The University of Iowa
IOWA INITIATIVE
FOR SUSTAINABLE COMMUNITIES

FINAL DELIVERABLE

Title	Clinton - Downtown Commercial Building Rehabilitation (239 5th Ave South)
Completed By	Brad Brown, Jarod Concha, Sean Stevens
Date Completed	December 2019
UI Department	Department of Civil & Environmental Engineering
Course Name	CEE:4850:0001 Project Design & Management
Instructor	Christopher Stoakes
Community Partners	Downtown Clinton Alliance

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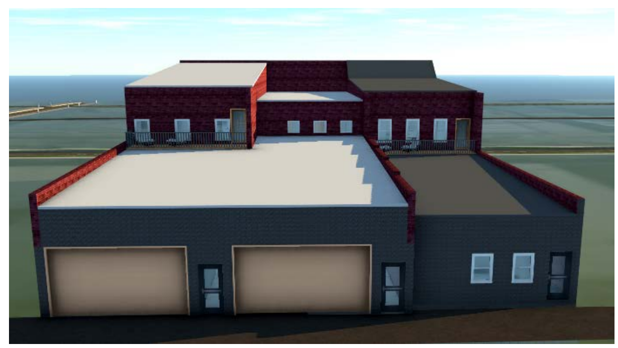
COMMERCIAL BUILDING REHABILIATION

239 5TH AVENUE SOUTH
CLINTON, IOWA 52732



PREPARED FOR: KAREN ROWELL, DIRECTOR OF DOWNTOWN CLINTON ALLIANCE

DATE SUBMITTED: DECEMBER 13TH , 2019

SUBMITTED BY: JBS CONSULTANTS

BRAD BROWN, PROJECT MANAGER
JAROD CONCHA, REPORT PRODUCTION
SEAN STEVENS, TECH SERVICES

Civil and Environmental Engineering
College of Engineering
The University of Iowa

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I. EXECUTIVE SUMMARY

The following report comprises the analysis, design, and cost estimate for the rehabilitation of three two-story buildings at 239 5th Ave. S, Clinton, Iowa. The main objective of this project is to renovate the second floors of the buildings to be used as residential space. This goal opened many possibilities for design, including the addition of parking garages to the first floor, new back entrances to the apartments, and second floor balconies. Other objectives expressed by the client include the restoration of the building façade, removal of the boilers and hazardous materials in the basement and partitioning the bar to better utilize the space. The existing structural systems, including floors and bearing walls, were sized and analyzed to ensure the buildings provide adequate strength for habitation. Due to limited access to the second-floor system, joist sizes had to be assumed and analyzed to resist residential live and dead loads. Arrangement of the partition walls of the second-floor residential units were made to accommodate one- and two-bedroom apartments. Demolition of existing and installation of new partition walls has been proposed to augment the floor plans of all the second-floor residential units and two of the first-floor commercial spaces.

Proposed additions to the buildings' structural systems, from the ground up, include two-car parking garages for residential use, entrance and exit staircases, a raised hallway, and walk-out balconies. The garages would be behind the insurance office and bar (middle building). The garage floors have been designed as slab-on-grade, with granular backfill subgrade supported by a mechanically reinforced concrete retaining wall. Demolition of part of the existing first floor system and south-side exterior wall is required to excavate and install the concrete garage pad and new garage openings. Header members have been sized to support the garage doors. Addition of a garage would coincide with changes in the floor plan of the bar to better utilize the space. The interior staircases are necessary to provide back entrance/exit points for the residential units and coincides with changes in the floor plan of the collector's shop and second floor apartment. The balconies would be installed in the apartments above the collector's shop (easternmost building) and the insurance office (westernmost building). Due to differing elevations in the existing floor and roof systems, installation of the balconies would include demolition of part of the existing roof to make space for the new framing members.

Cost estimates have been conducted for the demolition and construction of new structural systems, as well as for the refurbishment of the building façade and cleanup of the basements and apartments. The façade of the collector's shop will be restored to its original condition by removing the existing cladding. Although there are parts of the façade that are in obvious need of repair, contingencies have been included in the cost estimates for tuckpointing and replacement of clay masonry where visual inspection cannot be conducted. This includes places where the original façade is obstructed by existing cover materials that will be removed. Basement renovations include grouting of the limestone foundation walls and removal of the large cast iron boilers that are no longer in use. One of the boilers is set into a concrete pad that is at a lower elevation than the existing floor; this recess will be filled after removal of the boiler. Also included is the abatement of hazardous materials in the basements. For the apartments, old floor cover and parts of the sheathing will be removed and replaced. When going through the demolition, it is recommended to ensure the joists in the second-floor system are at least as large as the sizes used in design. If not, they will need to be replaced to support ASCE 7-16 residential live loads.

II. ORGANIZATION QUALIFICATIONS AND EXPERIENCE

Organization Name: JBS Consultants

Organization Location and Contact Information

Location:

Seamans Center
103 South Capitol Street
Iowa City, IA 52242

Contact Information:

Brad Brown (Project Manager)
Email: bradley-brown@uiowa.edu
Cell Phone: (815)-341-5140

Organization and Design Team Description

The JBS design team consists of a group of civil engineering students enrolled in the Senior Capstone Design Course at the University of Iowa. At JBS Consultants, our expertise is in structural design and we aim to deliver a product that is not only safe, but also accommodates our clients' desires. Each of the team members role in the project and area of expertise are as follows:

Brad Brown, Project Manager: Brad is a 4th year student pursuing a civil engineering major with a business administration minor. Brad's project roles include communication with the client, coordination of meetings, and cost estimate production. Brad has management experience through his internship with Golf Construction. Throughout his tenure at Golf he shadowed various project managers and assisted them with tasks that included assistance with estimates and concrete inspections. He was also able to gain some experience in façade restoration where he learned the various repairs that may come up during a restoration project.

Jarod Concha, Report Production: Jarod is a 4th year civil engineering major with a focus in pre-architecture. Jarod's role in the project was to conduct the structural analysis and structural design for all three of the existing buildings. Jarod has expertise in programs like MathCAD and is well-versed in structural analysis. Jarod has had past engineering experience on various design projects, including designing apartment foundations in Champaign, IL and designing wood framing for floor, roof, and bearing walls.

Sean Stevens, Technology Services: Sean is a 4th year civil engineering major with a focus in structures. Sean's role in the project was to produce the Revit model as well as construction drawings for the team's designs. Sean has experience in various computer design programs including AutoCAD, Revit, Robot and ArcGIS. Sean has also gained structural design experience through the courses he took at the University of Iowa. The structural design courses include Design of Wood Structures, Foundation of Structures and Structural Systems for Buildings. In Design of Wood Structures, he gained experience with analyzing various roof and truss systems. In Foundation of Structures, Sean gained experience in analyzing foundations and retaining walls. In Structural Systems for Buildings, he was able to analyze building systems using Revit.

III. DESIGN SERVICES

Project Scope

The main objective of this project is to rehabilitate the second-floor spaces to accommodate one- and two-bedroom apartments. Included in this is the addition of amenities such as parking and balconies, and the addition of new entrance and exit stairways to comply with the 2012 International Building Code (IBC). Other requests expressed by the client include restoring the brick façade, removing boilers and hazardous materials from the basement, and partitioning the bar to better utilize the space. Proposed changes in the bar were discussed with the client, including the addition of a back patio or garage parking for tenants. It was determined that garages would be preferable, as it would increase the value of the apartments. Originally, the foundation walls, floor, and roof systems were to be analyzed to ensure they provide adequate strength for residential and commercial use. However, it was determined that only those systems that would receive new loads would need to be assessed. These systems include the first and second floor joists and beams as well as the clay masonry bearing walls. Since no modifications are being made to the roof, it was deemed unnecessary to analyze the roof framing. As for the foundation, after visual inspection the design team concluded that re-grouting and tuckpointing is required to maintain structural strength of the limestone.

Currently, the residential spaces above the insurance office and bar have an open floor plan. They share a hallway that leads to the front and back entrances, but the floor elevations differ by approximately two feet. The design team proposed two solutions to the change in floor elevation. The first solution was to raise the floor and roof system above the bar to increase the south-facing windows and add space for a balcony. The second solution was to raise part of the connecting hallway. The latter design alternative was chosen because raising the floor and roof systems would be too expensive. The units above the collector's shop are partitioned but require complete redesign of the floor layout in accordance with client request and the IBC window to floor area ratio. The client gave JBS Consultants the freedom to choose how to partition the residential spaces, and it was determined that one- and two-bedroom units would best suit the needs of potential tenants. Two, two-bedroom units have been proposed for the spaces above the collector's shop and bar, and two one-bedroom units for the space above the insurance office.

Existing and proposed floor plans, as well as demolition plans, have been drawn to specify the extent of work for the residential spaces. Design of the garage includes the addition of a mechanically stabilized concrete retaining wall to support the backfill to the slab-on-grade pad of the garage floor. Demolition of the floor system and excavation under the bar is required for this addition, as well as changes to the layout of the bar. Minor changes to the floor plan of the collector's shop are also included to accommodate the new exit staircases. The second-floor residential units above the collector's shop and insurance office will receive balconies that span from bearing wall to bearing wall. Ten-foot parapet privacy walls have also been included with the balcony design. Contingencies have been included in the cost estimate for façade demolition and repair, as well as cost estimates for removal of the boilers and general cleaning in the basements.

Work Plan

The following Gantt Chart shows the work schedule that was utilized for the design that was created. All project activities were completed over a 14-week period. Design work began on Monday, September 9th, 2019 and concluded on Friday, December 13th, 2019.

