it is important to limit the amount of emissions and environmental footprint left from the project. The owner, Santa Clara University, will have a better reputation by implementing a sustainable residence hall. The surrounding community will also be positively affected more by an environmentally-friendly project in comparison to a less sustainable project.

Project Duration – 3

The duration of the project will have economic, technical, and social/cultural impacts. The longer a project takes to build, the more construction costs there will be, such as labor and materials. There will be an economic impact on SCU because the duration directly impacts the amount of money earned from student housing. Duration also impacts students because they will have to make housing decisions based on how long the new housing takes. Students may have to spend more money on non-affiliated housing if the new solution is not completed in time. The scope of work that will be completed is limited by the team's technical knowledge, and said scope will change the project duration. The more scopes are included, the longer a solution will take. The construction time also impacts the on-campus and surrounding community, which influences the campus culture. This criteria was ranked a three (3) because the implemented solution will be in place for much longer than it will take to construct, and the long-term benefits of more housing outweigh the short-term impacts of construction time.

Cost Benefit – 2

The cost-benefit for a given solution is an economic criteria. The investment that needs to be made for each solution will differ, and each one will have a different outcome regarding the amount of housing that will be provided. More housing accommodations will return a larger benefit economically for the school and generally for the student body, however, more costs must be made for more housing. There will also be a higher cost-benefit for solutions that efficiently use the given space for housing. This criteria was ranked a two (2) because the team did not know the exact cost-benefit ratio of each solution at this stage of the process.

Alternative Solutions

The following section includes alternative solutions to be considered to resolve the housing problem at SCU. The proposed solutions include sticking to the status quo of the current housing accommodations, building a student apartment complex, building student tiny homes, expanding the University Villas, and retrofitting an existing residence hall. Each of these solutions has a description along with the scores of how well the given solution met the previously stated criteria.

Some solutions require land to be built, including the student apartment complex, tiny homes, and University Villas expansion. Each of these solutions would be placed on a lot within a half mile radius of SCU's campus. The lots considered were a vacant lot at 2615 The Alameda, Santa Jose, CA 95126 and the lot that currently hosts The Garage, SCU's machine shop, located at 3305 The Alameda, Santa Clara, CA 95050. Even though The Garage is a building that is currently used by students and faculty, much of the mechanical engineering shop space has been moved into the Sobrato Campus for Discovery and Innovation (SCDI). By demolishing The Garage, the University has the opportunity to use that land to accommodate more students and has the opportunity to develop an upgraded version of the shop space in addition to the SCDI upgrades.

Status Quo

The status quo alternative is to not provide additional housing for the students at Santa Clara University. For any problem there is always an option to do nothing, however, this option does not satisfy the constraint of providing a housing solution. With this alternative, the University will still lack the amount of housing needed to address the problem statement. Since this option does not meet all the constraints, it will not be considered as a viable alternative solution for the project, nor will it be ranked for the different criteria categories selected.

Student Apartment Complex

The new student apartment complex will be a ground up residence hall located on a lot within a half mile radius of campus. This residence option will be able to accommodate approximately 100-200 students in various different room layouts.

1. Active Campus Feasibility – 10. Since this is an off-campus solution, it will not interfere with the active campus at all.

2. Land Use Efficiency – 7. The student density provided by an apartment building is greater than the other solutions, however, SCU would have to build the apartment on land that is not currently serving the purpose of housing students, reducing the efficiency of land use.
3. Proximity to Campus – 8. The apartment building would be on a lot less than 0.5 miles away from campus. Although it is not as convenient as being on-campus, it is still very accessible.
4. Sustainability – 3. Building an apartment from the ground up has more limitations on how sustainable it can be, so it received the lowest sustainability score.
5. Project Duration – 1. The project will take the longest to complete and therefore received the lowest duration score.
6. Cost-Benefit – 6. While there will be a large investment into the project, there will be a large return since many students will benefit.

<u>Tiny Homes</u>

The tiny homes solution would include a community of tiny homes on a lot within a half mile radius of campus. Each house would accommodate two to three (2-3) students and there could be bathrooms either in the tiny home itself, or in a communal facility on the property. If this solution is implemented, there would be opportunities to have prefabricated or modular housing. This solution would accommodate approximately 50-100 extra students.

1. Active Campus Feasibility – 10. Since this is an off-campus solution, it will not interfere with the active campus at all.
2. Land Use Efficiency – 1. Having tiny houses that individually house two to three (2-3) students would require a very large lot to provide housing to a significant number of students. Since the tiny houses will be built on the lot with The Garage, there would not be very many units and construction could not occur vertically to maximize land. This solution received the lowest score.
3. Proximity to Campus – 8. The tiny houses would be on a lot less than 0.5 miles away from campus. Although it is not as convenient as being on-campus, it is still very accessible.
4. Sustainability – 9. Tiny homes have the most opportunities for sustainable construction by using prefabricated or modular materials. This solution received the highest score.
5. Project Duration – 8. Putting tiny homes on a lot will take time, but if modular structures are used, time will be reduced. This received the second highest score for this category.

6. Cost-Benefit – 3. This solution does not make efficient use of horizontal and vertical space. Therefore, this received the lowest score.

University Villas Expansion

The expansion of the University Villas, SCU's affiliated off-campus housing units, would be implemented on a lot within a half mile radius of campus. The new housing structures will be similar to that of the existing villas. This housing option will provide approximately 68 townhouse room units and there will be the options of single, double and quadruple room layouts. This will allow anywhere from 150-250 additional students to be accommodated.

1. Active Campus Feasibility – 10. Building more villas will happen away from the main SCU campus, but will still cause some disturbance since people would live at the villas during construction.
2. Land Use Efficiency – 4. Using the villas layout on The Garage's lot will result in more housing than tiny houses, but less housing than an apartment complex. This option does not use the vertical space as efficiently as the retrofit solution. Therefore, it received a score between those solutions.
3. Proximity to Campus – 8. The villas would be expanded on a lot less than 0.5 miles away from campus. Although it is not as convenient as being on-campus, it is still very accessible.
4. Sustainability – 3. Building villas from the ground up has more limitations on how sustainable it can be, so it received the lowest sustainability score.
5. Project Duration – 3. These buildings would take more time to construct since they would not be modular, but they would not take as long as a larger apartment complex. This solution received the second lowest score.
6. Cost-Benefit – 5. The investment would be significant since these villas would expand from the ground up, however, there will be less housing available than if an apartment was built, so this received a score lower than apartments.

Residence Hall Retrofit

The retrofit of an existing residence hall will include a vertical expansion of a given structure to accommodate the increasing number of students. The solution does not require demolition or additional lot purchase. The proposed residence hall to be expanded upon is

McLaughlin-Walsh since it is rectangular and only has three residential levels currently. This housing option will accommodate an additional 200-250 students.

1. Active Campus Feasibility – 3. This on-campus solution could potentially displace students in the particular residence hall that would be retrofitted. Regular on-campus traffic would be impacted as well due to construction.
2. Land Use Efficiency – 10. Efficient use of existing land and structure, since the retrofit involves building vertically rather than horizontally.
3. Proximity to Campus – 10. On-campus housing is the most accessible, so this received the highest score.
4. Sustainability – 7. Since the new housing would be added onto existing housing, the environmental impacts and emissions caused by construction would decrease. Structure consistency will limit the sustainability rating for this option.
5. Project Duration – 7. This project will be relatively quick since it does not involve starting a building and foundation from scratch.
6. Cost-Benefit – 6. The investment into the retrofit will be lower than the investment for apartments, but will also result in a smaller amount of housing units. The ratio of cost to benefit will be similar in both solutions, so they received the same score.

Final Solution

After completing the alternative analysis matrix (Appendix A, Table A-1), the top alternative is a residence hall retrofit. This solution received 249 points, which is 118 more than the second place alternative. It is evident that this solution will be the most beneficial for the clients and the users. It is the most suitable solution for the objectives of this project.

03. CODES AND REGULATIONS

To complete the McLaughlin-Walsh Hall retrofit, various codes and regulations had to be considered to ensure safety throughout the course of the project. The codes and regulations that were taken into consideration for this project are the California Building Code (CBC), Santa Clara City Code, Green Building Code, American Institute of Steel Construction (AISC), American Concrete Institute (ACI-318), American Society of Civil Engineers (ASCE) Code of Ethics, Occupational Safety and Health Administration (OSHA), Americans with Disabilities Act (ADA) and the plans for the existing structure (both architectural and structural). Applicable sections from the CBC and Santa Clara City Code are outlined in this report.

California Building Code

The purpose of this section is to establish the minimum requirements to safeguard the public health, safety and general welfare through structural strength means of egress facilities, stability, access for persons with disabilities, sanitation, adequate lighting and ventilation and energy conservation; safety to life and property from fire and other hazards attributed to the built environment; and providing safety to firefighters and emergency responders during emergency operations. The following list is numbered based on the corresponding chapter within the California Building Code.

Section 1: Scope and Administration

- 1.1.3.2 State regulated buildings, structures and applications - State-owned buildings, including buildings constructed by the Trustees of the California State University, and to the extent permitted by California laws, buildings designed and constructed by the Regents of the University of California, and regulated by the Building Standards Commission.
- Section 1.5 - reserved for the California Energy Commission.
- Section 1.8.2.1.1 Hotels, motels, lodging houses, apartments, dwellings, dormitories, condominiums, shelters for homeless persons, congregate residences, employee housing, factory-built housing and other types of dwellings containing sleeping accommodations with or without common toilets or cooking facilities.
- Section 1.8.2.1.3 - Permanent buildings and permanent accessory buildings or structures constructed within mobile-home parks and special occupancy parks regulated by the Department of Housing and Community Development.
- Section 1.9.1 - Accommodations for persons with disabilities regulated by the Division of the State Architect.

Section 3: Occupancy Classification and Use

- 302.1. Occupancy Classification
    - 11. Residential (section 310)
    - 302.2. Use Designation
- 310. Residential group R
    - Residential groups include the use of a building or structure, or a portion for sleeping purposes when not classified as an institutional group or when not regulated by the CRC (California Residential Code).

- 310.3 Residential Group R-2
    - Dormitories are considered Residential Group R-2. More than two dwelling units where the occupants are primarily permanent in nature

Section 4: Special Detailed Requirements Based on Occupancy Use

- 420. Groups R-1, R-2, R-2.1, R-2.2, R-3, R-3.1 and R-4
    - Occupancies in Groups R-1, R-2, R-2.1, R-2.2, R-3, R-3.1 and R-4 shall comply with the provisions of Sections 420.1 through 420.11 and other applicable provisions of this code.
    - 420.2 Separation Walls
    - 420.3 Horizontal Separation
    - 420.4 Automatic Sprinkler System
    - 420.5 Fire Alarm Systems and Smoke Alarms
    - 420.6 Smoke Barriers in Group R-2.1
    - 420.6.1 Smoke Barriers in Group R-2.2
    - 420.6.2 Refuge Area
    - 420.9 Domestic Cooking Appliances
    - 420.10 Group R Cooking Facilities
    - 420.11 Group R-2 Dormitory Cooking Facilities
    - 420.11.1 Cooking Appliances
    - 420.11.2 Cooking Appliances in Sleeping Rooms
    - 420.12 [HCD 1] Construction Waste Management

Section 5: General Building Heights and Areas

- 502.1 Address Identification
    - New and existing buildings shall be provided with approved address identification. The address identification shall be legible and placed in a position that is visible from the street or road fronting the property.
- 503.1 General Building Height and Area Limitations
- 504.1 General
    - The height, in feet, and the number of stories of a building shall be determined based on the type of construction, occupancy classification and whether there is an automatic sprinkler system installed throughout the building.
- 504.3 Height in Feet

- The maximum height, in feet, of a building shall not exceed the limits specified in Table 504.3.
- 85 ft for sprinkler with no increase in area for type I B construction

- 505.2.2 Means of Egress
- 505.3.1 Area Limitation
- 506.1 General
    - The floor area of a building shall be determined based on the type of construction, occupancy classification, whether there is an automatic sprinkler system installed throughout the building and the amount of building frontage on public way or open space.
- 506.1.1 Unlimited Area Buildings
- 506.1.3 Basements
- 506.2 Allowable Area Determination
    - The allowable area of a building shall be determined in accordance with the applicable provisions of Sections 506.2.1, 506.2.2 and 506.3.
    - Unlimited for type I B construction
- 510.2 Horizontal Building Separation Allowance

Section 10: Means of Egress

- 1001.2 Minimum Requirements
    - It shall be unlawful to alter a building or structure in a manner that will reduce the number of exits or the minimum width or required capacity of the means of egress to less than required by this code.
- 1002.2 Fire Safety and Evacuation Plans
    - Fire safety and evacuation plans shall be provided for all occupancies and buildings where required by the California Fire Code. Such fire safety and evacuation plans shall comply with the applicable provisions of Sections 401.2 and 404 of the California Fire Code.
- 1003.2 Ceiling Height
    - The means of egress shall have a ceiling height of not less than 7 feet 6 inches (2286 mm) above the finished floor.
- 1003.3.1 Headroom
- 1004.1 Design Occupant Load

- In determining means of egress requirements, the number of occupants for whom means of egress facilities are provided shall be determined in accordance with this section.

- 1004.2 Cumulative Occupant Loads
    - Where the path of egress travel includes intervening rooms, areas or spaces, cumulative occupant loads shall be determined in accordance with this section.

- 1011 Stairways
- 1016 Exit Access
- 1017 Exit Access Travel Distance
- 1019 Exit Access Stairways and Ramps

Section 16: Structural Design + Chapter 16A Structural Design

- 1604 General Design Requirements
- 1605 Load Combinations
- 1606 Dead Loads
- 1607 Live Loads
- 1608 Snow Loads
- 1609 Wind Loads
- 1610 Soil Loads and Hydrostatic Pressure
- 1611 Rain Loads
- 1612 Flood Loads
- 1613 Earthquake Loads

Section 18:Soils and Foundations

- 1801 General
- 1802 Design Basis
- 1804 Excavation, Grading, and Fill
- 1805 Dampproofing and Waterproofing
- 1806 Presumptive Load-Bearing Values of Soils
- 1807 Foundation Walls, Retaining Walls, and Embedded Posts and Poles
- 1808 Foundations
- 1809 Shallow Foundations

- 18010 Deep Foundations
- 1812 Earth Retaining Shoring
- 1813 Vibro Stone Columns for Ground Improvement

Section 19: Concrete

- 1904 Durability Requirements
- 1905 Modifications to ACI 318
- 1906 Footings for Light-Frame Construction
- 1907 Minimum Slab Provisions

Section 22: Steel

- 2203 Protection of Steel for Structural Purposes
- 2204 Connections
- 2205 Structural Steel
- 2206 Composite Structural Steel and Concrete Structures
- 2208 Steel Cable Structures

Section 33: Safeguards During Construction

- 3301 General
- 3304.1 Excavation and Fill
- 3306 Protection of Pedestrians
- 3307 Protection of Adjoining Property
- 3308 Temporary Use of Streets, Alleys, and Public Property

California Green Building Code

- LEED Certified: 40-49 pt
- Sustainable construction

Santa Clara City Code

The applicable sections from the Santa Clara City Code are as follows:

12. Streets, Sidewalks, and Public Places
    - 12.25 Excavation and Use of City Rights of Way
15. Buildings and Construction
    - 15.15 Building Code

- 15.17 Residential Code
- 15.36 Energy Code
- 15.38 Green Building Code
- 15.55 Seismic Hazard Identification

American Institute of Steel Construction (AISC)

Design of steel frames utilizing the AISC steel member information and limitation using the Steel Construction Manual 15th Edition by American Institute of Steel Construction.

American Concrete Institute (ACI)

Concrete design will be based on the information and regulations provided by the American Concrete Institute.

ASCE Code of Ethics

This code will be used to ensure commitment to ethical responsibility as it relates to society, natural and built environment, profession, clients and employers, and peers.