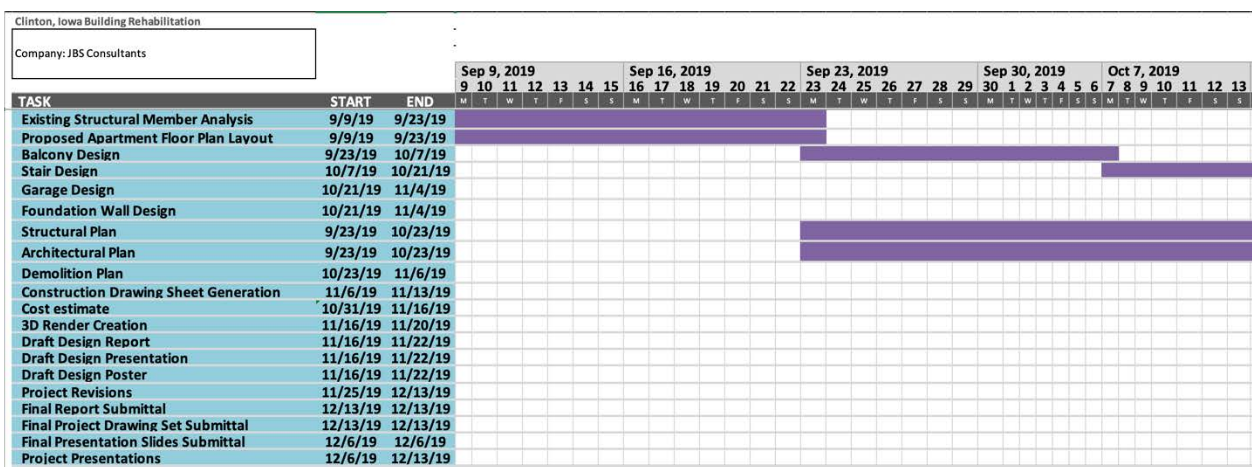

Figure 1: Displays Gantt Chart for Work Completed from the September 9th, 2019 – October 13th, 2019

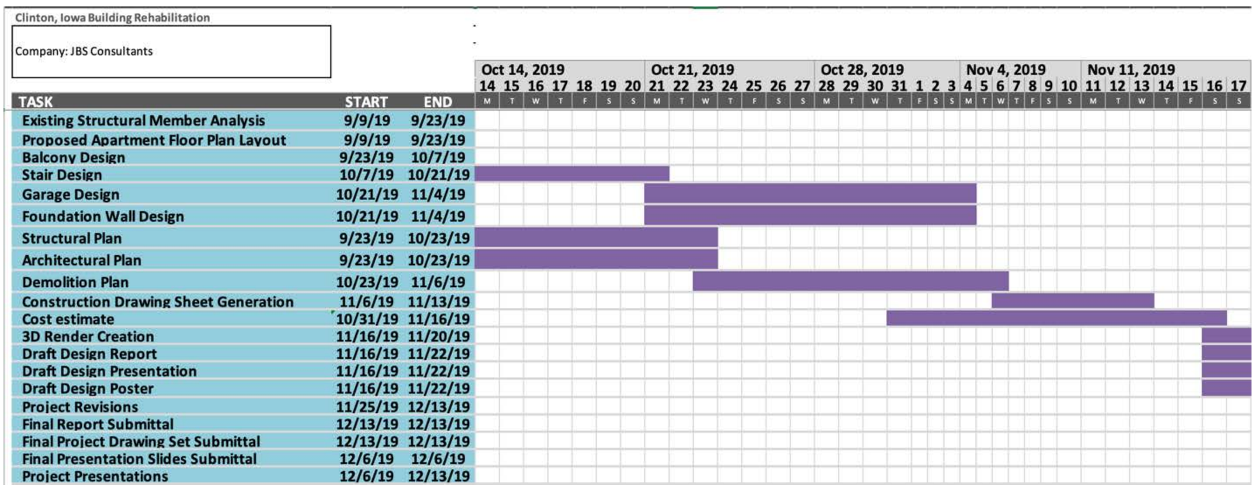

Figure 2: Displays Gantt Chart for Work Completed from the October 14th, 2019 – November 17th, 2019

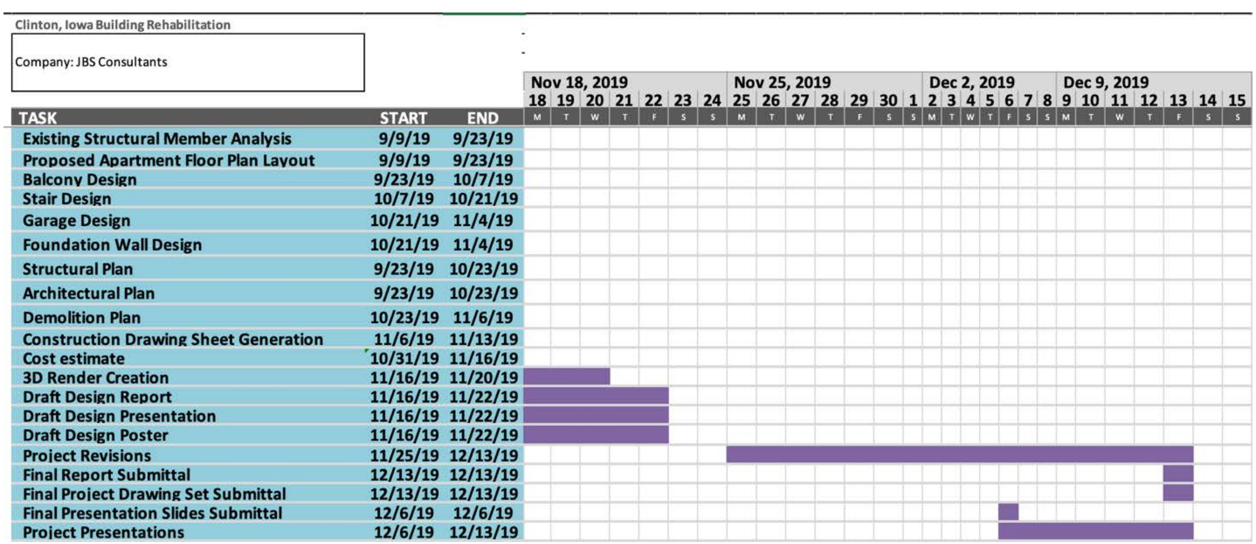

Figure 3: Displays Gantt Chart for Work Completed from November 18th, 2019 – December 7th, 2019

IV. CONSTRAINTS, CHALLENGES AND IMPACTS

Constraints

One of the major constraints posed by this project was time. The size of these buildings and the amount of structural and architectural design work required to ensure their adequate stability and habitation was quite challenging to complete within five months. Due to the time constraint, many engineering assumptions had to be made to complete the major tasks of the project. Prioritizing was key to keep the work flowing; for example, design calculations had to come before installation of member sizes into the Revit model, which then came before estimating the price of materials. Thus, clear communication between team members was essential to complete the design tasks in this project.

Another constraint the team faced was the fact that these are existing buildings. There were no previous drawings of the layout of the buildings, so measurements of the interiors had to be conducted by the design team during site visits. The measuring tools used are accurate, but a certain margin of error must be assumed for the existing layout. Another issue this constraint posed is that the façade and second floor members are obscured by cover materials. Since these aspects of the existing architecture could not be observed without demolition, contingencies had to be included in the cost estimate for the façade restoration and floor joists designed for the second-floor system.

Although JBS Consultants was not given a budget for this project, it was made clear by the client to provide designs that were cost effective. This constrained the team from making major structural changes to the buildings, like raising the floor and roof system above the bar as previously mentioned. Project costs also drove decision making for design alternatives for the layout of the bar. Garage parking for residential use was determined to be the most profitable option for the building owner, since a back patio would not significantly increase the value of the commercial space.

Challenges

Difficulties that stood out in this project mostly arose from the existing characteristics of the buildings. The spacing between bearing walls was measured as 25 feet, which required larger than typical joist sizes to satisfy design requirements. The apartments above the insurance office and bar use the same hallway for access, which is problematic because they are at two different elevations. To resolve this issue, it was deemed necessary to raise the hallway and add a short staircase to bring the hallway to the higher floor level. This allowed the team to utilize the existing back stairway as an alternative entrance for both apartment units. This is one of multiple issues posed by the existing buildings, as they are not entirely uniform.

Another challenge was the fact that many of the structural systems were obscured by existing floor covers, ceiling materials, and wall finish. The basements were especially tricky, as the geometry of the walls made it difficult to accurately measure and place the foundations. The second-floor system was entirely obscured by the ceiling panels on the ground level and floor cover on the second. This meant the second-floor framing members had to be designed.

One major dilemma that had to be considered was determining mechanical properties for existing materials. Wood type was assumed to be spruce-pine-fir No. 2, as it is a cheap and widely used lumber with moderate strength. This was a conservative assumption so that actual loads would not exceed design strength. The bearing walls were composed of structural clay masonry, which historically has been widely used throughout the United States. The compressive strength of the walls was determined from research on historic clay brick conducted by Witzany, SyKora, and Holicky and published by *The Journal of Civil Engineering and Management*; this value was also taken as the lowest reported strength to remain conservative.

From an environmental standpoint, it is imperative that all working activities are protected from potential hazards. More specifically, there must be precautions taken when working on the exterior of the building and in the basements. The basements must be accounted for because of the large amounts of mold and asbestos. Since these buildings are occupied by businesses, it must be ensured that none of the materials contaminate the commercial spaces while being removed. The next environmental impact that must be accounted for is the dust that may come about when working on the exterior of the buildings. The team's designs call for the rehabilitation of the facades of these buildings. The work that is to be completed will have the potential to create dust that will spread into the air. For JBS Consultants' work strategies, the team must ensure that all designs will allow the potential contractor to place enough protections to prevent the transmission of hazardous particulates.