04. PRELIMINARY DESIGN

For the preliminary design of this project, the team determined the basic layout of the podium for the vertical extension and the potential column locations. A steel frame structure design that spans over the existing structure was determined to be the most feasible for this project. The steel frame will then fully support the extension structure. The team decided after receiving and analyzing the existing structural plans. When studying the plans for the Mclaughlin-Walsh residence hall, the team discovered that the existing dormitory building could not support any extra load. Based on the general notes on the structural plans, the concrete structure only has a 28-day strength of 2000 pounds per square inch (psi). This discovery caused significant changes in the design consideration and the preliminary design. The initial design idea of construction directly on top of the existing structure has been considered unsafe and impractical, as the existing structure will not be able to handle the additional load. The other option was to design a steel frame around the existing structure and place the new vertical extension on the steel frame. The frame will transfer the load of the extension structure back to the ground. This design option will include the design of the new steel frame, the extension structure, and the new foundation.

In Appendix B, two plans were created for the graphic portion of the preliminary design. The plans indicated the initial placement of the columns and the design concept of having truss frames sitting on top of the columns as support. The load from the extension structure can cause buckling in the steel column if no reinforcement is applied.

Furthermore, the columns will be placed on the north and south sides of the building. Currently, the north side of the building sits along Santa Clara Street, a street used mainly for student loading and unloading purposes. Since there will not be an extensive amount of vehicles occupying Santa Clara street, it is the most adequate and feasible placement. The north and south side are also the longitudinal sides of the building. The extra length will allow more columns to be placed, making the frame more stable and durable. As shown in the preliminary design plans in Appendix B (S1), 16 columns will be on either side of the residence structure (North and South). The columns will extend approximately two (2) feet above the top of the existing structure. The final height was determined. The truss members will span over the existing building, from one side of the column to the other.

This design will also be considering the existing gable roof. The finalized decision was to demolish the gable roof and replace it with truss frames. As stated in S2, the elevation view, the structure is standing at a current height of 28’-6” above ground elevation. The existing roof will be removed from the current structure, which would decrease the height of the existing structure. The column will extend above the existing structure, at a length of . The columns were estimated (without the actual load estimation) to have a four foot (4') radius and placed around the windows and doorways on the existing structure.

Some design aspects that will continue after the preliminary design are the consideration of egress and elevator situation. According to the CBC 2022, Section 10, means of egress, 1001.2, the minimum requirement will need to be reevaluated and met before the implementation of the project. There is a minimum distance between all locations and the nearest safety exit. This distance will increase as the height of the structure increases. Additional stairways that lead to the emergency exits might be required to satisfy this code.

Before designing the skeleton frame, the loads for the new extension structure were calculated. Estimated load for the vertical extension structure includes:

- Unit weight of wood
- Unit weight of carpet and flooring
- Unit weight of doors and windows
- Unit weight of the roofing
- Unit weight of interior walls
- Unit weight of exterior walls
- Estimated weight of MEPs
- Estimated weight of partitions

The design used the same architectural design and floor plan layout as the existing structure, which can keep the entire residence hall cohesive and uniform. The current layout of McLaughlin-Walsh hall was used to estimate loads of the extension structure. The load calculation allowed the design of the skeleton frame to begin. After calculating and estimating the loads on the skeleton frame, the fountain design began. The soil reports were studied per the structural load calculation. Once loads were complete, it contributed to calculating the bearing capacity needed from the foundation to support the additional structure. The new foundation was then selected and designed using an alternative analysis of different foundation types. The final project includes designs of the residence hall extension structure, the steel frame support, and the foundation supporting both structures.

05. STRUCTURAL DESIGN

Structural Design Introduction

The structural design scope was intended to design the podium frame for the additional extension structure. The podium is composed of columns, trusses and beams. The design of the extension structure was outside of the structural design scope, but the construction materials were chosen and estimated loads were calculated based on the construction materials and other various loading conditions. An alternative analysis was performed for the construction materials for the extension structure, which are steel, concrete and timber. Timber was chosen due to its lighter self-weight, higher seismic resistance, sustainable nature, durability and cost effectiveness. As for the podium structure, three (3) different column sections were considered and analyzed. Reinforced concrete was chosen due to the constructability, cost effectiveness and the high compressive strength. The structural approach section includes the structural notes, structural calculations and design process, column section analysis and comparison, structural layout, and recommended future work that will be outside of the project scope.

Structural Notes

This design was designed based on the 2019 California Building Code (CBC), and the occupancy was determined to be a risk category II, R-2 residence structure. The structural analysis was performed in accordance with the ASCE/SEI 7-16. The reinforced concrete portion of the structure was designed in accordance with the ACI 318-19 from American Concrete Institute.

Structural Calculations

Gravity Loads

The gravity loads include both the dead and live load applied on the designed structure. Shown in Table 1 below, the self-weight of the two materials considered in the design, along with miscellaneous dead load and occupancy determined live loads, are presented.

Table 1: Dead and Live loads Estimated for the Extension Structure.

		Loads (psf)	
		Timber	Concrete
Dead Load	Self-weight	20	145 pcf
	Miscellaneous	15	
Live Load	Rooms	50	
	Corridor	100	
	Stairway	100 & 300 kip	
	Elevator	300 kip	

The deflection limits for the structure were referenced from the CBC 2019 and are shown in the table below. The podium slab was designed according to the deflection limits.

Table 2: Deflection Limits.

	Deflection Limits (in)		
	Dead Loads & Live Loads	Live Loads	Wind Loads
Floor	L/240	L/240	-

<u>Lateral Loads</u>

The lateral loads for the structural portion include the wind loads and seismic loads. Considering the City of Santa Clara's location and geographic condition, the seismic loads governed the lateral load design.

Preliminary seismic analysis on the podium was performed in order to determine the seismic shear and moment acting. The seismic drift limit was referenced from the ASCE/SEI 7-22 code. This limit was used in designing column reinforcement.

Seismic D - Stiff Soil

Site Soil Class:

Results:

PGA_M :	0.53	T_L :	12
S_{MS} :	1.62	S_S :	1.5
S_{M1} :	1.7	S_1 :	0.62
S_{DS} :	1.08	V_{S30} :	260
S_{D1} :	1.13		

Seismic Design Category: D

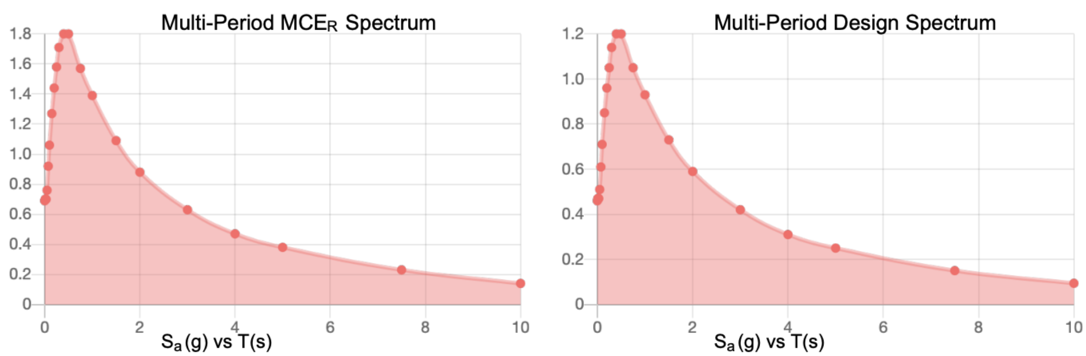

Figure 1: ASCE 7 Seismic Design Data and ARS Curve for Multi-Period.

Structural Design

Column Placement

The placement of the columns were determined with the consideration of the existing building layout and the loading from the residential building addition. During the preliminary design phase, the team decided to place one column (1) every three (3) windows. This was decided with the constraints that the columns shall not block natural lighting from the existing dorm, however, after much evaluation, the team noticed that this placement choice will increase the tributary area of the decking and slab, which would cause significant increase in the foundation loading. It is acknowledged that this placement choice would be inefficient and possibly unfeasible. To accommodate this issue, the team decided to have the placement at one (1) column every two (2) windows. This iteration successfully met the team's design expectation and is feasible. In Table 3, the majority spacing, variation spacing, as well as the maximum spacing is presented.

Table 3: Column Spacing.

Column #	1-6	7	8	9	10	11	12-16
Spacing Distance (ft - inch)	23'-8"	31'	28'-6"	20'-0"	21'-0"	28'-0"	23'-8"

Structural Elements and Member Sizes

The podium structure is designed with 4000 psi concrete and grade 60 steel. The primary structural elements designed in this project were the podium column, truss members and beams, slab and decking. The individual member sizes for the structure can be found in Table 4. The demand calculation for each of the components mentioned previously can be accessed in Appendix C-2, Appendix C-3, and Appendix C-4, respectively.

Podium Column Selection

Two (2) different podium column selections were analyzed prior to the design of the member. An alternative analysis was performed on the concrete filled steel section and traditional reinforced concrete column section. Though the concrete filled steel column section was able to provide excellent confinement for the concrete and additional stiffness to the steel tubular section for buckling prevention, the traditional concrete column section was chosen as the final column type due to the strength, potential size, aesthetic and cost efficiency.

The column height of 35' accounts for the roof demolition and the connection process between the existing residence hall and the extension residence structure. While designing the podium column, the team considered two (2) types of loading; gravity loads and lateral loads. The gravity load applied on the column was estimated with the loads displayed in Table 2, with the addition of the self weight of the truss members, beams, slab and decking. Preliminary seismic analysis was performed with obtained location information, soil data, and structural risk category. The detailed seismic data and ARS curves can be referenced in Figure 1. The column design summary is shown in Table 4, below.

Table 4: Podium Column Dimension Summary.

Column Height	Column Diameter	Longitudinal Reinforcement	Transverse Reinforcement
35’	28”	No.10	No.5 spiral

The reinforcement of the column was designed to accommodate the lateral force estimated from the preliminary seismic analysis. Nine (9) No.10 size rebars will be utilized for longitudinal reinforcement, and No. 5 spiral stirrups will be utilized for transverse reinforcement. A detailed reinforcement section view can be seen in Figure 2, below.

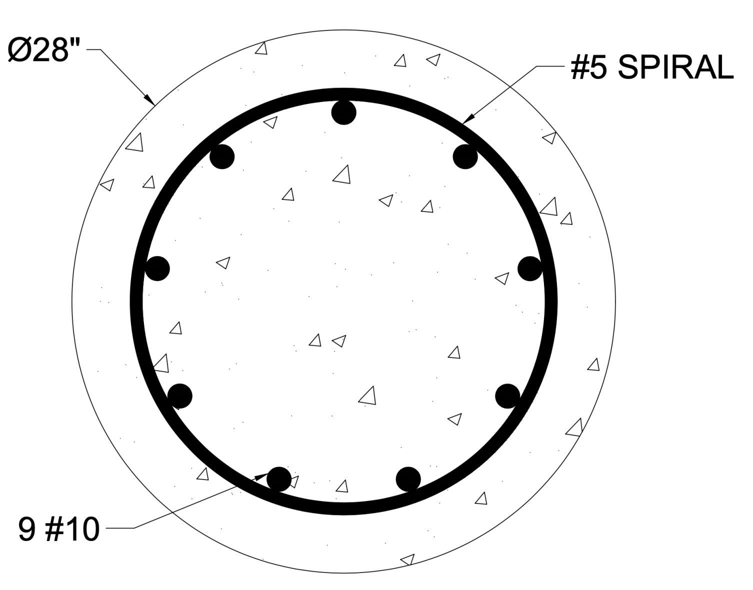

Figure 2: Spiral Reinforced Concrete Column Section.

Truss Member Selection

Initial design for the truss member spanning over the columns was conducted based on the span to depth ratio. Referencing the Santa Clara University Leavey Athletic Center structure plans, the depth to span ratio for the Leavey Center truss was obtained. A minimum depth of 36 in was required based on the clear span for the podium truss and the typical truss depth to span ratio. Prefabricated truss joists were chosen as a more cost effective and efficient method. The detailed truss specifications are shown in Table 5 below. The 48LH16 truss joist manufactured by Vulcraft has the desired ability to accommodate the long clear span in the podium frame. The yield strength of the truss chosen was based on 50 kips maximum yield strength.

Table 5: Truss Member Summary.

Truss Type	Max Clear Span	Yield Strength (pounds per linear foot)
48LH16	65’	1409

Beam Selection

The beam element was designed to connect into the podium truss. The beams were designed as load bearing and will be able to withstand the gravity loads exerted by the structural system above, including extension structure, slabs, and decking. The final beam size designed based on the gravity loads estimated are presented in Table 6, shown below.

The W section beam was chosen because of its typical use in structural systems and its great efficiency at bearing loads. The layout of the beams can be seen in Figure 4, Section Cut A-A. Seven (7) beams will be spaced at 9.25 ft typical throughout the 65 feet span.

Table 6: Beam Section Summary.

Beam Size	Height	Flange Width	Flange Depth	Web Thickness	Weight
W18X97	18.59 in	11.145 in	0.87 in	0.535 in	92 plf*

*Pounds per linear foot

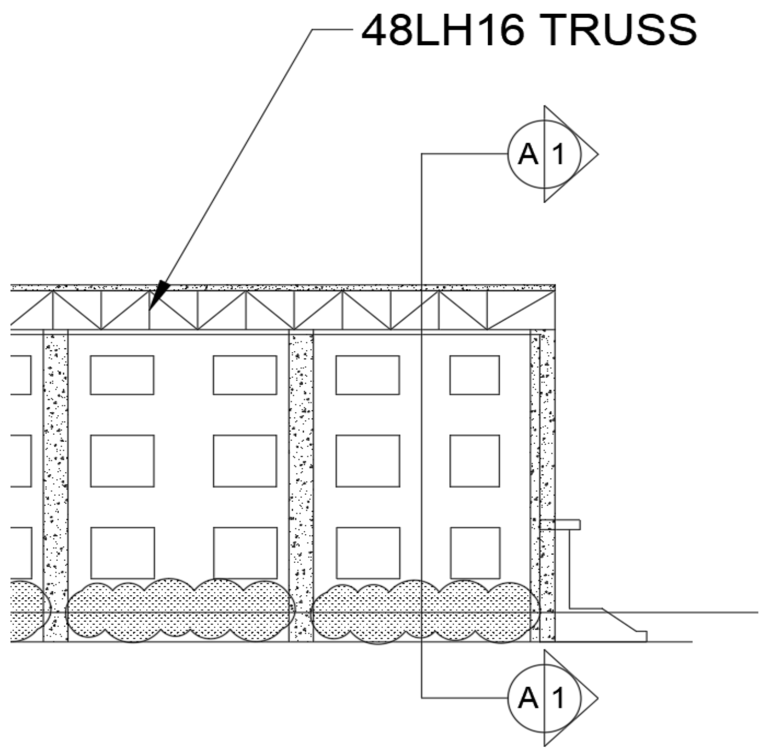

Figure 3: Section of the Elevation View.

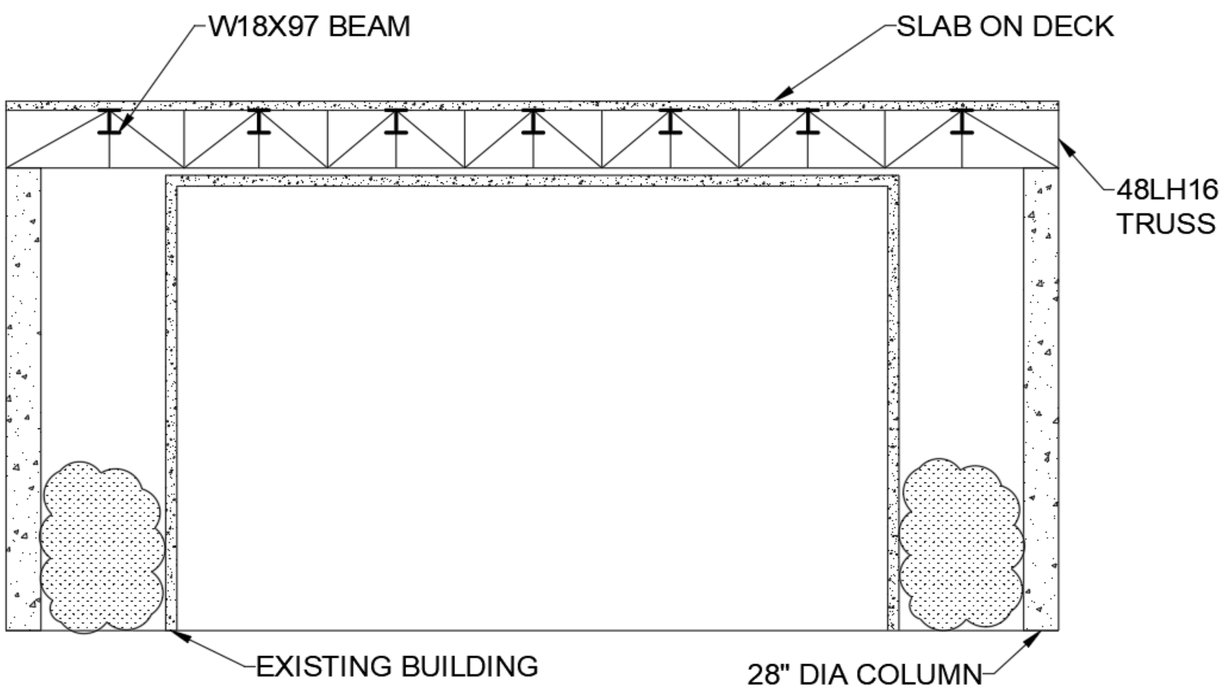

Figure 4: Section Cut A-A.

Slab and Decking Selection

The slab and decking were chosen based on the loads obtained in the extension structure load estimation and the deflection limit for flooring. Composite decking was selected for its good load bearing capacity and the compatibility with the remaining structural components.

Summary

The following figure presents the combined structural elements with respect to the existing residential structure.

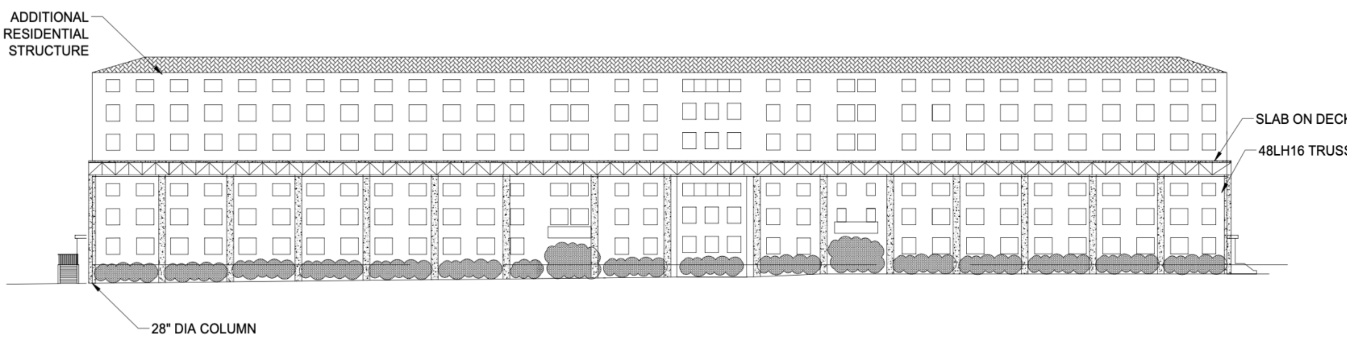

Figure 5: Elevation View of The Podium Frame

06. GEOTECHNICAL DESIGN

Geotechnical Design Introduction

The geotechnical scope involved compiling soil data to be used for all geotechnical calculations. These calculations include calculating bearing capacity and settlement to meet necessary load requirements. The new foundations must be able to support the loads applied from the podium structure and extension structure. Preliminary foundation design focused on sizing foundation elements for bearing capacity and settlement. Final design included sizing, reinforcement, material properties, spacing and layout, and CAD drawings for section cuts.

Soil Data

Soil data could not be located for the areas right beneath McLaughlin-Walsh, however, soil data was available from Swig Residence Hall, the neighboring residence hall to McLaughlin-Walsh. This data was collected in 1964 and included two boring samples obtained using a California sampler from elevation level to 110 ft depth. The boring logs listed soil type, which is mainly medium to very stiff organic silty clay, and the groundwater table being at 47 ft below ground surface. The groundwater level was updated to account for changes in the groundwater levels that have occurred since the initial data collection. An updated groundwater estimate of 20 ft below ground surface was taken from Santa Clara Valley Water’s groundwater monitoring data. Multiple samples were taken within each boring to provide values for dry density, moisture content, and undrained compressive strength. The strength data provided by these samples varied between the two (2) boring logs such that averaging of this data was necessary. All data was averaged and combined into a comprehensive boring log to be used for all necessary calculations, as seen in Figure 6 below.

DEPTH (FT)	SOIL TYPE	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	UNDRAINED COMPRESSIVE STRENGTH (PSF)
0-40	STIFF TO VERY STIFF, MOIST ORGANIC SILTY CLAY WITH FINE GRAVEL	94	27	3080
40-60	MEDIUM DENSE TO STIFF SILTY SANDY CLAY	101	26	1740
60-110	MEDIUM DENSE TO VERY STIFF SANDY SILTY CLAY WITH OCCASIONAL GRAVEL	105	22	2380

WATER TABLE 20FT

Figure 6: McLaughlin-Walsh compiled boring logs with soil data.

Based on the data in Figure 6, the soil being used was rather weak. Unconfined compressive strength is the maximum compressive stress that soil can bear under zero (0) confining stress. This measurement is typically used for saturated, cohesive soils recovered from thin walled sampling tubes. For calculations, the undrained shear strength was used, and the undrained shear strength is half of the undrained compressive strength shown in the figure. Since the saturation of the soil was already considered with the provided strength data, groundwater effects were accounted for. The undrained shear strength in each soil layer group is 1190 pounds per square foot (psf), 820 psf, and 1540 psf, respectively.

<u>Loads on Foundation</u>

As mentioned in a previous section, there will be a total of 32 columns to support the podium structure. The largest load for a podium column was 450 kips, which included the weight of the residential extension structure, the podium deck weight, and the podium column self weight. This load was assigned to all the columns to be conservative and properly account for the large loading. Therefore, the designed foundation system needed to support 450 kips per podium column. Due to this heavy loading, deep foundations are necessary for this design.

Foundation Alternatives

The most commonly used deep foundations are piles because they are economical and efficient to install. For this design, driven and cast-in-place pile foundations were considered.

Driven piles require precast piles to be hammered into the ground. The required hammering of piles posed environmental and social concerns, as the loud noise would likely disturb residents and animals. Driven piles are usually no more than 12 inches in diameter, and to properly support the loads, nine (9) piles would be necessary per podium column and 306 piles would be required to support the entire podium and extension structures.

Cast-in-place piles, also known as drilled shafts, require a hole to be excavated so that the concrete and reinforcement can be cast into the hole. These piles allow for a larger range of diameters to be installed, as this pile type can be designed having up to 48 inches diameter. If cast-in-place piles with 12 inch diameters were installed, the same amount of piles would be required as previously stated if using 12 inch driven piles. By using a larger diameter pile between 24 inches to 48 inches, each podium column could be adequately supported by a smaller amount of piles instead of nine (9). This installation method also allows for the bottom of the deep foundation to bell out to a larger diameter, which increases the amount load resistance of the foundation. For these reasons, cast-in-place drilled shafts with a belled-bottom were selected for this foundation design.

Foundation Design

Geotechnical

Sizing of the foundations was designed based on the geotechnical analysis of bearing capacity and settlement presented in the reference book titled Foundation Design: Principles and Practices by Donald P. Coduto (Coduto). Bearing capacities were met by determining the toe resistance at the bottom of the foundation element and the side friction developed along the length of the foundation shaft. Various combinations of diameters and lengths were iterated for the design. For a given belled-bottom diameter, the toe resistance was calculated in each soil layer. The total necessary skin friction was then calculated from the actual load and the toe resistance. The diameter and length of the shaft needed to provide the total skin friction amount was iterated. A combination was considered to work if the length needed to provide the skin friction fell within the same soil layer as the belled-bottom toe resistance, while successfully having enough strength to resist the load. The final size of each deep foundation was determined to be a 36 inch shaft with a 48 inch belled-bottom, and a total length of 75 ft. With this size, two (2) drilled shafts would support each podium column, resulting in a total of 68 piles to support the entire structure.

Considerations were made for the connection between the podium column and the drilled shafts. Installation of a pile cap for each pair of drilled shafts would be necessary to properly distribute the load from the structure to the foundation. The sizing and design of these pile caps were outside of the scope of this geotechnical design.

Structural

Each foundation element was designed for structural integrity. The codes used for this portion of the design were the American Concrete Institute (ACI) Code and the California Building Code (CBC). A concrete strength of 4000 pounds per square inch (psi) and Grade 60 steel were used for the design. Each drilled shaft was designed for gravity loads, which included concentrated and bending loads. Lateral seismic loads would govern the design, but this was outside the scope of this project. The following table summarizes the design of each drilled shaft. The following figures below provides cross sections of the drilled shaft with the reinforcement.

Table 7: Drilled shaft structural design summary.

Shaft Diameter (in)	36
Belled-Bottom Diameter (in)	48
Total Length (ft)	75
Longitudinal Reinforcement	15 No. 11 Bars
Transverse Spiral Reinforcement	No. 6 Bars
Transverse Reinforcement Spacing in Confinement Region (inches OC)	8
Transverse Reinforcement Spacing in Confinement Region (inches OC)	12
Longitudinal Lap Splice Length (ft)	8.75

Shaft Diameter (in)	36
Transverse Lap Splice Length (ft)	4.5

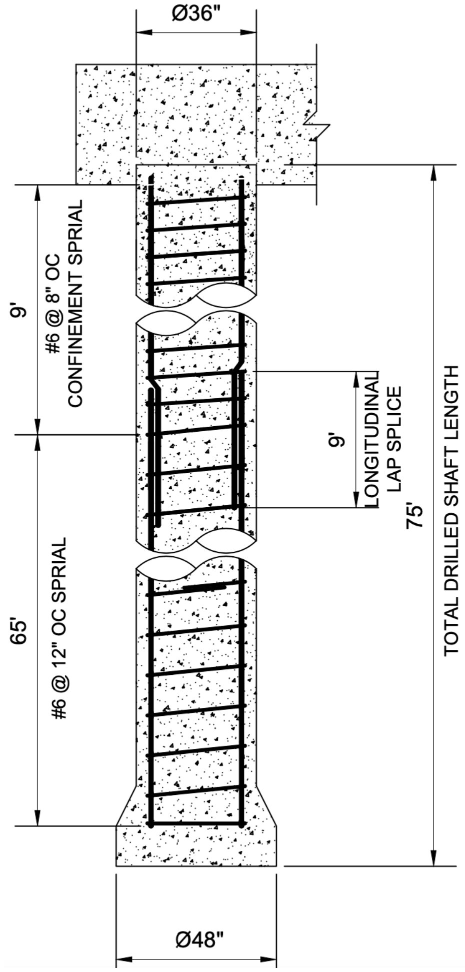

Figure 7: Drilled shaft elevation cross section.

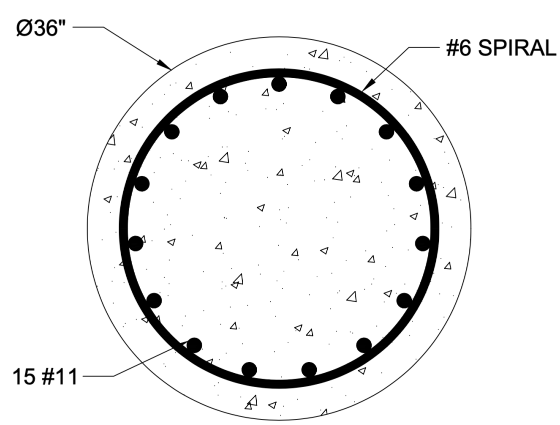

Figure 8: Spiral Reinforced Concrete Drilled Shaft Section.

Settlement Analysis

A settlement analysis was performed to verify that the drilled shafts would not exceed accepted values for foundation settlement. Settlement was determined using the O'Neill and Reese Method described in the Coduto text. The settlement of each drilled shaft would be 0.17 inches. Calculations for settlement are located in Appendix D. The reference stated that adequate settlement is less than 0.5 inches, so the expected settlement was acceptable.

Spacing and Layout

The spacing between each drilled shaft had to be large enough to minimize interaction between adjacent foundation elements. According to the Coduto text, standard center-to-center spacing between piles ranges from two to three times the drilled shaft diameter. To meet this requirement, the drilled shafts were designed to be spaced 10 ft on-center. This spacing met spacing requirements for both the shaft diameter and the belled bottom diameter. Figure 9 shows a schematic plan view of the spacing between the drilled shafts that support one podium column.

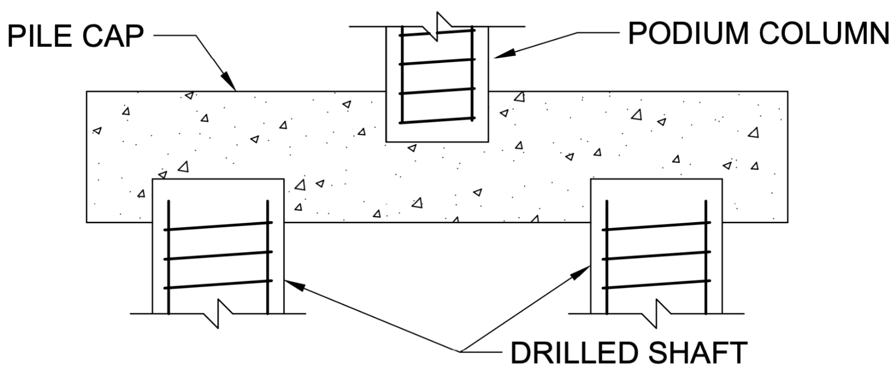

Figure 9: Schematic of drilled shafts connected to the podium column using a pile cap.

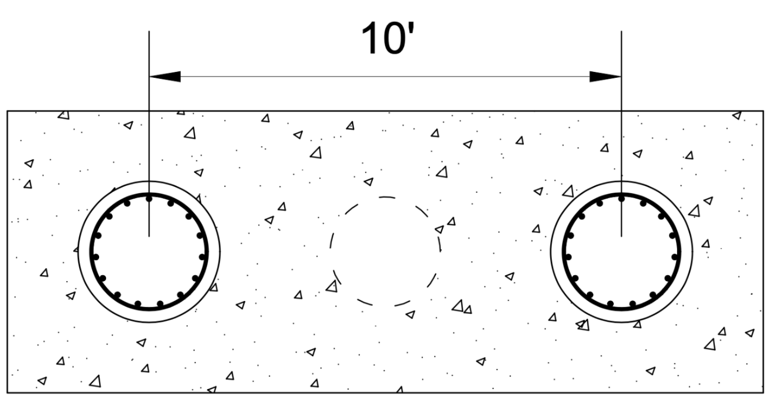

Figure 10: Plan view of drilled shaft spacing for one podium column.

Summary

The following figure shows an elevation view of the drilled shafts in relation to the podium structure and the residential expansion.

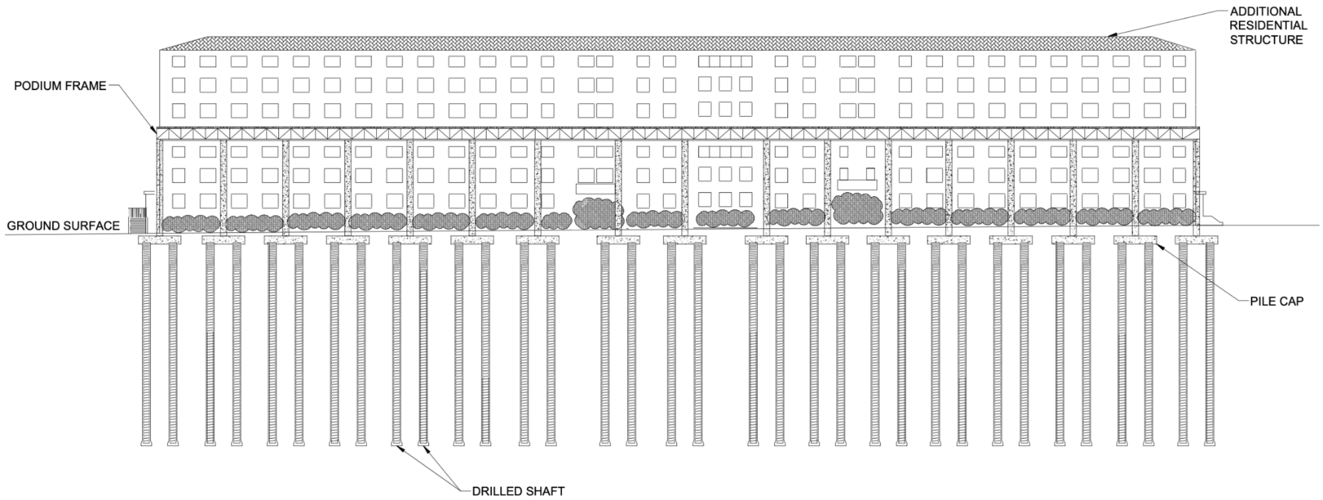

Figure 11: Elevation view of drilled shafts in relation to the ground surface, the podium structure, and the residential expansion.