Societal Impacts

An important impact this project has is the addition of new housing for the residents of Clinton. With the four new units that have been added to these buildings, the previously unused space will help draw young professionals to the downtown area. This is important for a city that has experienced a slight population decline over the past 10 years. One of the client goals is to attract more young people to the Clinton area. With the mean age being above the national average, it is important to the social health of the city to attract a younger demographic.

Another impact brought by this project is aiding the restoration of downtown Clinton. As a city with over 160 years of history, it is important to preserve the historic aesthetic of the buildings in the downtown area. The brick facades are a staple of Iowa architecture, and represent over 100 years of industry. Upholding that cultural heritage and historic charm plays a substantial role in attracting more people to the city.

As these buildings currently provide commercial space to tenants, it is necessary to consider the impact changes will have on current and future businesses. The biggest changes to the first-floor space comes with the bar, which is being reduced in total square footage to allow the installation of a garage for residential use. The team considers the change in space a positive, as it coincides with better use of the front area which is currently separated from the rest of the bar. This will make for a more inviting social atmosphere, as the street side entrance will be favored over the alley entrance in the back.

The economic impact these renovations may bring was also considered. Designs for the first-floor plan were left open to allow for other prospective businesses to be leased from the commercial space and contribute to the Clinton economy. The second-floor residential space could attract professionals who will fill important jobs in the area. Filling these residential spaces will also provide income for the building owner, who may use that revenue to further invest in the city. This renovation project could provide many new businesses and living opportunities to downtown Clinton and serves as a testament to the growth the Downtown Clinton Alliance strives for.

V. ALTERNATIVE SOLUTION CONSIDERATIONS

There were multiple alternatives that were presented to the client. Walkout balconies for the apartments, residential garages, a patio for the bar, and a raised floor system were among the proposed changes. Other rehabilitation work includes the restoration of the historic façade, cleanup of the basements, and rework of the floor plans. After considering the square footage of the second floors, various layouts for one- and two- bedroom apartments were proposed. Three floor plans were discussed: first, six one-bedroom units, two in each building; second, three two-bedroom units, one in each building, and third, a combination of the two, with two two-bedroom units above the collector's shop and bar and two one-bedroom units above the insurance office. The final floor layout was determined after discussing Clinton residential demographics with the client.

One major consideration that was decided against was raising the floor system of the apartment above the bar. The reasoning for this was to make the floor of the connected apartments above the bar and insurance office level and allow for balconies to be installed for the unit above the bar. However, this was decided against because of the significant cost associated with raising the floors and possibly even the roof system. As a compromise, the hallway that connects these two apartment spaces was raised to allow for the front and back stairwells to be utilized by tenants of both units. Another design decision was the choice of the use of the south portion of the bar. The team chose between a patio and a residential garage; after discussing the alternatives with the building owner, the garage was chosen because it made the second-floor apartments more attractive to residents. Although the patio would provide a lively addition to the bar, the garage presented the opportunity to increase rental income.

Aside from the patio and raised floor system, all design considerations have been carried out by the JBS team. Apartment floor plans have been detailed, and an additional exit stairwell for the apartment above the collector's shop has been designed to comply with code egress requirements. Structural systems for the balconies, stairs, and retaining walls, as well as existing floors and bearing walls have been analyzed and designed for. It was decided early on that most of the existing structural systems are in good condition. The limestone foundation was deemed structurally sound, but tuckpointing and grouting is required to maintain stability. Only those structures that are being changed for design purposes were analyzed; this means the roof and foundation were not analyzed and presumed adequate for serviceability.

VI. FINAL DESIGN DETAILS

It should be noted that most of this structure, existing and new additions, consists of wood dimensional lumber. An assumption was made before any designs or analyses were conducted that the wood is spruce-pine-fir (SPF) No. 2, as it is a widely available type of wood that has relatively low strength. This was a conservative assumption, so that if the existing lumber species is different from SPF, it would most likely have stronger characteristics. The Allowable Strength Design (ASD) method was used to size each member, and standard sizes and strength properties were taken from the National Design Specification (NDS) Design Values for Wood Construction Manual and Supplement. All members were analyzed to resist bending moment, shear force, and deflection serviceability requirements using standard design loads.

Load Calculations

Standard weights of architectural materials such as floor covers, ceiling fixtures, lighting, and insulation were determined using the Boise Cascade weights of building materials. Dead load and live loads were determined in pounds per square foot (PSF), then converted to pounds per linear foot to analyze structural members as two-dimensional simply supported beams. Live loads were determined from ASCE 7-16 Chapter 4 Live Loads. Residential live loads were used for both the floor and stair systems. The balconies were designed to sustain uniform and unbalanced snow loads, whose calculations are detailed in Appendix A Section I of this report. The unit weight of snow and maximum average ground snow load for the Clinton area were taken from ASCE 7-16 Chapter 7 Snow Loads.

Second Floor Joists Analysis

The second-floor system was assumed to be consistent through all three buildings, so that one set of calculations would suffice for strength analysis. As the existing floor framing was obscured by the floor and ceiling cover, the existing floor joists were designed using a 25-foot span and loads calculated as described above. The joists were determined to be 3x12 with 12 inches on-center (O.C.) spacing. Supporting calculations for the second-floor framing can be found in Appendix A Section II.

First Floor Joists Analysis

The first-floor system was dimensioned during the team's first site visit. The existing framing for the first-floor was determined to be 2x12 @ 16" O.C. The span was significantly less than the second floor, because the joists are supported by a continuous beam with column reinforcements going into the foundation. The span of these joists was measured as 12 feet; supporting calculations are provided in Appendix A Section III.

First Floor Beam Design

The existing beam that supports the first-floor joists was measured and analyzed to resist the design loads applied by current and proposed additions. This beam was measured as 8x10 dimensional lumber with a maximum unbraced length of 9.5 feet. It was modeled as a continuous beam to reduce the applied negative bending moment, which governed the design. Supporting calculations for this analysis can be found in Appendix A Section IV.

Bearing Wall Analysis

The bearing walls of the buildings were assumed to be 12-inch clay wythe made of historic brick. Strength properties for the clay masonry were determined from research conducted by Witzany et al. and published by *The Journal of Civil Engineering and Management*; the lowest reported compressive strength of 2466 psi was used to remain conservative with the analysis. Even with this characteristic, the compressive strength greatly exceeded the maximum applied load. Further details on this conclusion can be found in Appendix A Section V.

Design of Stair System

A stair system was designed in order to allow for entrance and exit to the units above the collector’s shop. The stair system consisted of three subsystems that needed to be designed for: the stringers, landing joists, and landing studs. The stairs were designed with a width of four feet, which exceeds the minimum stairway width of three feet according to the IBC. This is to allow for the potential installation of an ADA accessible chairlift. The stringers were designed as sawtooth 3x14 dimensional lumber, with an effective depth of 7.25 inches. The design length of the stringers was 16.25 feet, which was the longest spanning staircase. Design calculations for the stringers can be found in Appendix A Section VI-i. The landing joists were designed to be 2x4 @ 16” O.C. and spanned the three-foot width of the landing. Design calculations for the landing joists can be found in Appendix A Section VI-ii. The landing studs were designed to be 2x4 @ 16” O.C. and support the joists. Design calculations for the landing studs can be found in Appendix A Section VI-iii. The detail for this stair system can be seen in Figure 4.

Two shorter spanning stair and raised floor systems were added. One was added to the shared hallway between the apartments above the insurance office and bar to compromise the change in second-floor elevation. The other was added to the southern unit above the insurance office to provide access to the balcony. These stairs and raised hallway were designed with the same member sizes as the stair system and landing to the apartment above the collector’s shop.

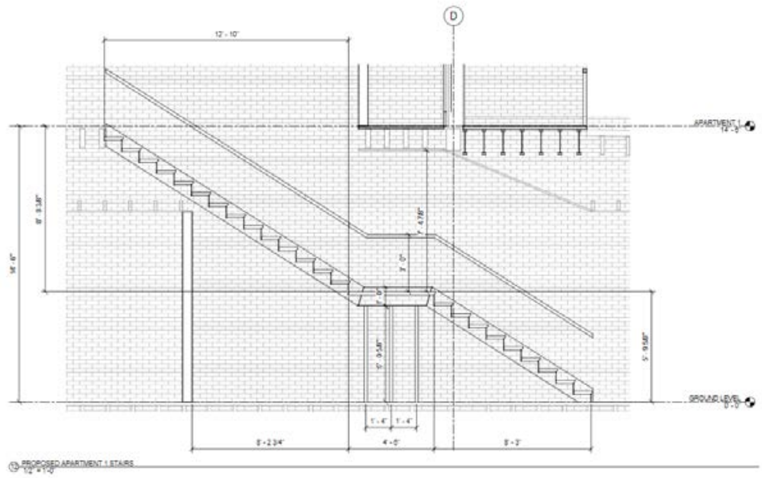

Figure 4: Proposed Stair Detail for Collector's Shop

Design of Balcony System

The balconies consist of two systems: the joists and a built-up beam. Properties of the TJI engineered wood I joists were taken from the Weyerhaeuser design catalogue. The balcony joists span 25 feet from bearing wall to bearing wall and are separated at 12" O.C. extending six feet from the end of the second floor. The specific name of the joists is TJI 360 with 16-inch depth. Design calculations for the balcony joists can be found in Appendix A Section VII-i. The back-exterior brick wall of the apartments was also accounted for in the balcony system by supporting it with a built-up beam. The beam was designed to span 25 feet, much like the joists, with the added dead load of the brick wall. It was designed to be (3) 3x14 timbers. Supporting calculations for this built-up beam section can be found in Appendix A Section VII-ii. The balcony details can be seen in Figure 5.