07. NON-TECHNICAL CONSIDERATIONS

To complete this senior design project, various impacts of the residence hall retrofit needed to be considered. As engineers who serve the public, it is essential to consider how the project will affect the community. Ethical considerations are made based on the ASCE Code of Ethics. The ASCE Code of Ethics *Fundamental Canon*s are listed below and are referenced in the document. Ethical considerations include the rules and duties of the engineer, the rights of the people, and the consequences of the project. This document also includes social, sustainable, and environmental considerations for this project.

Fundamental Canons

1. Engineers shall hold paramount the safety, health and welfare of the public and shall strive to comply with the principles of sustainable development in the performance of their professional duties.
2. Engineers shall perform services only in areas of their competence.
3. Engineers shall act in professional matters for each employer or client as faithful agents or trustees, and shall avoid conflicts of interest.
4. Engineers shall build their professional reputation on the merit of their services and shall not compete unfairly with others.
5. Engineers shall act in such a manner as to uphold and enhance the honor, integrity, and dignity of the engineering profession and shall act with zero-tolerance for bribery, fraud, and corruption.
6. Engineers shall continue their professional development throughout their careers, and shall provide opportunities for the professional development of those engineers under their supervision.
7. Engineers shall, in all matters related to their profession, treat all persons fairly and encourage equitable participation without regard to gender or gender identity, race, national origin, ethnicity, religion, age, sexual orientation, disability, political affiliation, or family, marital, or economic status.

Ethical Considerations

Rules (Relevant canons: Canon 1-8)

The rules shall be followed strictly by the engineers and the team. The rules stated in the canons are basic guidelines to protect the safety, rights and welfare of the general public. In this project, the local communities and all students on campus will be the primary groups of people in the general public protected by these rules. If rules are not followed by the engineers and engineering team, significant consequences and damages can be caused. The project shall be constructed on the basis of ethics. If the project fails to accomplish and address the ethical rules, it should not be tolerated and will not be beneficial to the public.

Rights (Relevant canons: Canon 1, 3, 6, 8)

The people whose rights will be prioritized in this project are the students who will live in the McLaughlin-Walsh residence hall, students who live in nearby residence halls, the general student population, and stakeholders such as the Board of Trustees. The rights of the people include safety during the construction process and full disclosure of information that directly affects them, such as project purpose and goals. People also have the right to express concerns and thoughts, especially since the residence hall retrofit has a significant impact on daily life and the housing decisions made by students. The primary problem this project addresses is the lack of student housing to accommodate the growing SCU student population. Due to this problem, it can be said that one of the rights of the people is to have sufficient on-campus housing options.

Duties (Relevant canons: Canon 1-8)

The duties of the engineer include all items stated in the ASCE Code of Ethics. These duties include ensuring public safety in the design and construction of the project. An important duty to consider is working in the engineer's area of competence. Since this project is being completed by two engineers with specific scopes, it is unethical for the engineers to do work outside of their scope because that may put the public at risk. It is also the duty of the engineer to disclose aspects of the project that will not be considered over the course of the project, which shows that the engineers understand their limitations and do not complete portions of the project that are out of their understanding. It is also part of the engineer's responsibility to ensure that as the project continues into the future, the needs of the clients are still being met along the way.

Consequences

In the retrofit project, the scope of work involved the determination of whether the existing foundation, columns, and beams are able to withstand additional loads from adding extra stories. During those corresponding scopes of work, it is possible to discover that the existing foundation, columns, or beams are not able to take on any more additional load. The discovery will have to be solved by a design decision. While making the design decision, various impacts might occur. If the foundation cannot take the additional loads from the stories, the team will have to analyze the different design and the environmental impact, social impact, and the code of ethics. It is important to decide in alignment with the code of ethics, especially in terms of the rules and rights. Similarly, if the structural analysis shows insufficient support when adding additional stories, the design was analyzed and the decision was made upon the most ethical and least negatively impactful option. Furthermore, a very important decision that was considered is the social impact. Where shall the students currently living in the residence hall be relocated once the project construction phase starts. The decision was made carefully and all relevant code of ethics (rights and safety) should be reviewed prior to finalizing the decision.

Social Considerations

One social impact this project will have is the displacement of students in McLaughlin-Walsh Hall. To retrofit the building safely, no person can be living in the existing structure. This issue is concerning because displacing students is the opposite of the project's overall goal of providing more housing. Construction on McLaughlin-Walsh reduces the available housing to students.

By changing the existing building, there would be a shift in existing resident hall culture. It may not be drastic, but adding more floors, more students, more community facilitators will change the dynamic that currently exists. A positive change could be an increase in diverse residential spaces, since McLaughlin-Walsh's residential learning community theme is Unity.

Additional housing will make staying on-campus more available to students. This will reduce stress and extra costs for students who do want to stay on-campus but have not been able to in recent years. An increased number of available housing units, however, may cause SCU to admit even more students than they already are, which is the same issue this problem seeks to resolve.

The additional housing on campus will impact the surrounding community by decreasing the needs of housing in the community. This will pose a positive and a negative impact. Positively looking, the decrease in student housing needs will provide housing openings and opportunities in the neighborhood; and other people, such as employees of nearby organizations will have more housing options. On the other hand, the decrease in students' housing needs might impact the surrounding landlords or people who purchased a house specifically to rent out. Both impacts were considered closely.

Sustainable Considerations

One of the most noticeable and impactful factors towards sustainability of the retrofit is the type of materials used. In order to comply with the existing structure, the project will utilize the potential solution of designing a structure using the same materials, steel and concrete. It is known, however, that concrete is one of the most unsustainable (in terms of environmental impact and carbon emissions) existing construction materials. Once the concrete fails, although it can be repurposed to be added into a new concrete mix, it cannot be reused to construct other structures. Steel, although able to be recycled, requires enormous amounts of energy to fabricate and to go through the recycling process. When concrete experiences damage or once demolished due to failure, the process of disposing concrete involves either landfill disposal or being recycled to be used in concrete work. Not all failed concrete is qualified to be recycled into concrete works, and some failed concrete will be disposed of in the landfill, which in the long term impact poses a great threat to the environment.

The second sustainability impact of the retrofit project is the project itself. The project has the goal and purpose of providing a long term solution and a permanent structure. The structure, though requires steel and concrete, will provide extra housing for the students in future decades. The social impact of the residence hall will exceed the environmental impact made from the project, giving it a positive sustainability impact overall.

Environmental Considerations

A significant environmental impact of the retrofit project is the amount of environmental contamination caused on the surrounding area. Wastes disposal, construction debris and water contamination are all adequate considerations prior to the start of the project. The construction debris, having the most environmental impact, will cause the air quality to decrease, which will cause the surrounding natural organisms to be impacted. Animals are very important in the ecosystem and the overall environment. Santa Clara University campus is known to have various species of birds and squirrels, which build their habitat around and on the existing structures. During the construction period, the existing buildings and the surrounding spaces can be impacted, therefore their habitat might potentially be impacted as well. This aspect of the environmental impact will not be as intense or severe, but it was taken into consideration throughout the project.

The vertical extension for this project will be aiming to have a better environmental impact and will be referencing the LEED credits. The potential implementation of solar panels will allow more efficient energy to be used and decrease the amount of negative impacts energy poses on the environment. Aspects such as water usage, window positions (lighting purposes), insulation, and waste management systems were all considered in the project. All of the categories mentioned above can positively impact the surrounding environment and make the building more beneficial in the long run.

08. PRELIMINARY COST ESTIMATION

The cost for this project was estimated with the following material items with labor included. Since the cost estimation was beyond the scope of this project, the cost estimation presented in this section was a preliminary estimate based on the quantity and materials designed in this project. The summary of the estimation can be found in Table 8 presented below. The team decided on a contingency of 8 percent due to the uncertainty in the quantity of materials, as well as the potential additional costs from the connection design. The total cost for the podium frame will be approximately 4.7 million dollars. The reference used in this section of the report can be found in **Section 11 Reference.**

Table 8: Cost Estimation Breakdown Summary.

Steel Truss Joists (48LH16)	$454,500
Steel Decking (3WxHF-36 Hi Form)	$202,280
W-section Steel Beam (W18x92)	$700,200
Reinforced Concrete (Columns)	$558,470
Reinforced Concrete (Foundations)	$1,028,355
Contingency	8%
Summary	$4,707,950

09. CONCLUSION

In summary, this senior capstone project aims to design a podium frame structure spanning over the existing Mclaughlin-Walsh wall for an additional dormitory to be constructed to accommodate the increasing number of admitted students at Santa Clara University. The Design package includes structural design, geotechnical design and various other considerations. With the design for the podium frame, the team hopes to create more opportunities for the university to expand student's living space and have more housing provided in the future.

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APPENDIX A – FIGURES AND TABLES

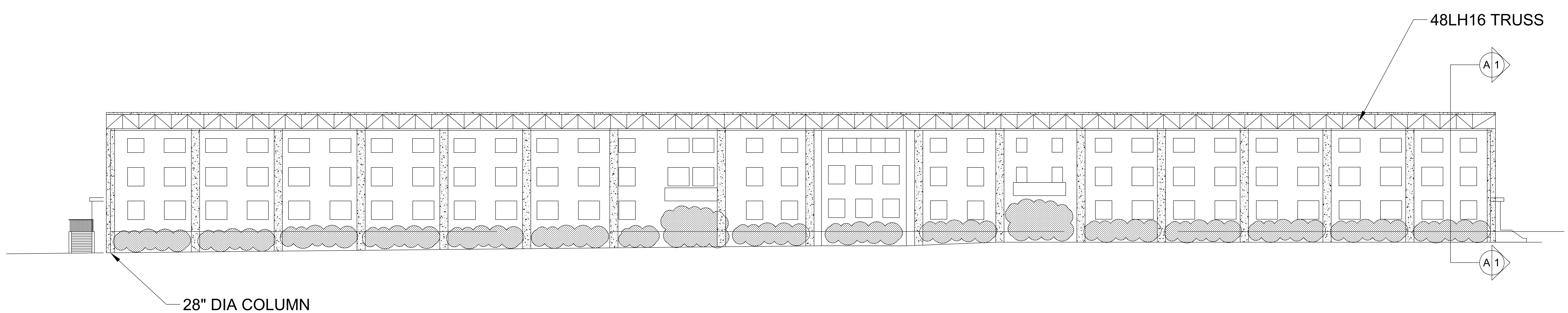
Table A-1: Alternative Analysis Matrix.

	Active Campus Feasibility	Land Use Efficiency	Proximity to Campus	Sustainability	Project Duration
Weight	5	4	4	3	3
Apartment Complex	10	7	8	3	1
Residence Hall Retrofit	3	10	10	7	7
Tiny Houses	10	1	8	9	9

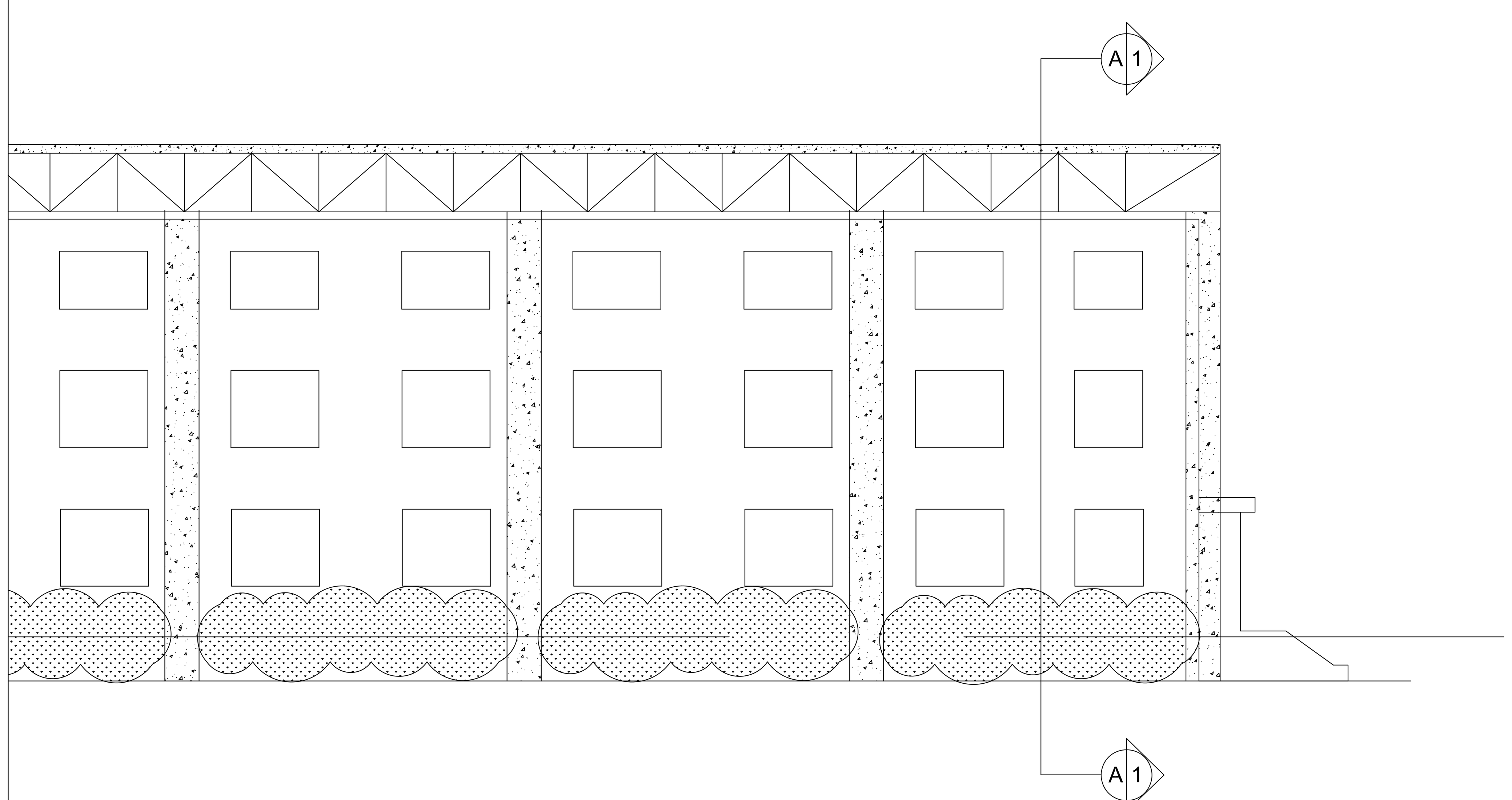
APPENDIX B – PLANS

List of Design Plans

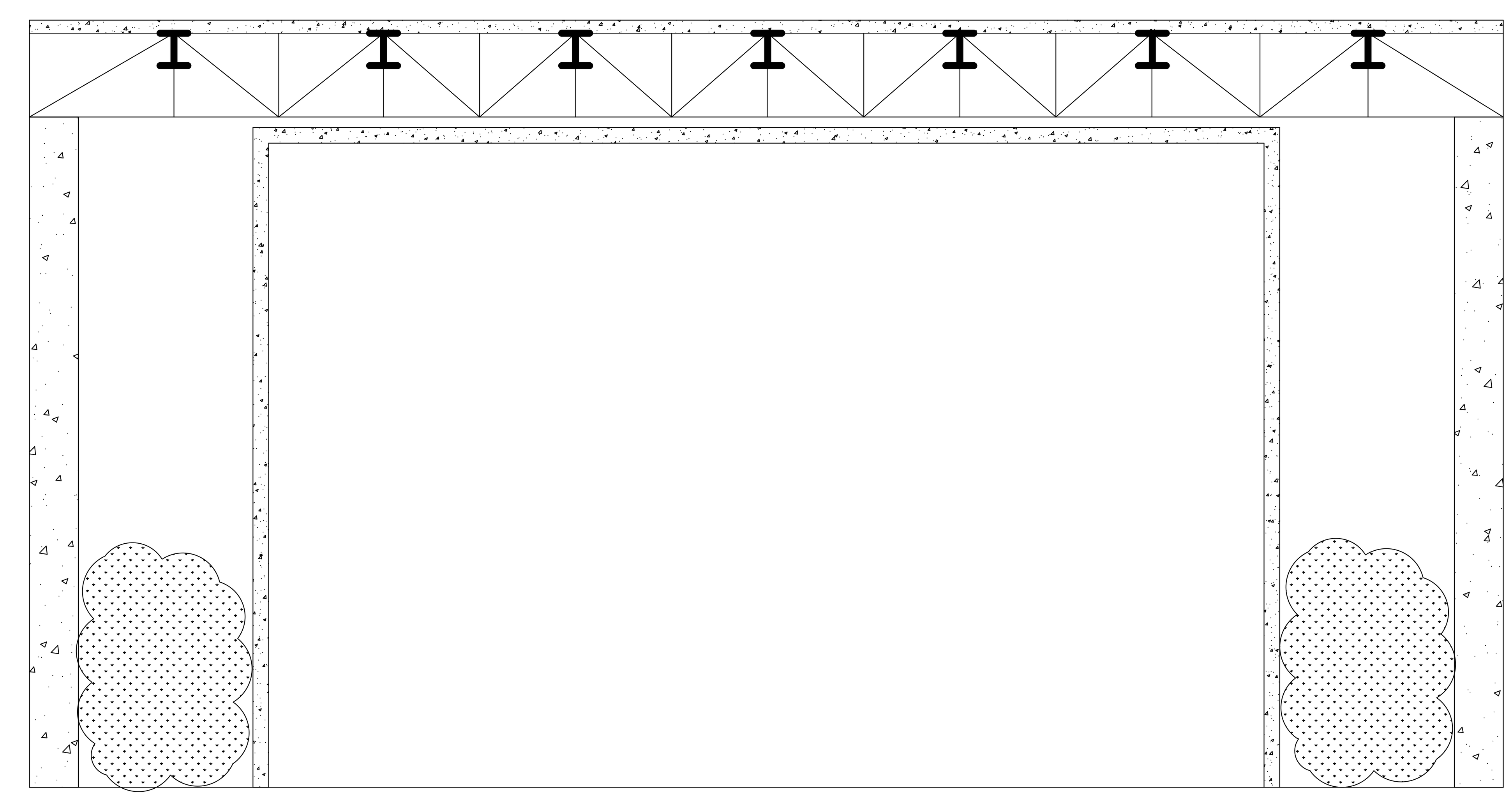
- S1. North Elevation View
- S2. Section Cut
- S3. North Elevation View with Expansion
- S4. North Elevation View with Drilled Shafts



STRUCTURAL ENGINEER REBECCA HUANG	SCALE
GEOTECHNICAL ENGINEER SHANELLE SMITH	3/32" = 1'- 0"
PROJECT MCLAUGHLIN-WALSH HALL VERTICAL EXTENSION	S1
SHEET TITLE MCLAUGHLIN-WALSH RESIDENCE HALL NORTH ELEVATION VIEW	



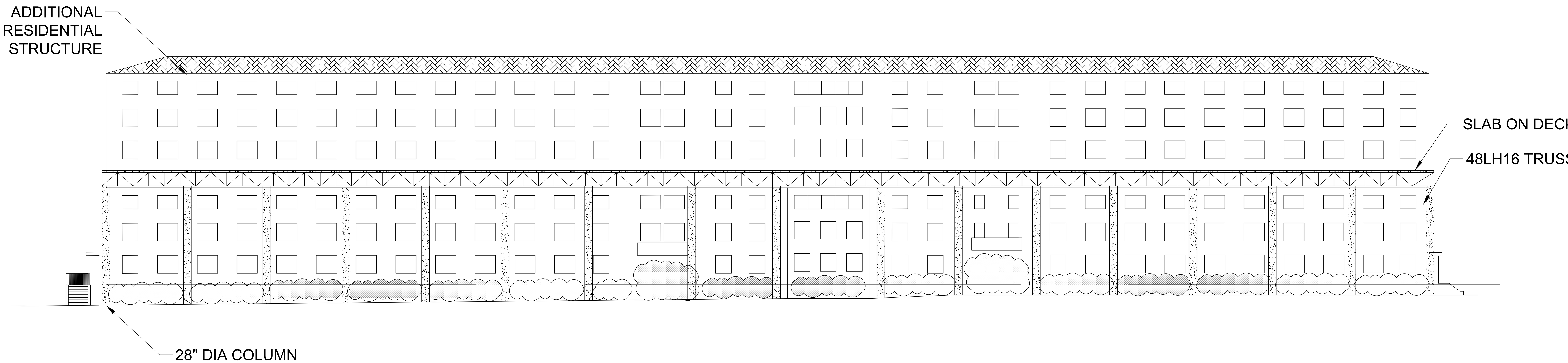
1 NORTH ELEVATION VIEW WITH SECTION A-A CUT
SCALE: 3/16" = 1'-0"



2 SECTION CUT A-A
SCALE: 1/4" = 1'-0"

STRUCTURAL ENGINEER REBECCA HUANG	SCALE See Details
GEOTECHNICAL ENGINEER SHANELLE SMITH	
PROJECT MCLAUGHLIN-WALSH HALL VERTICAL EXTENSION	S2
SHEET TITLE MCLAUGHLIN-WALSH RESIDENCE HALL SECTION CUT	

STRUCTURAL ENGINEER REBECCA HUANG	SCALE
GEOTECHNICAL ENGINEER SHANELLE SMITH	3/32" = 1'- 0"
PROJECT MCLAUGHLIN-WALSH HALL VERTICAL EXPANSION	S4
SHEET TITLE MCLAUGHLIN-WALSH RESIDENCE HALL NORTH ELEVATION VIEW WITH DRILLED SHAFTS	



STRUCTURAL ENGINEER REBECCA HUANG	SCALE 3/32" = 1'- 0"
GEOTECHNICAL ENGINEER SHANELLE SMITH	
PROJECT MCLAUGHLIN-WALSH HALL VERTICAL EXPANSION	S3
SHEET TITLE MCLAUGHLIN-WALSH RESIDENCE HALL NORTH ELEVATION VIEW WITH EXPANSION	

APPENDIX C – STRUCTURAL DESIGN

McLaughlin - Self Weight

Vertical Framing Members

$L_{M.perim} := 2 \cdot (199.5\ ft + 49\ ft) = 497\ ft$

$n_{M.perim} := \text{round}\left(\frac{(L_{M.perim})}{16\ in}\right) = 373$

$L_{M.hall} := ((2 \cdot 199.5\ ft) - 32\ (2\ ft + 10\ in)) = 308.333\ ft$

$n_{M.hall} := \text{round}\left(\frac{(L_{M.hall})}{16\ in}\right) = 231$

$L_{M.room} := (29\ (21\ ft)) = 609\ ft$

$n_{M.room} := \text{round}\left(\frac{(L_{M.room})}{16\ in}\right) = 457$

$qty_{M.door} := 3\ (32)$

$n_{M.door} := 2\ (qty_{M.door}) = 192$

$qty_{M.window} := 3\ (32)$

$n_{M.window} := 2\ (qty_{M.window}) = 192$

Horizontal Framing Members

$L_{M.floor} := 199.5\ ft$

$n_{M.floor} := \text{round}\left(\frac{(L_{M.floor})}{16\ in}\right) = 150$

$L_{M.roof} := 199.5\ ft$

$n_{M.roof} := \text{round}\left(\frac{(L_{M.roof})}{16\ in}\right) = 150$

Total Self Weight

$$W_{M.vert} := \left(\begin{array}{l}((n_{M.perim} + n_{M.hall} + n_{M.room}) \cdot 28.5\ ft) \\ + (n_{M.door} \cdot 80\ in) + (n_{M.window} \cdot 4.33\ ft)\end{array}\right) \cdot 2\ in \cdot 6\ in \cdot 34\ pcf = 91.658\ kip$$

$$W_{M.horiz} := \left(\begin{array}{l}(((n_{M.floor} \cdot 3) + n_{M.roof})\ 49\ ft \\ + (n_{M.window} \cdot 5\ ft) + (n_{M.door} \cdot 3\ ft)\end{array}\right) \cdot 2\ in \cdot 6\ in \cdot 34\ pcf = 87.652\ kip$$

$W_{M.self.kip} := W_{M.vert} + W_{M.horiz} = 179.31\ kip$

$$W_{M.self.psf} := \frac{W_{M.self.kip}}{(9.869 \cdot 10^3)\ ft^2} = 18.169\ psf$$

McLaughlin - Gravity Loads

Dead

$D_{floor} := 31\ psf$ includes floor, misc, tile

$D_{roof} := 20\ psf$

Live

$L_{floor} := 100\ psf$

$L_{roof} := 20\ psf$

$L_{str.dist} := 100\ psf$

$L_{str.pnt} := 500\ lbf$

Is this per staircase or per flight of stairs?

Floor Area

$A_M := (199.5\ ft \cdot 49\ ft) + ((2 \cdot (11\ ft + 8\ in)) \cdot 4\ ft) = (9.869 \cdot 10^3)\ ft^2$

Factored Loads - Without additional stair load

$W_{M.grav} := 3\ (1.2\ D_{floor} + 1.6\ L_{floor}) = 591.6\ psf$

$W_{M.roof} := 1.2\ D_{roof} + 1.6\ L_{roof} = 56\ psf$

How will internal stairs tie together between existing and new?

Factored Loads - With additional stair loads

$W_{M.grav.str} := 3\ (1.2\ D_{floor} + 1.6\ L_{str.dist}) = 591.6\ psf$

$W_{M.roof.str} := 1.2\ (D_{roof}) + 1.6\ (L_{roof}) = 56\ psf$

Total Gravity Loads

$W_{M.T.psf} := W_{M.grav} + W_{M.roof} = 647.6\ psf$

$W_{M.T.kip} := W_{M.T.psf} \cdot A_M = (6.391 \cdot 10^3)\ kip$

$W_{M.T.str.psf} := W_{M.grav.str} + W_{M.roof.str} = 647.6\ psf$

$W_{M.T.str.kip} := W_{M.T.str.psf} \cdot A_M = (6.391 \cdot 10^3)\ kip$

Walsh - Self Weight

Vertical Framing Members

$L_{W.perim} := 2 \cdot (164.5\ ft + 49\ ft) = 427\ ft$

$n_{W.perim} := \text{round}\left(\dfrac{(L_{W.perim})}{16\ in}\right) = 320$

$L_{W.hall} := ((2 \cdot 164.5\ ft) - 25\ (2\ ft + 10\ in)) = 258.167\ ft$

$n_{W.hall} := \text{round}\left(\dfrac{(L_{W.hall})}{16\ in}\right) = 194$

$L_{W.room} := (12 + 11)\ (20\ ft + 10\ in) = 479.167\ ft$

$n_{W.room} := \text{round}\left(\dfrac{(L_{W.room})}{16\ in}\right) = 359$

$qty_{W.door} := 3\ (32)$

$n_{W.door} := 2\ (qty_{W.door}) = 192$

$qty_{W.window} := 3\ (30)$

$n_{W.window} := 2\ (qty_{W.window}) = 180$

Horizontal Framing Members

$L_{W.floor} := 164.5\ ft$

$n_{W.floor} := \text{round}\left(\dfrac{(L_{W.floor})}{16\ in}\right) = 123$

$L_{W.roof} := 164.5\ ft$

$n_{W.roof} := \text{round}\left(\dfrac{(L_{W.roof})}{16\ in}\right) = 123$

Total Self Weight

$$W_{W.vert} := \begin{pmatrix} (n_{W.perim} + n_{W.hall} + n_{W.room}) \cdot 28.5\ ft \\ + (n_{W.door} \cdot 80\ in) + (n_{W.window} \cdot 4.33\ ft) \end{pmatrix} \cdot 2\ in \cdot 6\ in \cdot 34\ pcf = 76.33\ kip$$

$$W_{W.horiz} := \begin{pmatrix} ((n_{W.floor} \cdot 3) + n_{W.roof})\ 49\ ft \\ + (n_{M.window} \cdot 5\ ft) + (n_{W.door} \cdot 3\ ft) \end{pmatrix} \cdot 2\ in \cdot 6\ in \cdot 34\ pcf = 72.658\ kip$$

$W_{W.self.kip} := W_{W.vert} + W_{W.horiz} = 148.988\ kip$

$$W_{W.self.psf} := \frac{W_{W.self.kip}}{(8.154 \cdot 10^3)\ ft^2} = 18.272\ psf$$

Walsh - Gravity Loads

Dead

$D_{floor} = 31\ psf$ includes floor, misc, tile

$D_{roof} = 20\ psf$

Live

$L_{floor} = 100\ psf$

$L_{roof} = 20\ psf$

$L_{str.dist} = 100\ psf$

$L_{str.pnt} = 500\ lbf$

Is this per staircase or per flight of stairs?

Assume to be 'per staircase', for now.

Floor Area

$A_W := (164.5\ ft \cdot 49\ ft) + ((2 \cdot (11\ ft + 8\ in)) \cdot 4\ ft) = (8.154 \cdot 10^3)\ ft^2$

Factored Loads - Without additional stair load

$W_{W.grav} := 3\ (1.2\ D_{floor} + 1.6\ L_{floor}) = 591.6\ psf$

$W_{W.roof} := 1.2\ D_{roof} + 1.6\ L_{roof} = 56\ psf$

Factored Loads - With additional stair loads

How will internal stairs tie together between existing and new?

$$W_{W.grav.str} := 3\ (1.2\ D_{floor} + 1.6\ L_{str.dist}) + 1.6 \left(\frac{L_{str.pnt}}{(17\ ft + 7.5\ in) \cdot (12\ ft + 3.5\ in)} \right) = 595.293\ psf$$

$W_{W.roof.str} := 1.2\ (D_{roof}) + 1.6\ (L_{roof}) = 56\ psf$

Total Gravity Loads

$W_{W.T.psf} := W_{W.grav} + W_{W.roof} = 647.6\ psf$

$W_{W.T.kip} := W_{W.T.psf} \cdot A_W = (5.28 \cdot 10^3)\ kip$

$W_{W.T.str.psf} := W_{W.grav.str} + W_{W.roof.str} = 651.293\ psf$

Lobby - Self Weight

Vertical Framing Members

$$L_{L.perim} := (2 \cdot ((24\ ft + 2\ in) + (49\ ft) - (10\ ft))) = 126.333\ ft$$

$$n_{L.vert} := \text{round}\left(\frac{(L_{L.perim})}{16\ in}\right) = 95$$

$$qty_{L.door} := 3\ (8)$$

$$n_{L.door} := 2\ (qty_{L.door}) = 48$$

$$qty_{L.window} := 3\ (6)$$

$$n_{L.window} := 2\ (qty_{L.window}) = 36$$

Horizontal Framing Members

$$L_{L.floor} := 49\ ft$$

$$n_{L.floor} := \text{round}\left(\frac{(L_{L.floor})}{16\ in}\right) = 37$$

$$L_{L.roof} := 49\ ft$$

$$n_{L.roof} := \text{round}\left(\frac{(L_{L.roof})}{16\ in}\right) = 37$$

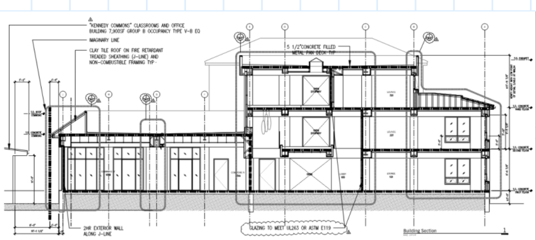
Total Self Weight

$$W_{L.vert} := \left(n_{L.vert} \cdot 28.5\ ft + (n_{L.door} \cdot 80\ in) + (n_{L.window} \cdot 4.33\ ft)\right) \cdot 2\ in \cdot 6\ in \cdot 34\ pcf = 9.02\ kip$$

$$W_{L.horiz} := \left(((n_{L.floor} \cdot 3) + n_{L.roof}) \cdot (24\ ft + 2\ in) + (n_{L.door} \cdot 3\ ft) + (n_{L.window} \cdot 5\ ft)\right) \cdot 2\ in \cdot 6\ in \cdot 34\ pcf = 11.052\ kip$$

$$W_{L.self.kip} := W_{L.vert} + W_{L.horiz} = 20.071\ kip$$

$$W_{L.self.psf} := \frac{W_{L.self.kip}}{(2.335 \cdot 10^3)\ ft^2} = 8.596\ psf$$