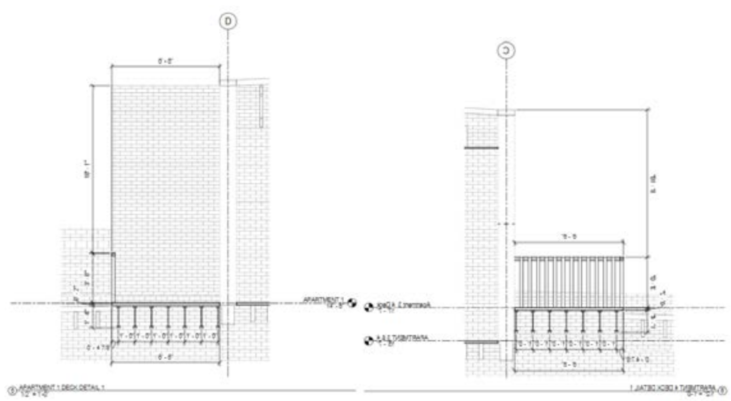

Figure 5: Proposed Balcony Details

Garage Design

The garage design is composed of two new additions: a header to support the garage door and a retaining wall to support the slab-on-grade concrete pad for vehicle parking. The header was designed to be 2.0E grade LVL whose properties were taken from the Weyerhaeuser design catalogue. The header has a depth of 9.25 inches and breadth of 3.5 inches and spans 19 feet, which is the width of the opening to the garage. Design calculations for the header can be found in Appendix A Section VIII-i. The slab-on-grade is composed of a 4-inch reinforced concrete pad with 6x6-W2.9xW2.9 steel wire mesh, 2 mm vapor barrier, and 4-inch crushed stone layer with 250 cubic yards of granular backfill subgrade. The mechanically stabilized retaining wall was designed as a precast concrete gravity wall with galvanized steel strip reinforcement to support the backfill. Detailed calculations for the design of the gravity wall and reinforcing strips can be found in Appendix A Section VIII-ii. The detail for the garage header can be seen in Figure 6.

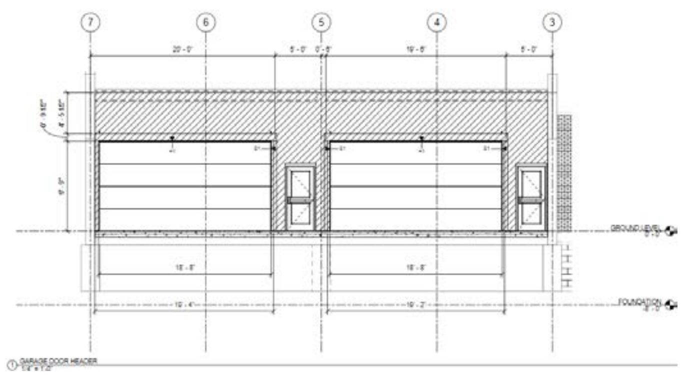

Figure 6: Proposed Garage Header Detail

Apartment Layout

The second floor has received a complete alteration to the floor layout. All the existing partition walls in the space above the collector’s shop have been removed and replaced to create a two-bedroom, two-bathroom apartment. Part of the existing second floor system has been demolished and replaced to make room for a new stair system that provides a second south-facing entrance. A balcony has been installed in the south end of the apartment, which brings the total area to 1410 square feet. The space above the bar has also been partitioned into a two-bedroom, two-bathroom apartment, but due to constraints in the floor elevation will not receive a balcony. This unit will have a total area of 1190 square feet. The space above the insurance office has been separated into two, one-bedroom one-bathroom apartments. The southern apartment will receive a balcony and short stairway to compromise the change in elevation between the balcony and existing apartment floor. With the addition of a balcony, this brings the total area of the southern unit to 912 square feet, which is equal to the northern single bedroom apartment. The layout for this floor plan can be seen below in Figure 7.

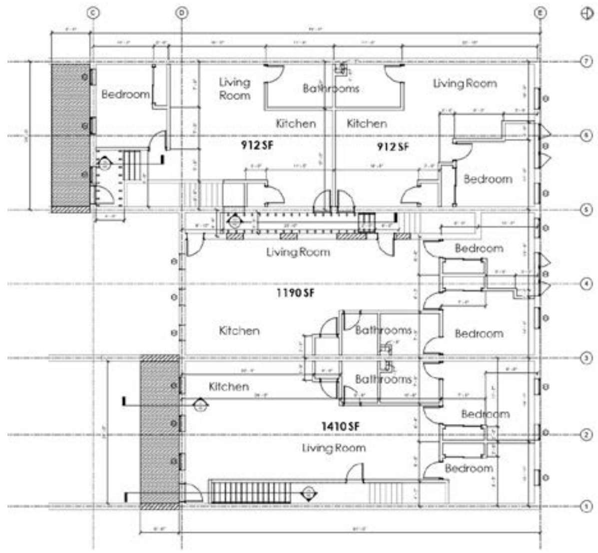

Figure 7: Proposed Apartment Layout with Balconies

VII. COST ESTIMATE

All cost estimate data was obtained using the 2018 RSMeans Data and Craftsman's 2019 National Construction Estimator by Richard Pray. The following spreadsheet shows our full estimate for the designed project. The total estimated cost includes the construction cost, any overhead costs, the general contractors projected profit markup and any project contingencies. The total cost of the project is estimated to be approximately $561,000.00.

239 5th Ave. S., Clinton, IA 52732 Building Rehabilitation Cost Estimate		
Earthwork Total Estimate		$2,500.00
Existing Conditions Total Estimate		$26,000.00
Concrete Total Estimate		$60,000.00
Masonry Total Estimate		$40,200.00
Wood and Composites Total Estimate		$53,000.00
Thermal and Moisture Protection Total Estimate		$3,075.00
Openings Total Estimate		$72,500.00
Plumbing Total Estimate		$50,000.00
HVAC Total Estimate		$43,000.00
Electrical Total Estimate		$73,000.00
Indirect Overhead Estimate		$32,000.00
Direct Overhead Estimate		$28,000.00
Building Inspection Total Estimate		$3,000.00
Total Construction Cost	$426,500.00	
Overhead (53% Indirect; 47% Direct)	$60,000.00	
Profit (Rate is 7.5%)	$32,000.00	
Contingencies (10%)	$42,650.00	
Total Estimate	$561,000.00	

Figure 8: Summarized Cost Estimate

Prices from . . .	Rounded to the nearest . . .
$.01 to $5.00	$.01
$5.01 to $20.00	$.05
$20.01 to $100.00	$.50
$100.01 to $300.00	$1.00
$300.01 to $1,000.00	$5.00
$1,000.01 to $10,000.00	$25.00
$10,000.01 to $50,000.00	$100.00
$50,000.01 and above	$500.00

Figure 9: RS Means Rounding Criteria for Cost Estimates

VIII. APPENDIX A: DESIGN CALCULATIONS

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I. Load Calculations

Assumptions

-floor sheathing and underlayment is 1" thick

-ASCE 7-16 standard live loads are used

  -residential live load is 40psf

  -roof live load is 20psf

Define Variables

residential floor live load	$LL_{floors} := 40\ psf$	unit weight of water	$\gamma_w := 62.4\ \frac{lbf}{ft^3}$
roof live load	$LL_{roof} := 20\ psf$	gross area of 3x12 solid sawn lumber	$A_j := 28.13\ in^2$
stair live load	$LL_{stairs} := 100\ psf$		
garage live load	$LL_g := 40\ psf$		
ground snow load	$p_g := 25\ psf$		
		tributary width of joists	$t_b := 12\ in$
specific gravity of spruce-pine-fir	$SG := 0.42$		
		length of longest joist	$l_j := 25\ ft$
live load element factor (for interior beams)	$K_{LL} := 2$		

Design Calculations

determine floor dead load using dead load calculator

Floor Dead Load Components		
Floor Finish Materials	weight (psf)	(psf)
1. Lightweight	110	(pcf)
Concrete Thickness	0	(in)
5. Wood panel, underlayment, 1/4 in	0.8	(psf)
4. Plywood/OSB, 5/8 in	2	(psf)
Floor Framing	0	(psf)
7. Loose insulation, fiberglass	0.04	(psf/in)
Ceiling Insulation Thickness	0	(in)
8. Suspended steel channel system wi	3	(psf)
Lighting - Lights and conduit	1	(psf)
Mechanical - Duct allowance	4	(psf)
Plumbing - Piping allowance	1	(psf)
Determine total dead load	11.80	(psf)

dead load of floor cover materials $DL_{cover} := 11.8\ psf$

determine dead load due to self-weight of joists

$$DL_{joists} := A_j \cdot SG \cdot \gamma_w = 5.12\ \frac{lbf}{ft}$$

determine total dead load of floor system

$$DL_{floors} := DL_{cover} + \frac{DL_{joists}}{t_b} = 16.92\ psf$$

use live load reduction for joist design

calculate tributary area of longest joist $A_T := t_b \cdot l_j = 25\ ft^2$

use unitless values in the reduction equation $A_T := 24$ $K_{LL} \cdot A_T = 48$

determine reduced live load

$$LL_{reduced} := LL_{floors} \cdot \left(0.25 + \left(\frac{15}{\sqrt[2]{K_{LL} \cdot A_T}}\right)\right) = 96.603\ psf$$

*tributary area At is too small to use live load reduction

Design Summary

Dead and live loads were determined using typical material weights and ASCE 7-16, respectively. They are as follows:

dead load: $DL_{floors} = 16.92\ psf$

live load: $LL_{floors} = 40\ psf$

II. 2nd Floor Joist Analysis

<u>Assumptions</u>
-all wood members are spruce-pine-fir (SPF) No. 2
-all floor joists are 3x12 @12" OC
-lateral support is provided to prevent LTB
-joists rest on brick bearing wall

<u>Define Variables</u>

reference design values

specific gravity of SPF	$SG := 0.42$
modulus of elasticity	$E := 1400000\ psi$
	$E_{min} := 510000\ psi$
bending strength	$F_b := 875\ psi$
tension parallel to grain	$F_t := 450\ psi$
shear parallel to grain	$F_v := 135\ psi$
compression perpendicular to grain	$F_{c\_p} := 425\ psi$
compression parallel to grain	$F_c := 1150\ psi$

section properties

length	$l := 25\ ft$
depth	$d := 11.25\ in$
breadth	$b := 2.5\ in$
tributary width	$t_b = 12\ in$
section modulus	$S := 52.73\ in^3$
moment of inertia	$I := 296.6\ in^4$

Design Calculations

determine applicable adjustment factors

load duration factor $C_D := 1.0$

wet service factor $C_M := 1.0$

temperature factor $C_t := 1.0$

size factor...