Source for pcf of Doug Fir lumber

https://roofonline.com/weights-measures/weight-of-dimensional-lumber/

Lobby - Gravity Loads

Dead

$D_{floor} = 31\ psf$ includes floor, misc, tile

$D_{roof} = 20\ psf$

Live

Elevator loads - should only be concentrated

$L_{floor} = 100\ psf$

$L_{roof} = 20\ psf$

$L_{elev.dist} := 200\ psf$

$L_{elev.pnt} := 5000\ lbf$

Floor Area

$A_L := (24\ ft + 2\ in) \cdot 49\ ft = (1.184 \cdot 10^3)\ ft^2$

$A_{L.elev} := (9.5\ ft \cdot 8\ ft) = 76\ ft^2$

Factored Loads - Without additional elevator load

$W_{L.grav} := 3\ (1.2\ D_{floor} + 1.6\ L_{floor}) = 591.6\ psf$

$W_{L.roof} := 1.2\ D_{roof} + 1.6\ L_{roof} = 56\ psf$

Factored Loads - With additional elevator loads

$$W_{L.grav.elev} := 3\ (1.2\ D_{floor}) + 1.6\ \left(\frac{L_{elev.pnt}}{A_{L.elev}}\right) = 216.863\ psf$$

$W_{L.roof.elev} := 1.2\ (D_{roof}) + 1.6\ (L_{roof}) = 56\ psf$

Total Gravity Loads

$W_{L.T.psf} := W_{L.grav} + W_{L.roof} = 647.6\ psf$

$W_{L.T.kip} := W_{L.T.psf} \cdot A_L = 766.866\ kip$

$W_{L.T.elev.psf} := W_{L.grav.elev} + W_{L.roof.elev} = 272.863\ psf$

Summary

	Self Weight	Gravity Loads
McLaughlin	$W_{M.self.psf} = 18.169\ psf$	$W_{M.T.psf} = 647.6\ psf$
	$W_{M.self.kip} = 179.31\ kip$	$W_{M.T.kip} = \left(6.391 \cdot 10^{3}\right)\ kip$
		$W_{M.T.str.psf} = 647.6\ psf$
		$W_{M.T.str.kip} = \left(6.391 \cdot 10^{3}\right)\ kip$
Walsh	$W_{W.self.psf} = 18.272\ psf$	$W_{W.T.psf} = 647.6\ psf$
	$W_{W.self.kip} = 148.988\ kip$	$W_{W.T.kip} = \left(5.28 \cdot 10^{3}\right)\ kip$
		$W_{W.T.str.psf} = 651.293\ psf$
Lobby	$W_{L.self.psf} = 8.596\ psf$	$W_{L.T.psf} = 647.6\ psf$
	$W_{L.self.kip} = 20.071\ kip$	$W_{L.T.kip} = 766.866\ kip$
		$W_{L.T.elev.psf} = 272.863\ psf$

Deck Loads

$$W_D := 75.1\ psf$$

$$A_D := \left(49\ ft + 9.5\ ft + 8.5\ ft\right) \cdot \left(199.5\ ft + 164.5\ ft + 24\ ft + 2\ in\right) = \left(2.601 \cdot 10^{4}\right)\ ft^2$$

$$W_{D.kip} := W_D \cdot A_D = \left(1.953 \cdot 10^{3}\right)\ kip$$

Design for the Column C-3 stories 10-12 and 13-14

Assumptions and considerations

a. Columns are tied columns

1. P_u from the upper timber residence structure

$$P_{u.McLaugh.frame} := \left(5.823 \cdot 10^3\right) \, kip$$

$$P_{u.Walsh.frame} := \left(4.811 \cdot 10^3\right) \, kip$$

$$P_{u.connector.frame} := 698.658 \, kip$$

$$P_{deck} := 76.9 \, psf \cdot (65 \, ft \cdot 390 \, ft) = \left(1.949 \cdot 10^3\right) \, kip$$

$$P_{truss} := 34.3 \, plf \cdot 65 \, ft \cdot 16 = 35.672 \, kip$$

$$P_{beam} := (390 \, ft \cdot 6 \cdot 192 \, plf) = 449.28 \, kip$$

$$P_{u.column.each} := \frac{\left(P_{u.Walsh.frame} + P_{u.McLaugh.frame} + P_{u.connector.frame} + P_{deck} + P_{truss} + P_{beam}\right)}{32} = 430.22 \, kip$$

$e := 2.5 \, in$ - P_u is applied at an eccentricity of 2.5in

2. Preliminary Dimensioning for the columns

a. Slenderness Check for the column

$$L := 29 \, ft$$

$330 \, mm = 12.992 \, in$

$$I := \frac{\pi \cdot r^4}{4} \qquad A := \pi \cdot r^2$$

$$R := \frac{\left(\frac{\pi \cdot r^4}{4}\right)}{\pi \cdot r^2} \qquad R := \frac{r}{4}$$

$$KLu\_R := 22$$

$K := 0.65$ - for fixed rotation and fixed translation on both end

$$R := \frac{(K \cdot L)}{22} = 10.282 \, in$$

$$r := R \cdot 4 = 3.427 \, ft$$

$r_{slender} := 3.5\ ft$

b. Fire resistance for column (3hr rating) according to IBC 7.22

$Dia_{column} := 10\ in$

Column Preliminary Dimension

$$r_{final} := \max\left(r_{slender}, \frac{(Dia_{column})}{2}\right) = 3.5\ ft$$

Try Slender Column

$H := L = 29\ ft$

$f'c := 4\ ksi$

$P_{u.column.each} = 430.22\ kip$

$f_y := 60\ ksi$

$e_{bot} := 2\ in$

$e_{top} := 3\ in$

Moment for the top of column and the bottom of column based on load and eccentricity

$M_{top} := P_{u.column.each} \cdot e_{top} = 107.555\ kip \cdot ft$

$M_{bot} := P_{u.column.each} \cdot -(e_{bot}) = -71.703\ kip \cdot ft$

Typically by definition, the greater magnitude moment will be taken as M2 and and in this case $M_1 := M_{bot}$ and $M_2 := M_{top}$

$R_{M1\_M2} := \frac{M_1}{M_2} = -0.667$ the ratio is negative because $n := 1$, the column only bends in single curvature

Attempt with calculating the cross sectional area, $\varphi_g := 0.015$, which is the most economical in the real world

$$A_{g.trial} := \frac{P_{u.column.each}}{(0.4 \cdot (f'c + (f_y \cdot \varphi_g)))} = 219.5\ in^2$$

Obtain the column sizes, since using circular column, the radius and diameter is as follow

$r_{trial} := \sqrt{\frac{A_{g.trial}}{\pi}} = 8.359\ in$ use $\boxed{r_{trial}} := 10\ in$

$D_{trial} := r_{trial} \cdot 2 = 20\ in$

Check for Slenderness (need to consider slenderness)
Design the system to be a non-sway frame

$K := 0.8$ - consider the top to be pinned and bottom to be fixed

$KL_u := K \cdot L = 23.2\ ft$

$$r := \sqrt{\frac{\left(\frac{\pi \cdot D_{trial}^4}{64}\right)}{\frac{\pi \cdot D_{trial}^2}{4}}} = 5\ in$$

$$KL_{u\_}r := \frac{KL_u}{r} = 55.68$$

$$\begin{aligned} &\text{if } KL_{u\_}r \le 34 + 12 \cdot \left(\frac{M_1}{M_2}\right) \\ &\quad \| \text{“Short Column”} \\ &\text{else} \\ &\quad \| \text{“Slender Column”} \end{aligned} = \text{“Slender Column”}$$ - ACI code 6.8.5

Since the column is considered Slender, the trial diameter and radius will most likely be insufficient, therefore, using increments of 2in, the second iteration of trial diameter will be 14in (radius) and 28in (diameter)

$r_{2.it} := 14\ in$ $D_{2.it} := 28\ in$ $A_g := \frac{\pi \cdot D_{2.it}^2}{4} = 615.752\ in^2$

Check whether the moments are less than the minimum

$$\begin{aligned} M := &\text{ if } \max\left(e_{top}, e_{bot}\right) \ge \left(0.6\ in + 0.003\ D_{2.it}\right) \\ &\quad \| \max\left(\left|M_1\right|, \left|M_2\right|\right) \\ &\text{ else} \\ &\quad \| P_{u.column.each} \cdot \left(0.6 + 0.003\ D_{2.it}\right) \end{aligned} = 107.555\ kip \cdot ft$$

Compute EI_{eff}

$$E_c := 57000 \cdot \sqrt{4000} \cdot psi = \left(3.605 \cdot 10^3\right)\ ksi$$

$$I_g := \left(\frac{\pi \cdot D_{trial}^{\ 4}}{64}\right) = \left(7.854 \cdot 10^3\right)\ in^4$$

$$L_{dead} := \frac{\left(\begin{array}{l}864.308\ kip + 20.071\ kip + 366.923\ kip + 148.988\ kip \\ + 444.098\ kip + 179.31\ kip\end{array}\right)}{32} = 63.241\ kip$$

$$\beta_{dns} := \frac{\left(1.2 \cdot L_{dead}\right)}{P_{u.column.each}} = 0.176$$

$$EI_{eff} := \frac{\left(0.4 \cdot E_c \cdot I_g\right)}{\left(1 + \beta_{dns}\right)} = \left(6.686 \cdot 10^4\right)\ kip \cdot ft^2$$

$$K_{tot} := \left(32 \cdot \frac{\left(12 \cdot EI_{eff}\right)}{(29\ ft)^3}\right) = 87.719\ \frac{kip}{in}$$

Magnified moment calculation

$$P_c := \frac{\left(\pi^2 \cdot EI_{eff}\right)}{KL_u^{\ 2}} = \left(1.226 \cdot 10^3\right)\ kip$$ - Textbook 12-25

$$C_m := 0.6 - 0.4 \left(\frac{M_1}{M_2}\right) = 0.867$$ - Textbook 12-14

$$\delta := \frac{C_m}{1 - \frac{P_{u.column.each}}{0.75 \cdot P_c}} = 1.629$$ - Textbook 12-24

$$M_c := \delta \cdot M_2 = 175.186\ kip \cdot ft$$

The column will be designed as a circular section with radius of 14in and diameter of 28in

Design the reinforcement

$$C_{internal} := 1.5\ in$$

$$\gamma := \frac{\left(D_{2.it} - 2 \cdot C_{internal} - 2\ (0.5\ in) - 1.41\ in\right)}{D_{2.it}} = 0.807$$

- Assume No.4 stirrups and No.11 longitudinal reinforcements are used

Assume $\phi P_u := P_{u.column.each} = 430.22\ kip$

$$\phi P_{u\_}A_g := \frac{\phi P_u}{A_g} = 0.699\ ksi$$

Assume $\phi M_u := M_c = 175.186\ ft \cdot kip$

$$\phi M_{u\_}A_g d := \frac{\phi M_u}{A_g \cdot D_{2.it}} = 0.122\ ksi$$

With Interaction diagram for circular column and f'c=4ksi, fy=60ksi

$\varphi_{g.0.75} := 0.01$

$\varphi_{g.0.90} := 0.01$

- the point is seen to be within the 0.01 steel composition curve on both interaction diagram. Since the minimum of 0.01 steel has to be used as required by ACI, will use $\boxed{\varphi_g} := 0.01$

$A_s := A_g \cdot \varphi_g = 6.158\ in^2$ - try 8 No. 110 bars

$A_{10} := 1.27\ in^2$

$A_v := 8 \cdot A_{10} = 10.16\ in^2$ - OK!

Seismic Moment Analysis

$$K_{tot} = 87.719\ \frac{kip}{in}$$

$P_{u.mass} := P_{u.column.each} \cdot 32 = \left(1.377 \cdot 10^4\right)\ kip$

$$T := (2\ \pi) \cdot \sqrt{\frac{P_{u.column.each}}{K_{tot} \cdot 364\ \frac{in}{sec^2}}} = 0.729\ s$$

$S_a := 1.04\ g$

$V := P_{u.mass} \cdot 1.04 = \left(1.432 \cdot 10^4\right)\ kip$

$$V_{each} := \frac{V}{32} = 447.428\ kip$$

Spiral Design

$$d := D_{2.it} - C_{internal} - \left(\frac{1}{2} \cdot 1.27\ in\right) = 25.865\ in$$

$$V_c := \left(2 \cdot \sqrt{4000} \cdot 1.5 \cdot 28\right)\ kip = \left(5.313 \cdot 10^3\right)\ kip$$

$$A_{s.5} := 0.31\ in^2$$

$$S_{req} := \frac{A_{s.5} \cdot 2 \cdot f_y \cdot d}{V} = 0.067\ in$$

$$A_{c.min\_s} := \max\left(\frac{0.75 \cdot \sqrt{4000}\ psi \cdot d}{f_y}, \frac{50\ psi \cdot d}{f_y}\right) = 0.022\ in$$

$$s_{max.avmin} := min\left(\frac{A_{s.5} \cdot 2 \cdot f_y}{0.75 \cdot \sqrt{4000}\ psi \cdot d}, \frac{A_{s.5} \cdot 2 \cdot f_y}{50\ psi \cdot d}\right) = 28.765\ in$$

$$S_{prov} := 28\ in$$

$$0.75 \cdot \left(V_c + 8 \cdot \sqrt{4000}\ psi \cdot A_g\right) = \left(4.218 \cdot 10^3\right)\ kip$$

$$0.75 \cdot \left(V_c + \frac{\left(2 \cdot A_{s.5}\right) \cdot f_y \cdot d}{28\ in}\right) = \left(4.01 \cdot 10^3\right)\ kip$$

Truss Design

1. Determine the truss depth and span ratio

Reference Depth to Span Ratio from the Leavey Center

$Span_{leavey} := 225\ ft$ - from Leavey truss structural plans S3.1

$Depth_{leavey} := 6\ ft + 10\ in = 6.833\ ft$

$$R_{leavey} := \frac{Depth_{leavey}}{Span_{leavey}} = 0.03$$

Basic structural data

$W := 31.5\ ft$

$L := 68\ ft$

The spacing of the truss will be conservation, as it will use the largest spacing in the structure

Using the Leavey center Depth to Span Ratio to estimate the depth of the truss for the podium frame

$Span_{podium} := 49\ ft + 9.5\ ft \cdot 2 = 68\ ft$

$Depth_{podium} := Span_{podium} \cdot R_{leavey} = 2.065\ ft$

Use $Depth_{pod} := 3\ ft$ for initial iteration

2. Determine the design loading on the truss

Convert the load into usable dead and live load

$w_{dead.Mc} := 65\ psf$ $w_{live.Mc} := 320\ psf$

$w_{dead.wal} := 65\ psf$ $w_{live.wal} := 320\ psf$

$w_{dead.conn} := 65\ psf$ $w_{live.conn} := 320\ psf$

$w_{dead} := \max\left(w_{dead.conn}, w_{dead.wal}, w_{dead.Mc}\right) = 65\ psf$

$w_{selfweight.truss} := 0.04\ klf$ - from educated engineering judgement, will verify later

$w_{dead.ex} := w_{dead} \cdot 378.658\ ft = 24.613\ klf$ - multiply by tributary width

$w_{dead.tot} := w_{dead.ex} + w_{selfweight.truss} = 24.653\ klf$

$w_{live} := 320\ psf$

$w_{live.tot} := w_{live} \cdot 378.658\ ft = 121.171\ klf$

$w_{dead.live.working} := w_{live.tot} + w_{dead.tot} = 145.823\ klf$

$w_{dead.live.ultimate} := 1.6 \cdot w_{live.tot} + 1.2\ w_{dead.tot} = 223.456\ klf$

$w_{dead.live.serviceability} := 0.7 \cdot w_{live.tot} + 1 \cdot w_{dead.tot} = 109.472\ klf$

3. Determine the section using deflection

Break down of the deflection equation

$$\Delta_{limit.total} := \frac{L}{240} = 3.4\ in$$

- CBC

$$\Delta_{limit.live} := \frac{L}{360} = 2.267\ in$$

$w_{serviceability} := w_{dead.live.serviceability} = 109.472\ klf$

$L = 68\ ft$

$E := 29000\ ksi$ - Elasticity modulus for steel

$$I_{total} := \frac{5 \cdot w_{serviceability} \cdot \left(L^4\right)}{384 \cdot E \cdot \Delta_{limit.total}} = 534127.513\ in^4$$

$$I_{live} := \frac{5 \cdot 0.7 \cdot w_{live.tot} \cdot \left(L^4\right)}{384 \cdot E \cdot \Delta_{limit.live}} = 620765.637\ in^4$$

Middle section Dimension

$$c := \left(\sqrt{(3\ ft)^2 + (28\ in)^2}\right) = 3.801\ ft$$

Reaction

$$P_{des} := \frac{w_{dead.live.serviceability} \cdot L}{2} = 3722.054\ kip$$

$P_{n.fac} := 1189\ plf \cdot L = 80.852\ kip$

$$M_{des} := \frac{\frac{(w_{dead.live.ultimate})}{15 \cdot 6} \cdot L^2}{8} = 1435.086\ kip \cdot ft$$

$$w := \frac{(w_{dead.live.ultimate})}{15 \cdot 6}$$

$$M_{max} := M_{des} = 1435.086\ kip \cdot ft$$

$$L_{trib} := 32\ ft$$

$$M_A := \left(\frac{(w \cdot L_{trib})}{2} \cdot \frac{L_{trib}}{4}\right) - \left(w \cdot \left(\frac{L_{trib}}{4}\right)^2\right) \cdot \frac{1}{2} = 238.353\ kip \cdot ft$$

$$M_B := \frac{(w \cdot L_{trib}{}^2)}{8} = 317.804\ kip \cdot ft$$

$$M_C := \frac{3 \cdot w \cdot L^2}{32} = 1076.314\ kip \cdot ft$$

$$C_b := \frac{12.5 \cdot (M_{max})}{2.5\ M_{max} + 3\ M_A + 4\ M_B + 3\ M_C} = 2.038$$

Since the depth of the truss will be relatively small, will consider pre-fabricated truss.

According to the steel truss joist catalog and aligning the desired span of the truss, which is 65 ft, the first iteration option is 48LH16 truss.

$$P_{safe\_65} := 76950\ plf \cdot 68\ ft = 5232.6\ kip$$

$$P_{des} = 3722.054\ kip$$

The deflection check according to the table attached in below, is considered within safe load.

Beam design (beam framing into the truss)

$$\boxed{L_{trib}} := 32\ ft$$

Using the longest length throughout the whole project and applying throughout for conservative design

$$\boxed{P_{des}} := \left(\frac{(w_{dead.live.serviceability})}{15 \cdot 6} \cdot L_{trib}\right) = 38.923\ kip$$

$$\Delta_{limit.beam} := \frac{L_{trib}}{240} = 1.6\ in$$

$$w_{serv.defl} := w_{live} + w_{dead} = 385\ psf$$

$$I_{beam.req} := \frac{5 \cdot \frac{(w_{serviceability})}{15 \cdot 6} \cdot (L_{trib}{}^4)}{384 \cdot E \cdot \Delta_{limit.beam}} = 618.48\ in^4$$

Pre-dimensioning according to fire rating

Effective span to depth ratio

Slenderness check (AISC 360-16,Chp. B, Table B4.1b) - *to illustrate User Note*

$F_y := 60\ ksi$

$E := 29000\ ksi$

$\lambda_{p\_flange} := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 8.354$

$\lambda_{r\_flange} := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 21.985$

$\lambda_{p\_web} := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 82.663$

$\lambda_{r\_web} := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 125.314$

Try beam size "18X192"

Choose the section W18X192

$Section := \text{"W18X192"}$

$Beamsize := \| \text{vlookup}(Section, M, 0) = [\text{"W18X192"}]$

$w := \| \text{vlookup}(Section, M, 1) \| \cdot plf = 192\ plf$

$A := \| \text{vlookup}(Section, M, 2) \| \cdot in^2 = 56.2\ in^2$

$d := \| \text{vlookup}(Section, M, 3) \| \cdot in = 20.4\ in$

$b_f := \| \text{vlookup}(Section, M, 4) \| \cdot in = 11.5\ in$

$t_w := \| \text{vlookup}(Section, M, 5) \| \cdot in = 0.96\ in$

$t_f := \| \text{vlookup}(Section, M, 6) \| \cdot in = 1.75\ in$

$bf/2tf := \| \text{vlookup}(Section, M, 7) \| = 3.27$

$h/tw := \| \text{vlookup}(Section, M, 8) \| = 16.7$

$I_x := \| \text{vlookup}(Section, M, 9) \| \cdot in^4 = 3870\ in^4$

$Z_x := \| \text{vlookup}(Section, M, 10) \| \cdot in^3 = 442\ in^3$

$S_x := \| \text{vlookup}(Section, M, 11) \| \cdot in^3 = 380\ in^3$

$r_x := \|\text{vlookup}(Section, M, 12)\| \cdot in^3 = 8.28\ in^3$

$I_y := \|\text{vlookup}(Section, M, 13)\| \cdot in^4 = 440\ in^4$

$Z_y := \|\text{vlookup}(Section, M, 14)\| \cdot in^3 = 119\ in^3$

$S_y := \|\text{vlookup}(Section, M, 15)\| \cdot in^3 = 76.8\ in^3$

$r_y := \|\text{vlookup}(Section, M, 16)\| \cdot in = 0.233\ ft$

$J := \|\text{vlookup}(Section, M, 17)\| \cdot in^4 = 0.002\ ft^4$

$C := \|\text{vlookup}(Section, M, 18)\| \cdot in = 38000\ in$

$r_{ts} := 1.72\ in$ $\quad$ $h_o := 23.1\ in$

$$Flange_{wt\_ratio} := \left\| \begin{array}{l} \text{if } bf/2tf \le \lambda_{p\_flange} \\ \quad \| \text{“C”} \\ \text{else if } bf/2tf \ge \lambda_{r\_flange} \\ \quad \| \text{“S”} \\ \text{else} \\ \quad \| \text{“NC”} \end{array} \right\| = \text{“C”}$$

$$Web_{wt\_ratio} := \left\| \begin{array}{l} \text{if } h/tw \le \lambda_{p\_web} \\ \quad \| \text{“C”} \\ \text{else if } h/tw \ge \lambda_{r\_web} \\ \quad \| \text{“S”} \\ \text{else} \\ \quad \| \text{“NC”} \end{array} \right\| = \text{“C”}$$

$c := 1.0$ $\quad$ $C_b = 2.038$ $\quad$ $L_b := 32\ ft$ $\quad$ $\phi_b := 0.9$

$M_{px} := F_y \cdot Z_x = 2210\ kip \cdot ft$ $\quad$ $M_{rx} := 0.7 \cdot F_y \cdot S_x = 1330\ kip \cdot ft$

$$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 8.996\ ft$$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot 60\ ksi}{29000\ ksi}\right)^2}} = 20.629\ ft$$

$$F_{cr} := \frac{C_b \cdot \pi^2 \cdot E}{\left(\frac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_o} \cdot \left(\frac{L_b}{r_{ts}}\right)^2} = 53.365 \; ksi$$

$$M_n := \begin{array}{l} \text{if } L_b \le L_p \\ \quad \| M_{px} \\ \text{else if } L_b > L_r \\ \quad \| min\left(M_{px}, F_{cr} \cdot S_x\right) \\ \text{else} \\ \quad \| min\left(M_{px}, C_b \cdot \left(M_{px} - \frac{\left(M_{px} - M_{rx}\right)}{\left(L_r - L_p\right)} \cdot \left(L_b - L_p\right)\right)\right) \end{array} = 1689.901 \; kip \cdot ft$$

$$\phi_b M_n := \phi_b \cdot M_n = 1520.911 \; kip \cdot ft$$

SHEAR STRENGTH - AISC 360-16 (w/o tension-field action), Chp. G, Section G2

$h\_tw_{lim} := 2.24 \cdot \sqrt{\frac{E}{F_y}} = 49.246$ $\qquad k_v := 5.34$ ... no transverse stiffeners

Per Section G2.1(a) $\quad C_{v1} := 1.0 \qquad \phi_v := 1.00 \qquad A_w := d \cdot t_w = 19.584 \; in^2$

User Note: All current ASTM A6 W, S and HP shapes except W44×230, W40×149, W36×135, W33×118, W30×90, W24×55, W16×26 and W12×14 meet the criteria stated in Section G2.1(a) for F_y = 50 ksi (345 MPa).

The following illustrates the calc. for Cv1

$$C_{v1} := \begin{array}{l} \text{if } h/tw \le h\_tw_{lim} \\ \quad \| 1.0 \\ \text{else if } h/tw > h\_tw_{lim} \\ \quad \| \begin{array}{l} \text{if } h/tw \le 1.10 \cdot \sqrt{\frac{k_v \cdot E}{F_y}} \\ \quad \| 1.0 \\ \text{else} \\ \quad \| \dfrac{1.10 \cdot \sqrt{\frac{k_v \cdot E}{F_y}}}{h/tw} \end{array} \end{array} = 1$$

$$\phi_v V_n := \phi_v \cdot 0.6 \cdot F_y \cdot A_w \cdot C_{v1} = 705.024\ \boldsymbol{kip}$$

$$\Delta_{TL\_max} := \frac{5 \cdot \dfrac{\left(w_{serviceability}\right)}{15 \cdot 6} \cdot \left(L_{trib}{}^{4}\right)}{384 \cdot E \cdot I_x} = 0.256\ \boldsymbol{in}$$

Design Summary - DCRs

$$DCR_M := \frac{M_{des}}{\phi_b M_n} = 0.94$$

$$DCR_V := \frac{P_{des}}{\phi_v V_n} = 0.06$$

$$DCR_{\Delta\_TL} := \frac{\Delta_{TL\_max}}{\Delta_{limit.beam}} = 0.16$$

$$W_{beam} := 390\ \boldsymbol{ft} \cdot 6 \cdot 192\ \boldsymbol{plf} = 449.28\ \boldsymbol{kip}$$

$$W_{add} := \frac{W_{beam}}{32} = 14.04\ \boldsymbol{kip}$$

APPENDIX D – GEOTECHNICAL DESIGN

Structural Integrity of Piers (Coduto, Section 12.4)

The minimum drilled shaft size for this design was determined using Coduto, Section 12.4 and loads from structural design sections. The following equation provides the minimum diameter to be considered for a drilled shaft based on the loads.

$$B = \sqrt{\frac{3.86\ P}{f'_c}} \qquad \text{(Eq. 12.12)}$$

Where:

B is minimum shaft diameter

P is unfactored compressive load

f'_c is the 28-day compressive strength of concrete

For this drilled shaft design, the load per column will be 450 kips. Each column will be supported by 2 piers. Therefore the load per pier is $P_f := 225\ kip$. The unfactored load can be estimated as $P_{uf} := \frac{P_f}{1.4} = 160.714\ kip$ and $f'_c := 4000\ psi$.

Using Eq. 12.12 to solve for B_{min} gives the following result

$$B_{min} := \sqrt{\frac{3.86\ P_{uf}}{f'_c}} = 12.453\ in$$

According to Coduto Section 12.4, design diameter should be a multiple of 200mm or 6in. Therefore, the minimum design diameter for each pile should be at least **18in**.

Axial Load Capacity Based on Analytical Methods (Coduto, Chapter 14)

Sizes for shaft diameter and belled bottom diameter were iterated to properly support the load. The sample calculations of toe bearing and skin friction for the selected size and length are as follows.

Toe Bearing - Clays (Coduto, Section 14.2)

Tip Resistance

$q_t = (N_c \cdot s_u)$ — Tip resistance equation (Eq. 14-10)

$N_c := 9 \qquad s_{u3} := 1190 \ psf$ — Soil data for the third layer of soil

$q_t := (N_c \cdot s_{u3}) = 10.71 \ ksf$ — Tip resistance in the third layer of soil

$P_t = q_t \ A_t$ — Toe bearing equation

$A_t := \frac{\pi}{4} (4 \ ft)^2 = 12.566 \ ft^2$ — Area of the belled bottom

$P_t := q_t \ A_t = 134.586 \ kip$ — Total toe bearing

Skin Friction - Clays (Coduto, Section 14.3)

$f_s = \alpha \cdot s_u$ — Skin friction equation using Alpha method for clays (Eq. 14-29)

$\alpha_1 := 0.5$

$\alpha_2 := 0.8$

$\alpha_3 := 0.7$

$\alpha_4 := \alpha_3$

Alpha factors (Figure 14.13)

$s_{u1} := 1540 \ psf$

$s_{u2} := 870 \ psf$

$s_{u3} = 1190 \ psf$

$s_{u4} := s_{u3}$

Undrained shear strength in each soil layer

$f_{s1} := \alpha_1 \cdot s_{u1} = 770\ psf$

$f_{s2} := \alpha_2 \cdot s_{u2} = 696\ psf$

Eq. 14-29 for each soil layer

$f_{s3} := \alpha_3 \cdot s_{u3} = 833\ psf$

$f_{s4} := \alpha_4 \cdot s_{u4} = 833\ psf$

$A_1 := \pi \cdot 3\ ft \cdot 40\ ft = 376.991\ ft^2$

$A_2 := \pi \cdot 3\ ft \cdot 20\ ft = 188.496\ ft^2$

Surface area of a 3ft diameter drilled shaft for each layer of soil and a 4ft diameter belled bottom

$A_3 := \pi \cdot 3\ ft \cdot 14\ ft = 131.947\ ft^2$

$A_4 := 4071.5\ in^2 = 28.274\ ft^2$

$P_{s1} := f_{s1} \cdot A_1 = 290.283\ kip$

$P_{s2} := f_{s2} \cdot A_2 = 131.193\ kip$

Skin friction along the total length of the drilled shaft

$P_{s3} := f_{s3} \cdot A_3 = 109.912\ kip$

$P_{s4} := f_{s4} \cdot A_4 = 23.552\ kip$

$P_s := P_{s1} + P_{s2} + P_{s3} + P_{s4} = 554.94\ kip$

Total skin friction

$P_{all} := \dfrac{(P_t + P_s)}{3} = 229.842\ kip$

Allowable bearing capacity OK

Total Length: $L := 75\ ft$

Summary

Shaft diameter: 3ft
Belled bottom diameter: 4ft
Total drilled shaft length: 75ft

Drilled Shaft Design

The references used in the following design section were Foundation Design: Principles and Practices, 2nd Edition by Coduto; Reinforced Concrete Mechanics & Design, 8th Edition by Wright; the ACI 318-19 Code

Material Properties of Reinforced Concrete Drilled Shaft

$f'_c = 4000\ psi$

$f_y := 60\ ksi$

Diameter of Shaft

Using analytical methods from Coduto Chapter 14, shaft diameter B was selected as $B := 36\ in$ to met axial capacity requirements. This diameter will be used in the following design, as opposed to B_{min}.

Loads

The maximum factored axial and moment loads on each drilled shaft are as follows:

Axial

$P_u := 225\ kip$

Moment

$$e := \frac{11.5\ ft}{2} = 5.75\ ft$$

$$M_u := P_u \cdot e = 1293.75\ kip \cdot ft$$

Diameter of Reinforcement and γ Determination

Assuming No. 8 longitudinal bars, 0.5-inch diameter spiral reinforcement, and 3 inches of concrete cover as required by ACI , the γ will be

$$\gamma := \frac{B - 2\left(3\ in + 0.5\ in + 0.5\ in\right)}{B} = 0.778$$

Interaction Diagrams & Steel Ratio Determination

Computations for the x-axis and y-axis are below. The interaction diagrams for $\gamma = 0.75$ and $\gamma = 0.90$ are on the following page. Steel ratio will be determined by linear interpolation.