...for bending $C_{F\_b} := 1.0$ ...for tension $C_{F\_t} := 1.0$ ...for compression (parallel to grain) $C_{F\_c} := 1.0$

flat use factor $C_{fu} := 1.0$

incising factor $C_i := 1.0$

repetitive member factor $C_r := 1.15$

column stability factor $C_p := 1.0$

buckling stiffness factor $C_T := 1.0$

bearing area factor $C_b := 1.0$

beam stability factor (CL)

determine effective length of joist $\frac{l}{d} = 26.667$ $\frac{l}{d} \geq 7 = 1$

$l_e := 1.63 \cdot l + 3 \cdot d = 43.563\ ft$

calculate slenderness ratio

$R_B := \sqrt[2]{\frac{l_e \cdot d}{b^2}} = 30.675$ $R_B < 50 = 1$

calculate reference bending design value

$F_{b\#} := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_{F\_b} \cdot C_i \cdot C_r$

$E_{min}' := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T = \left(5.1 \cdot 10^5\right)\ psi$

$F_{bE} := \frac{1.2 \cdot E_{min}'}{{R_B}^2} = \left(9.366 \cdot 10^4\right)\ psf$

calculate beam stability factor

$$C_L := \frac{1+\left(\frac{F_{bE}}{F_{b\#}}\right)}{1.9} - \sqrt[2]{\left(\frac{1+\left(\frac{F_{bE}}{F_{b\#}}\right)}{1.9}\right)^2 - \frac{\left(\frac{F_{bE}}{F_{b\#}}\right)}{0.95}} = 0.601$$

*assume joists have adequate lateral bracing to prevent LTB: $C_L := 1.0$

convert loads into linearly distributed loads

calculate dead load (plf) $w_{DL} := DL_{floors} \cdot t_b = 16.92\ \frac{lbf}{ft}$

calculate live load (plf) $w_{LL} := LL_{floors} \cdot t_b = 40\ \frac{lbf}{ft}$

Bending Moment Design

bending strength of No. 2 SPF $F_b = 875\ psi$

calculate adjusted bending strength $F_b' := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_{F\_b} \cdot C_{fu} \cdot C_i \cdot C_r$

determine applied loads using load analysis program (ftool)

$w_{DL} = 16.92\ \frac{lbf}{ft}$ $w_{LL} = 40\ \frac{lbf}{ft}$ $w_{floors} := w_{DL} + w_{LL} = 56.92\ \frac{lbf}{ft}$

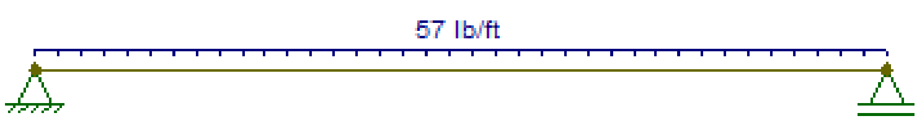


Figure 1.1 Distributed loading over joist

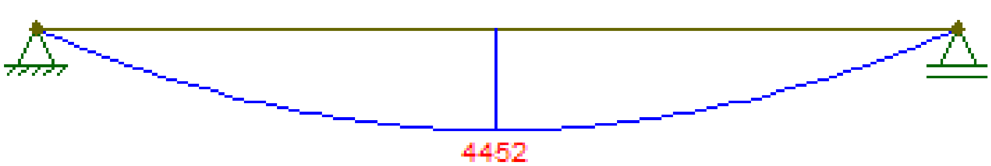


Figure 1.2 Bending moment diagram for joist

max moment in joist $M := 4452\ lbf \cdot ft$

determine applied bending stress using section modulus $f_b := \frac{M}{S} = \left(1.013 \cdot 10^3\right)\ psi$

compare applied stress to joist strength $F_b' = \left(1.006 \cdot 10^3\right)\ psi$

$$\frac{f_b}{F_b'} = 1.007$$

Shear Design

estimate Fv' using applicable adjustment factors

$$F_v' := F_v \cdot C_D \cdot C_M \cdot C_t \cdot C_i = \left(1.944 \cdot 10^4\right)\ psf$$

determine applied shear stress fv

$w_{floors} = 56.92\ plf$ $\qquad l = 25\ ft$

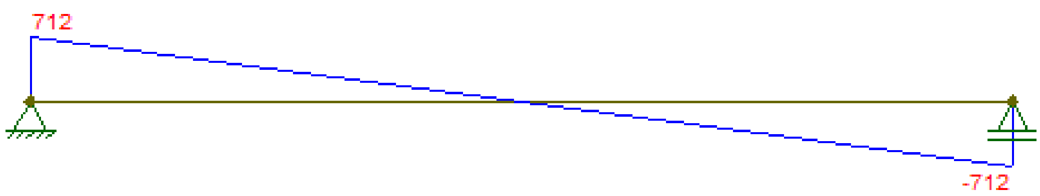


Figure 1.3 Shear force diagram for 25' joists

$V := 712\ lbf$

determine applied shear stress parallel to grain

$$f_v := \frac{3 \cdot V}{2 \cdot b \cdot d} = \left(5.468 \cdot 10^3\right)\ psf$$

Design Equation: $f_v \le F_v' = 1$

DCR: $\frac{f_v}{F_v'} = 0.281$ joists satisfy shear design

Deflection Design

determine appropriate short and long term loads

*use full live load for short term deflection

$$w_{st} := (LL_{floors}) \cdot t_b = 40\ plf$$

*neglect dead load for long-term deflection due to age of floor system

$$w_{lt} := (0.5 \cdot LL_{floors})\ t_b$$

apply load combination for total deflection

$$w_{tot} := 1.5 \cdot w_{lt} + 0.5 \cdot w_{st} = 50\ plf$$

check short-term deflection...

...using equations

$$\delta_{st} := \frac{5 \cdot w_{st} \cdot l^4}{384 \cdot E \cdot I} = 0.847\ in \qquad \Delta_{st} := \frac{l}{360} = 0.833\ in$$

...using load analysis program (ftool)

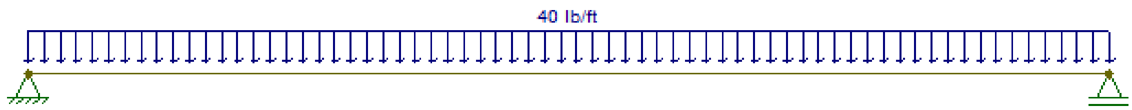


Figure 1.4 Distributed load for short-term deflection

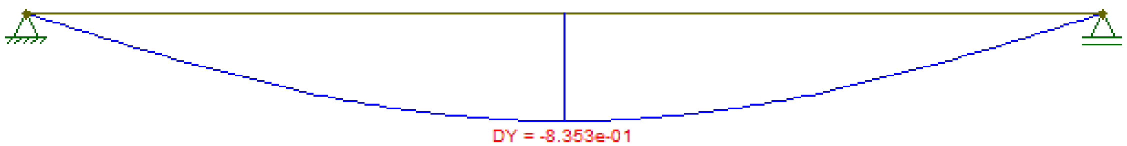


Figure 1.5 Diagram showing short-term deflection

short term deflection results from load analysis

$$\delta_{st} := 0.8353\ in$$

check total deflection

...using equations

$\delta_{lt} := \frac{5 \cdot w_{lt} \cdot l^4}{384 \cdot E \cdot I} = 0.423\ in$ $\delta_{tot} := 1.5 \cdot \delta_{lt} + \delta_{st} = 1.47\ in$ $\Delta_{tot} := \frac{l}{240} = 1.25\ in$

...using load analysis program (ftool)

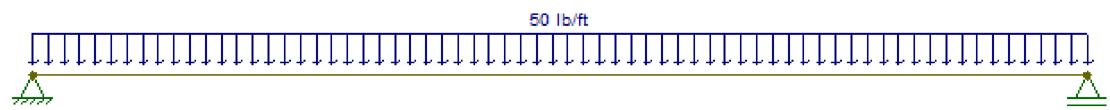


Figure 1.6 Distributed load for total deflection

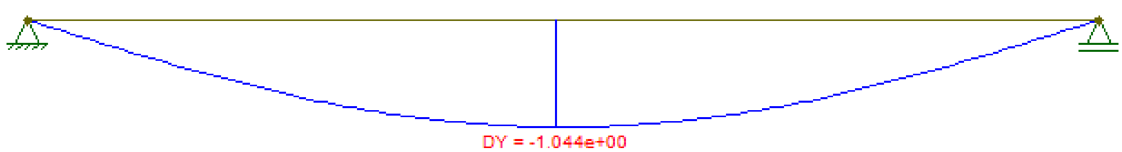


Figure 1.7 Diagram showing total deflection

$\delta_{tot} := 1.044\ in$

Design Equation: $\Delta_{tot} \geq \delta_{tot} = 1$ deflection requirements check out

DCR: $\frac{\delta_{tot}}{\Delta_{tot}} = 0.835$

Bearing Design

bearing length parallel to grain $l_b := 4\ in$

bearing breadth perpendicular to grain $b_b := b = 2.5\ in$

define bearing area $A_b := l_b \cdot b_b = 10\ in^2$

define bearing area factor $C_b := \frac{l_b + 0.375\ in}{l_b} = 1.094$

determine reaction force at column P

length of longest span beam $l=25\ ft$

$P:=0.5 \cdot l \cdot w_{floors}=711.496\ lbf$

$f_{c\_p}:=\frac{P}{A_b}=\left(1.025 \cdot 10^4\right)\ psf$

apply appropriate adjustment factors to design values

$F_{c\_p}=425\ psi$

$F_{c\_p}':=F_{c\_p} \cdot C_M \cdot C_t \cdot C_i \cdot C_b=\left(6.694 \cdot 10^4\right)\ psf$

ensure bearing is satisfied

Design Equation: $F_{c\_p}' \ge f_{c\_p}=1$

DCR: $\frac{f_{c\_p}}{F_{c\_p}'}=0.153$

<u>Design Summary</u>

Joists were designed to resist moment, shear, and deflection; the corresponding design equations are shown below:

moment $F_b'=\left(1.449 \cdot 10^5\right)\ psf$ $f_b=\left(1.459 \cdot 10^5\right)\ psf$

$DCR:=\frac{f_b}{F_b'}=1.007$

shear $F_v'=\left(1.944 \cdot 10^4\right)\ psf$ $f_v=\left(5.468 \cdot 10^3\right)\ psf$

$DCR:=\frac{f_v}{F_v'}=0.281$

deflection $\Delta_{st} = 0.833\ in$ $\delta_{st} = 0.835\ in$

$$DCR := \frac{\delta_{st}}{\Delta_{st}} = 1.002$$

$\Delta_{tot} = 1.25\ in$ $\delta_{tot} = 1.044\ in$

$$DCR := \frac{\delta_{tot}}{\Delta_{tot}} = 0.835$$

bearing $F_{c\_p}' = \left(6.694 \cdot 10^{4}\right)\ psf$ $f_{c\_p} = \left(1.025 \cdot 10^{4}\right)\ psf$

$$DCR := \frac{f_{c\_p}}{F_{c\_p}'} = 0.153$$

III. 1st Floor Joist Analysis

<u>Assumptions</u>
-all wood members are spruce-pine-fir (SPF) No. 2
-all floor joists are 2x12 @16" OC
-lateral support is provided

<u>Define Variables</u>

reference design values

specific gravity of SPF	$SG := 0.42$
modulus of elasticity	$E := 1400000\ psi$
	$E_{min} := 510000\ psi$
bending strength	$F_b := 875\ psi$
tension parallel to grain	$F_t := 450\ psi$
shear parallel to grain	$F_v := 135\ psi$
compression perpendicular to grain	$F_{c\_p} := 425\ psi$
compression parallel to grain	$F_c := 1150\ psi$

section properties

length	$l := 12\ ft$
depth	$d := 11.25\ in$
breadth	$b := 1.5\ in$
tributary width	$t_b := 16\ in$
section modulus	$S := 31.64\ in^3$
moment of inertia	$I := 178\ in^4$

Design Calculations

Adjustment Factors

load duration factor $C_D := 1.0$

wet service factor $C_M := 1.0$

temperature factor $C_t := 1.0$

size factor...

...for bending $C_{F\_b} := 1.0$ ...for tension $C_{F\_t} := 1.0$ ...for compression (parallel to grain) $C_{F\_c} := 1.0$

flat use factor $C_{fu} := 1.0$

incising factor $C_i := 1.0$

repetitive member factor $C_r := 1.15$

column stability factor $C_p := 1.0$

buckling stiffness factor $C_T := 1.0$

bearing area factor $C_b := 1.0$

beam stability factor (CL)

determine effective length of joist $\frac{l}{d} = 12.8$ $\frac{l}{d} \geq 7 = 1$

$l_e := 1.63 \cdot l + 3 \cdot d = 22.373\ ft$

calculate slenderness ratio

$R_B := \sqrt[2]{\frac{l_e \cdot d}{b^2}} = 36.638$ $R_B < 50 = 1$

calculate reference bending design value

$F_{b\#} := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_{F\_b} \cdot C_i \cdot C_r$ $E_{min}' := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T = \left(5.1 \cdot 10^5\right)\ psi$

$F_{bE} := \frac{1.2 \cdot E_{min}'}{{R_B}^2} = \left(6.565 \cdot 10^4\right)\ psf$

calculate beam stability factor

$$C_L := \frac{1+\left(\frac{F_{bE}}{F_{b\#}}\right)}{1.9} - \sqrt[2]{\left(\frac{1+\left(\frac{F_{bE}}{F_{b\#}}\right)}{1.9}\right)^2 - \frac{\left(\frac{F_{bE}}{F_{b\#}}\right)}{0.95}} = 0.436$$

*assume joists have adequate lateral bracing to prevent LTB: $C_L := 1.0$

convert loads into linear distributed loads

calculate dead load (plf) $w_{DL} := DL_{floors} \cdot t_b = 22.56 \ \frac{lbf}{ft}$

calculate live load (plf) $w_{LL} := LL_{floors} \cdot t_b = 53.333 \ \frac{lbf}{ft}$

Bending Moment Design

bending strength of No. 2 SPF $F_b = 875 \ psi$

calculate adjusted bending strength $F_b' := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_{F\_b} \cdot C_{fu} \cdot C_i \cdot C_r$

determine applied loads using load analysis program (ftool)

$w_{DL} = 22.56 \ \frac{lbf}{ft}$ $w_{LL} = 53.333 \ \frac{lbf}{ft}$

$w_{floors} := w_{DL} + w_{LL} = 75.893 \ \frac{lbf}{ft}$

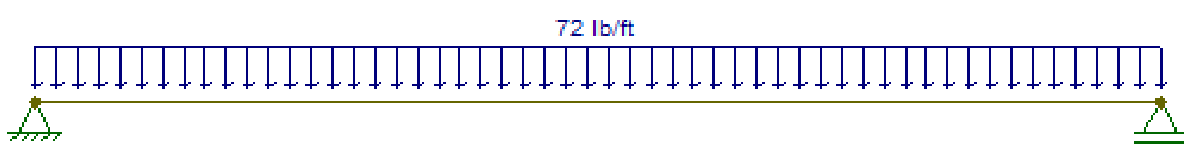


Figure 2.1 Distributed loading over joist

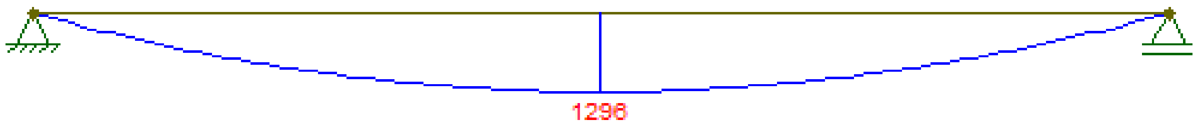


Figure 2.2 Bending moment diagram for joist

max moment in joist $M := 1296\ lbf \cdot ft$

determine applied bending stress using section modulus $f_b := \frac{M}{S} = 491.53\ psi$

compare applied stress to joist strength $F_b' = \left(1.006 \cdot 10^3\right)\ psi$

$$\frac{f_b}{F_b'} = 0.488$$

Shear Design

estimate Fv' using applicable adjustment factors

$$F_v' := F_v \cdot C_D \cdot C_M \cdot C_t \cdot C_i = \left(1.944 \cdot 10^4\right)\ psf$$

determine applied shear stress fv

$w_{floors} = 75.893\ plf$ $l = 12\ ft$

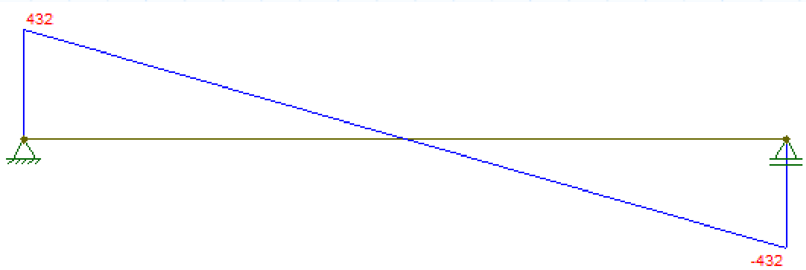


Figure 2.3 Shear force diagram for 25' joists

$$V := 432\ lbf$$

determine applied shear stress parallel to grain

$$f_v := \frac{3 \cdot V}{2 \cdot b \cdot d} = \left(5.53 \cdot 10^3\right)\ psf$$

Design Equation: $f_v \le F_v{}' = 1$

DCR: $\dfrac{f_v}{F_v{}'} = 0.284$ joists satisfy shear design

Deflection Design

determine appropriate short and long term loads

short term $w_{st} := (0.5 \cdot LL_{floors}) \cdot t_b = 26.667\ plf$

long term $w_{lt} := (DL_{floors} + 0.5 \cdot LL_{floors})\ t_b = 49.226\ plf$

apply load combination for total deflection

$w_{tot} := 1.5 \cdot w_{lt} + 0.5 \cdot w_{st} = 87.173\ plf$

check short-term deflection...