Gross section area

$$A_g := \frac{\pi}{4} B^2 = 1017.876\ in^2$$

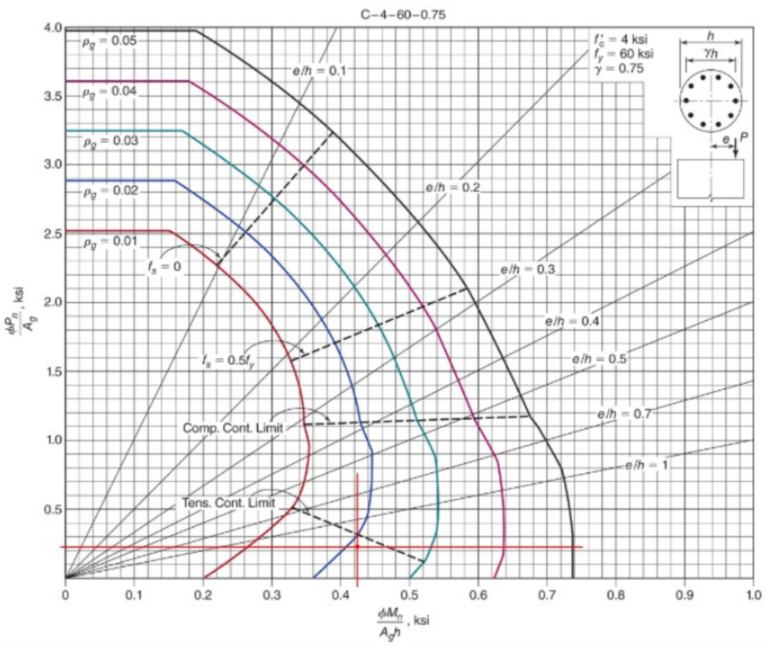
y-axis

$$\frac{P_u}{A_g} = 0.221\ ksi$$

x-axis

$$\frac{M_u}{A_g \cdot B} = 0.424\ ksi$$

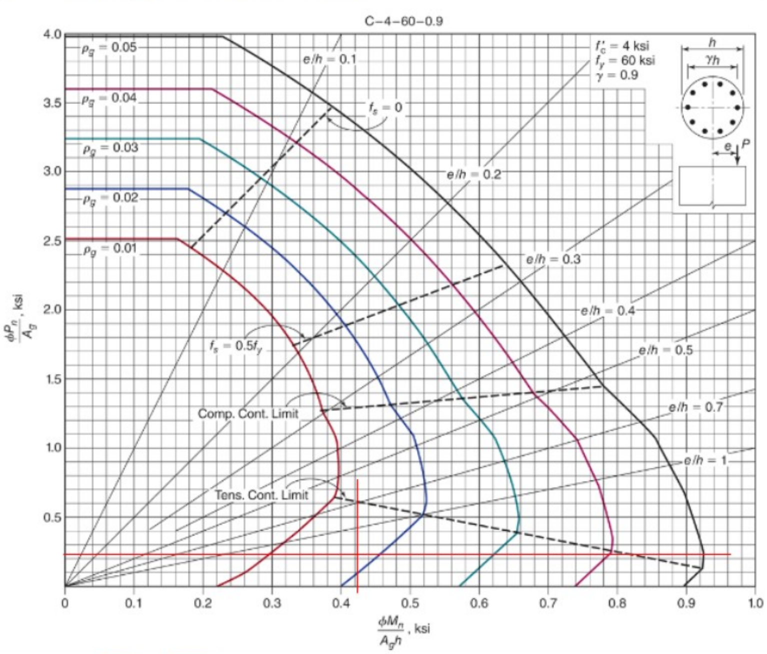
Fig. A-14b **Nondimensional interaction diagram for circular spiral column:** $f'_c = 4000$ **psi,** $f_y = 60$ **ksi and** $\gamma = 0.75$**.**



$\gamma_{0.75} := 0.75$

$\rho_{0.75} := 0.023$

Fig. A-14c **Nondimensional interaction diagram for circular spiral column:** $f'_c = 4000$ **psi,** $f_y = 60$ **ksi and** $\gamma = 0.90$**.**



$\gamma_{0.90} := 0.90$

$\rho_{0.90} := 0.018$

Note: Intersecting points on both interaction diagrams are within the tension-controlled failure zone. Tension reinforcement yields before maximum concrete strain is reached. This ensures ductile failure for the drilled shaft.

Interpolation for ρ corresponding to $\gamma = 0.778$

$$\rho_{0.778} := \rho_{0.75} + \left(\gamma - \gamma_{0.75}\right) \frac{\left(\rho_{0.90} - \rho_{0.75}\right)}{\left(\gamma_{0.90} - \gamma_{0.75}\right)} = 0.022$$

Steel Ratio

$$\rho := \rho_{0.778} = 0.022$$

Minimum Longitudinal Steel Reinforcement Requirements

Using the calculated/interpolated steel ratio, the minimum area of longitudinal steel reinforcement was calculated as follows.

Minimum area of longitudinal reinforcement

$$A_{s.req} := \frac{\rho \cdot \pi \cdot B^2}{4} = 22.469\ in^2$$

Reinforcement sizes to satisfy minimum area required:

Rebar No.	Rebar Area	# Rebar Needed	Total Area Provided
No. 8	$A_{s.8} := 0.79\ in^2$	$\#_8 := \frac{A_{s.req}}{A_{s.8}} = 28.441$	$A_{s.8.p} := 29 \cdot A_{s.8} = 22.91\ in^2$
No. 9	$A_{s.9} := 1.00\ in^2$	$\#_9 := \frac{A_{s.req}}{A_{s.9}} = 22.469$	$A_{s.9.p} := 23 \cdot A_{s.9} = 23\ in^2$
No. 10	$A_{s.10} := 1.27\ in^2$	$\#_{10} := \frac{A_{s.req}}{A_{s.10}} = 17.692$	$A_{s.10.p} := 18 \cdot A_{s.10} = 22.86\ in^2$
No. 11	$A_{s.11} := 1.56\ in^2$	$\#_{11} := \frac{A_{s.req}}{A_{s.11}} = 14.403$	$A_{s.11.p} := 15 \cdot A_{s.11} = 23.4\ in^2$

Try (15) No. 11 Bars

$$d_{b.11} := 1.41\ in$$

Check maximum axial load capacity

Strength reduction factor for tension-controlled sections: $\phi := 0.90$

$$\phi P_{n.max} := 0.85 \cdot \phi \cdot \left(0.85 \cdot f'_c \cdot \left(A_g - A_{s.11.p}\right) + \left(f_y \cdot A_{s.11.p}\right)\right) = 3660.692\ kip$$

$$\frac{P_u}{A_g} = 0.221\ ksi \qquad \frac{\phi P_{n.max}}{A_g} = 3.596\ ksi$$

$$\frac{P_u}{A_g} \le \frac{\phi P_{n.max}}{A_g} = 1$$ OK!

Spacing between bars. For longitudinal reinforcement in columns, clear spacing between bars shall be at least the greatest of 1.5 in., 1.5db, and (4/3)dagg.

$$d_{agg} := 0.75\ in$$

$$s_{min} := \max\left(1.5\ in, 1.5 \cdot d_{b.11}, \frac{4}{3} \cdot d_{agg}\right) = 2.115\ in$$

$$\boxed{s_{min}} := 2.25\ in$$

$$circumference := \pi \cdot \left(B - 2\left(3\ in + 0.5\ in + \frac{d_{b.11}}{2}\right)\right) = 86.677\ in$$

$$l_{longitudinal\_ring} := \left(15 \cdot d_{b.11}\right) + \left(15 \cdot s_{min}\right) = 54.9\ in$$

$$s_{max} := \frac{circumference - 15\left(d_{b.11}\right)}{15} = 4.368\ in$$

$$\boxed{s_{max}} := 4.25\ in$$

Minimum reinforced length. Should be the greatest of half the shaft length, 10-feet, three times the diameter, and the distance where the design cracking moment exceeds the required factored moment strength. For the sake of this design and the large loading on each drilled shaft, they will be reinforced the entire shaft length.

$$L_{s.min} := \max\left(\frac{L}{2}, 10\ ft, 3 \cdot B\right) = 37.5\ ft$$

Total length of reinforcement

$$L_{st} := 15 \cdot 75\ ft = 1125\ ft$$

<u>Minimum Transverse Reinforcement</u>

The transverse reinforcement for the drilled shafts will be spiral. Spiral reinforcement can support larger bearing capacity than tied hoop reinforcement. No.4 spiral reinforcement is required by CBC for elements with diameters larger than 20-inches.

Confinement Reinforcement. Minimum reinforcement ratio should be at least half of the larger of the following. This ratio is also equivalent to the ratio of volume of steel per turn to the volume of core per turn.

$$\rho_{s.confined} = 0.5 \cdot \max\left(\left(0.45\left(\frac{A_g}{A_{ch}} - 1\right)\left(\frac{f'_c}{f_{yt}}\right)\right), \left(0.12 \cdot \frac{f'_c}{f_{yt}}\right)\right)$$

$$A_{ch} := \pi \cdot \frac{(B - 2\ (3\ in))^2}{4} = 706.858\ in^2$$

$$f_{yt} := f_y = 60\ ksi$$

$$\rho_{s.confined} := 0.5 \cdot \max\left(\left(0.45\ \left(\frac{A_g}{A_{ch}} - 1\right)\left(\frac{f'_c}{f_{yt}}\right)\right), \left(0.12 \cdot \frac{f'_c}{f_{yt}}\right)\right) = 0.007$$

$$\rho_{s.confined} = \frac{\pi \cdot (B - cover) \cdot A_{st}}{\pi \cdot (B - cover)^2 \cdot \frac{p}{4}}$$

$$\pi \cdot (B - 6\ in) = 94.248\ in$$

$$\frac{\pi \cdot (B - 6\ in)^2}{4} = 706.858\ in^2$$

$$\rho_{s.confined} = \frac{94.248 \cdot A_{st}}{706.858\ p} = 0.007$$

If No.6 bars are used, $d_{b.6} := 0.75\ in$ and $A_{st} := 0.44\ in^2$. The pitch, p, is determined below.

$$p := \frac{94.248\ in \cdot A_{st}}{706.858\ in^2 \cdot 0.007} = 8.381\ in$$

$$p_{confined} := 8\ in$$

Non-Confinement Reinforcement. Transverse reinforcement along the element outside the confinement reinforcement region shall not exceed the following.

$$s_{t.max} := min\left(12 \cdot d_{b.11}, 0.5 \cdot B, 12\ in\right) = 12\ in$$

$$p_{nonconfined} := 12\ in$$

Lap Splice

The longitudinal bars are deformed in tension, so Table 25.5.2.1 will be used. The development length is the length needed to fully yield the reinforcement, and the splice length is the length needed to transfer the force from one bar to the next adjacent one.

Longitudinal lap splice

$\frac{A_{s.11.p}}{A_{s.req}} = 1.041$ less than 2

$$l_{st} = max\left(1.3 \cdot l_d, 12\ in\right)$$

$$l_d = max\left(\frac{f_y \cdot \psi_t \cdot \psi_e \cdot \psi_g}{20 \cdot \lambda \cdot \sqrt{f'_c \cdot psi}} \cdot d_{b.11}, 12\ in\right)$$

$\lambda := 1$ $\psi_t := 1$ $\psi_g := 1$ $\psi_e := 1.2$ (epoxy coated)

$$l_d := \max\left(\frac{f_y \cdot \psi_t \cdot \psi_e \cdot \psi_g}{20 \cdot \lambda \cdot \sqrt{f'_c \cdot psi}} \cdot d_{b.11}, 12\ in\right) = 6.688\ ft$$

$$l_{st} := \max\left(1.3 \cdot l_d, 12\ in\right) = 8.695\ ft$$

$$l_{st} := 8.75\ ft$$

Spiral lap splice. Length of spiral lap splice is the greater of 12-inches and 72 times the spiral diameter length, for deformed bars with epoxy without hooks.

$$l_{st.s} := \max\left(72 \cdot d_{b.6}, 12\ in\right) = 4.5\ ft$$

Total length of spiral reinforcement

$$L_{st.s} = n \cdot \sqrt{C^2 + p^2}$$

$n_1 := \dfrac{9\ ft}{\frac{2}{3}\ ft} = 13.5$ $p_{confined} = 0.667\ ft$ $C := \pi \cdot 30\ in = 7.854\ ft$

$n_2 := \dfrac{66\ ft}{1\ ft} = 66$ $p_{nonconfined} = 1\ ft$

$$L_{st.s} := \left(n_1 \cdot \sqrt{C^2 + p_{confined}{}^2}\right) + \left(n_2 \cdot \sqrt{C^2 + p_{nonconfined}{}^2}\right) = 628.958\ ft$$

Group efficiency check.

$$P_{ag} = \eta \cdot N \cdot P_a$$

$$P_a := P_u = 225\ kip$$

$$N := 2$$

$$\eta = 1 - \theta \cdot \frac{((n-1) \cdot m) + ((m-1) \cdot n)}{90 \cdot m \cdot n}$$

$m := 1$

$n := 2$

$B = 3\ ft$

$s := 11.5\ ft$

$$\theta := \text{atan}\left(\frac{B}{s}\right) = 14.621\ deg$$

$$\eta := 1 - \theta \cdot \frac{((n-1) \cdot m) + ((m-1) \cdot n)}{90 \cdot m \cdot n} = 0.999$$

$$P_{ag} := \eta \cdot N \cdot P_a = 449.362\ kip$$

The pile group is efficient and will adequately support the load.

Summary

Longitudinal Reinforcement: 15 No.11 bars
Transverse Spiral Reinforcement: No.6 bars
Longitudinal Lap Splice Length: 9 ft
Transverse Lap Splice Length: 4.5 ft
Transverse Spiral Confinement Spacing: 8 inch OC
Transverse Spiral Non-Confinement Spacing: 12 inch OC
Settlement per Drilled Shaft: 0.45 inch

Settlement Analysis

O'Neill and Reese's Method

The O'Neill and Reese Method is used to estimate settlements for drilled shafts in clays. Figures 14.16 and 14.17 in the Coduto text were used for this analysis.

Inputs

$B=3\ ft$ $\qquad P_s=554.94\ kip$

$B_b:=4\ ft$ $\qquad P_t=134.586\ kip$

$E:=57000\cdot\sqrt{f'_c\cdot psi}=3604.997\ ksi$ $\qquad P_{all}=229.842\ kip$

Finding an upper deflection bound

Try $\delta_1:=0.5\ in$

$\dfrac{\delta_1}{B}=1.4\%$ From Figure 14.26 $\quad P_{s.1}:=0.85\ \left(P_s\right)=471.699\ kip$

$\dfrac{\delta_1}{B_b}=1\%$ From Figure 14.27 $\quad P_{t.1}:=0.43\ \left(P_t\right)=57.872\ kip$

$P_{T.1}:=P_{s.1}+P_{t.1}=529.571\ kip$

$$\delta_{1.adj}:=\delta_1+\frac{P_{T.1}\cdot 0.75\cdot L}{A_g\cdot E}=0.597\ in$$

Finding a lower deflection bound

Try $\delta_2:=0.05\ in$

$\dfrac{\delta_2}{B}=0.1\%$ From Figure 14.26 $\quad P_{s.2}:=0.3\ \left(P_s\right)=166.482\ kip$

$\dfrac{\delta_2}{B_b}=0.1\%$ From Figure 14.27 $\quad P_{t.2}:=0.03\ \left(P_t\right)=4.038\ kip$

$P_{T.2}:=P_{s.2}+P_{t.2}=170.52\ kip$

$$\delta_{2.adj}:=\delta_2+\frac{P_{T.2}\cdot 0.75\cdot L}{A_g\cdot E}=0.081\ in$$

Interpolate for allowable load

$$\delta:=\delta_{2.adj}+\left(\delta_{1.adj}-\delta_{2.adj}\right)\frac{\left(P_{all}-P_{T.2}\right)}{\left(P_{T.1}-P_{T.2}\right)}=0.167\ in$$ OK!

Total settlement: 0.17 inch