...using equations

$\delta_{st} := \dfrac{5 \cdot w_{st} \cdot l^4}{384 \cdot E \cdot I} = 0.05\ in$ $\Delta_{st} := \dfrac{l}{360} = 0.4\ in$

...using load analysis program (ftool)

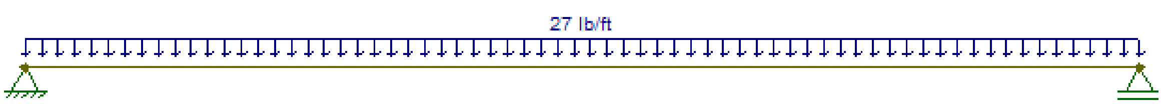


Figure 2.4 Distributed load for short-term deflection

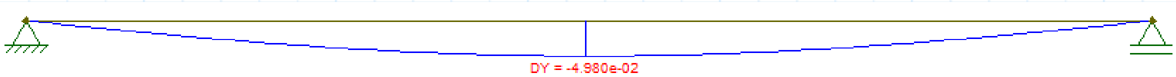


Figure 2.5 Diagram showing short-term deflection

short term deflection results from load analysis $\delta_{st} := 0.0498\ in$

check total deflection

...using equations

$\delta_{lt} := \frac{5 \cdot w_{lt} \cdot l^4}{384 \cdot E \cdot I} = 0.092\ in$ $\delta_{tot} := 1.5 \cdot \delta_{lt} + \delta_{st} = 0.188\ in$ $\Delta_{tot} := \frac{l}{240} = 0.6\ in$

...using load analysis program (ftool)

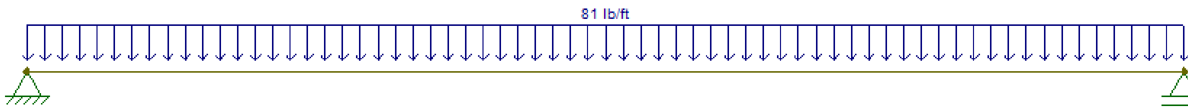


Figure 2.6 Distributed load for total deflection

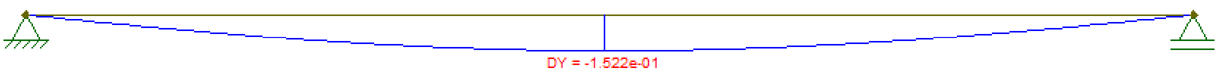


Figure 2.7 Diagram showing total deflection

$\delta_{tot} := 0.1522\ in$

Design Equation: $\Delta_{tot} \geq \delta_{tot} = 1$ deflection requirements check out

DCR: $\frac{\delta_{tot}}{\Delta_{tot}} = 0.254$

Bearing Design

bearing length parallel to grain $l_b := 4\ in$

bearing breadth perpendicular to grain $b_b := b = 1.5\ in$

define bearing area $A_b := l_b \cdot b_b = 6\ in^2$

define bearing area factor $C_b := \frac{l_b + 0.375\ in}{l_b}$

determine reaction force at column P

length of longest span beam $l = 12\ ft$

$P := 0.5 \cdot l \cdot w_{floors} = 455.357\ lbf$

$f_{c\_p} := \frac{P}{A_b} = \left(1.093 \cdot 10^4\right)\ psf$

apply appropriate adjustment factors to design values

$F_{c\_p} = 425\ psi$

$F_{c\_p}' := F_{c\_p} \cdot C_M \cdot C_t \cdot C_i \cdot C_b = \left(6.694 \cdot 10^4\right)\ psf$

ensure bearing is satisfied

Design Equation: $F_{c\_p}' \geq f_{c\_p} = 1$

DCR: $\frac{f_{c\_p}}{F_{c\_p}'} = 0.163$

<u>Design Summary</u>

Joists were designed to resist moment, shear, and deflection; the corresponding design equations are shown below:

moment $F_b' = \left(1.449 \cdot 10^5\right)\ psf$ $f_b = \left(7.078 \cdot 10^4\right)\ psf$

$DCR := \frac{f_b}{F_b'} = 0.488$

shear $F_v' = \left(1.944 \cdot 10^4\right)\ psf$ $f_v = \left(5.53 \cdot 10^3\right)\ psf$

$DCR := \frac{f_v}{F_v'} = 0.284$

deflection $\Delta_{st} = 0.4\ in$ $\delta_{st} = 0.05\ in$

$DCR := \frac{\delta_{st}}{\Delta_{st}} = 0.125$

$\Delta_{tot} = 0.6\ in$ $\delta_{tot} = 0.152\ in$

$$DCR := \frac{\delta_{tot}}{\Delta_{tot}} = 0.254$$

bearing $F_{c\_p}' = \left(6.694 \cdot 10^4\right)\ psf$ $f_{c\_p} = \left(1.093 \cdot 10^4\right)\ psf$

$$DCR := \frac{f_{c\_p}}{F_{c\_p}'} = 0.163$$

IV. First Floor Beam Design

<u>Assumptions</u>

-beams are No. 3 SPF dimensional lumber
-LTB is prevented by lateral bracing from joists

<u>Define Variables</u>

beam breadth (actual) $b := 8\ in$ beam depth (actual) $d := 10\ in$

beam breadth (nominal) $b_n := 8\ in$ beam depth (nominal) $d_n := 10$

beam section modulus $S := 112.8\ in^3$

beam MoI $I := 535.9\ in^4$

unbraced length $l := 9.5\ ft$

$\gamma_w \cdot SG = 26.208\ \frac{lbf}{ft^3}$

tributary width $t_b := 12.5\ ft$

section weight in lbf/ft $w_b := 13\ \frac{lbf}{ft}$

applicable dead loads

$DL_{floors} = 16.92\ psf$ $LL_{floors} = 40\ psf$ $DL_{beams} := \frac{w_b}{t_b} = 1.04\ psf$

Design Calculations

Adjustment Factors

load duration factor $C_D := 1.0$

wet service factor $C_M := 1.0$

temperature factor $C_t := 1.0$

size factor...

...for bending $C_{F\_b} := 1.0$ ...for tension $C_{F\_t} := 1.0$ ...for compression (parallel to grain) $C_{F\_c} := 1.0$

flat use factor $C_{fu} := 1.0$

incising factor $C_i := 1.0$

repetitive member factor $C_r := 1.0$

column stability factor $C_p := 1.0$

buckling stiffness factor $C_T := 1.0$

bearing area factor $C_b := 1.0$

beam stability factor (CL)

determine effective length of joist $\frac{l}{d} = 11.4$ $\frac{l}{d} \geq 7 = 1$

$l_e := 1.63 \cdot l + 3 \cdot d = 17.985\ ft$

calculate slenderness ratio

$R_B := \sqrt[2]{\frac{l_e \cdot d}{b^2}} = 5.807$ $R_B < 50 = 1$

calculate reference bending design value

$F_{b\#} := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_{F\_b} \cdot C_i \cdot C_r$ $E_{min}' := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T = \left(5.1 \cdot 10^5\right)\ psi$

$F_{bE} := \frac{1.2 \cdot E_{min}'}{R_B^2} = \left(2.613 \cdot 10^6\right)\ psf$

calculate beam stability factor

$$C_L := \frac{1+\left(\frac{F_{bE}}{F_{b\#}}\right)}{1.9} - \sqrt[2]{\left(\frac{1+\left(\frac{F_{bE}}{F_{b\#}}\right)}{1.9}\right)^2 - \frac{\left(\frac{F_{bE}}{F_{b\#}}\right)}{0.95}} = 0.997$$

Moment Design

estimate Fb' using applicable adjustment factors

$$F_b' := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_{F\_b} \cdot C_{fu} \cdot C_i = \left(1.257 \cdot 10^5\right)\ psf$$

estimate bending moment

$$M := \frac{\left(DL_{floors} + LL_{floors} + DL_{beams}\right) \cdot t_b \cdot l^2}{8} = \left(8.173 \cdot 10^3\right)\ lbf \cdot ft$$

estimate applied bending moment

$$f_{b\_ini} := \frac{M}{S} = \left(1.252 \cdot 10^5\right)\ psf$$

use load combination (in PLF) to calculate distributed load

distributed dead load $w_{DL} := \left(DL_{floors} + DL_{beams}\right) \cdot t_b = 224.496\ plf$

distributed live load $w_{LL} := LL_{floors} \cdot t_b = 500\ plf$

dead+live load combo $w_{floors} := w_{DL} + w_{LL} = 724.496\ plf$

Case 1: Dead+Live Load

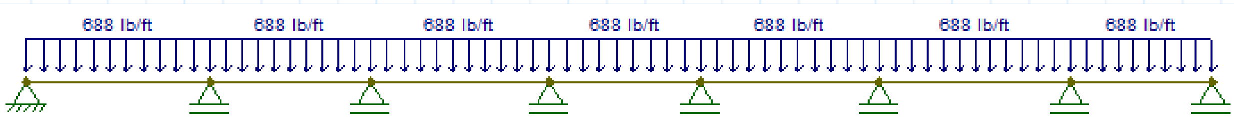


Figure 3.1 Distributed load over continuous beam

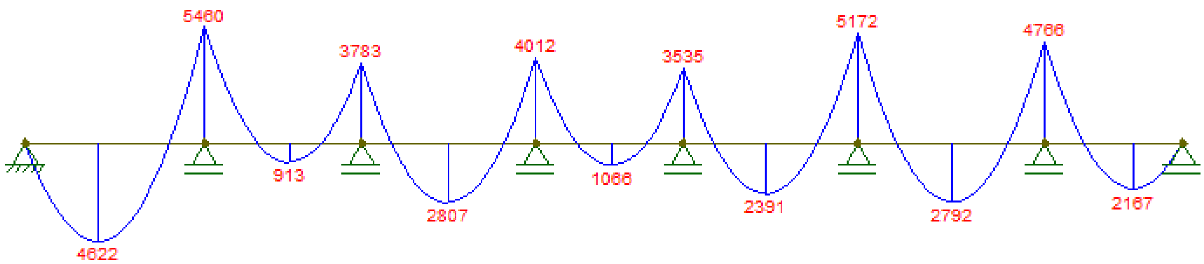


Figure 3.2 Moment diagram for load case 1

Case 2a: Distributed Dead Load

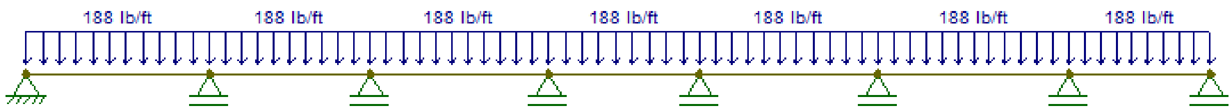


Figure 3.3 Load case 2 for negative moment design

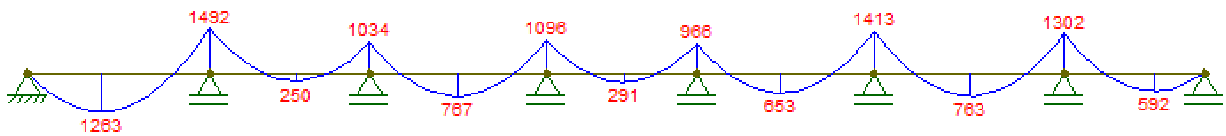


Figure 3.4 Moment diagram for distributed dead load

Case 2b: Staggered Live Load

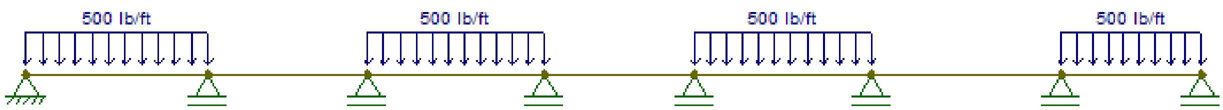


Figure 3.5 Live Load case 2b for negative moment design

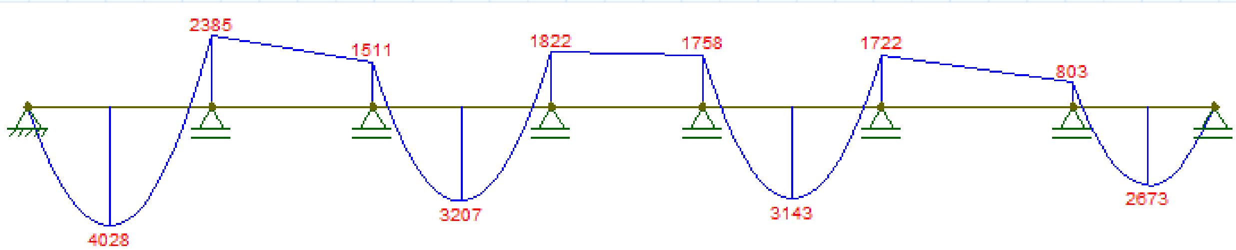


Figure 3.6 Moment diagram for load case 2b

<u>Case 3a:</u> Adjacent Span Live Load

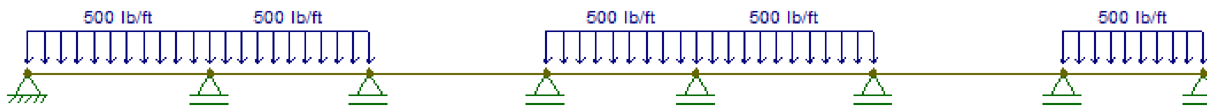


Figure 3.7 Live Load case 3a for negative moment design

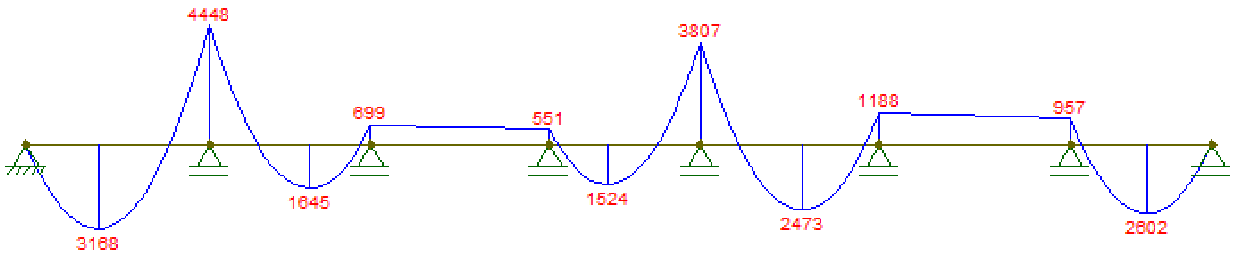


Figure 3.8 Moment diagram for load case 3a

<u>Case 3b:</u> Adjacent Span Live Load

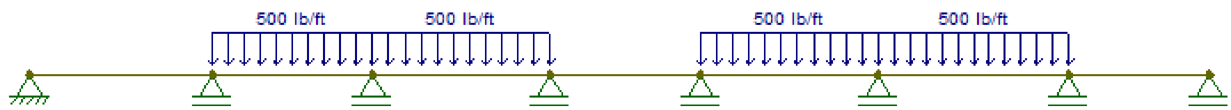


Figure 3.9 Live Load case 3b for negative moment design

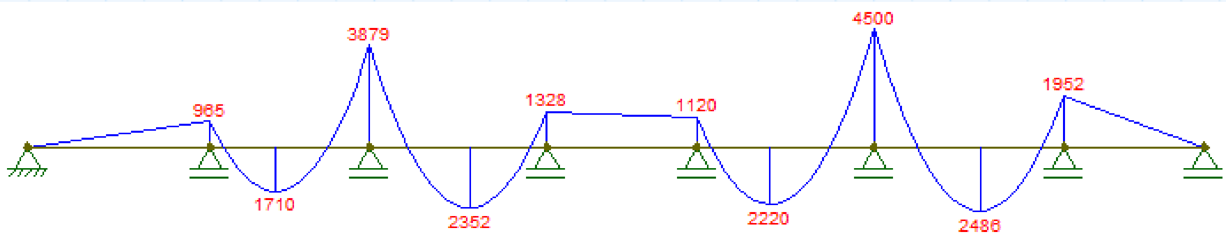


Figure 3.10 Moment diagram for load case 3b

case 3b governs; use moment to determine maximum applied moment

$$M := 4500\ lbf \cdot ft + 1413\ lbf \cdot ft$$

$$f_b := \frac{M}{S} = \left(9.058 \cdot 10^4\right)\ psf$$

compare applied moment to section strength

Design Equation: $F_b' \geq f_b = 1$ bending strength satisfies design equation

DCR: $\frac{f_b}{F_b'} = 0.721$

Shear Design

estimate Fv' using applicable adjustment factors

$$F_v' := F_v \cdot C_D \cdot C_M \cdot C_t \cdot C_i = \left(1.944 \cdot 10^4\right)\ psf$$

determine applied shear stress fv

$w_{floors} = 724.496\ plf$ $l = 9.5\ ft$

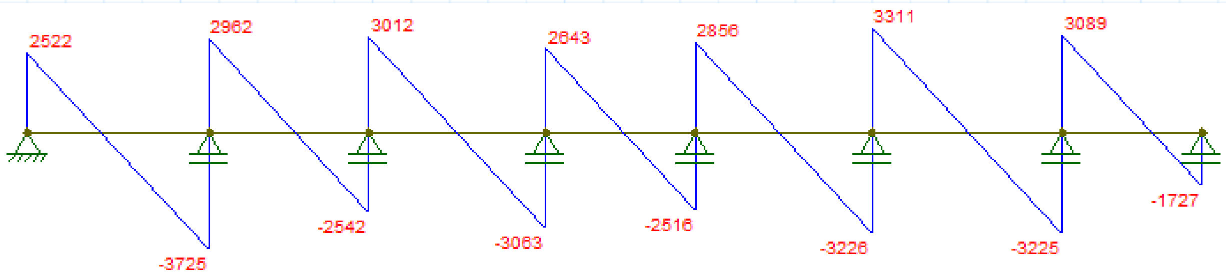


Figure 3.11 Shear force diagram for 10.4' beams

$V := 3725\ lbf$

determine applied shear stress parallel to grain

$$f_v := \frac{3 \cdot V}{2 \cdot b \cdot d \cdot 3} = \left(3.353 \cdot 10^3\right)\ psf$$

Design Equation: $f_v \leq F_v' = 1$ joists satisfy shear design

DCR: $\frac{f_v}{F_v'} = 0.172$

Deflection Design

*load case 3b governs

determine appropriate short and long term loads

$w_{st} := (0.5 \cdot LL_{floors}) \cdot t_b = 250\ plf$ $\quad$ $w_{lt} := ((DL_{floors} + DL_{beams}) + 0.5 \cdot LL_{floors}) \cdot t_b = 474.496\ plf$

apply load combination for total deflection

$1.5 \cdot w_{lt} = 711.744\ plf$

$w_{tot} := 1.5 \cdot w_{lt} + 0.5 \cdot w_{st} = 836.744\ plf$

check short-term deflection...

...using equations

$$\delta_{st} := \frac{5 \cdot w_{st} \cdot l^4}{384 \cdot E \cdot I} = 0.061\ in \qquad \Delta_{st} := \frac{l}{360} = 0.317\ in$$

...using load analysis program (ftool)

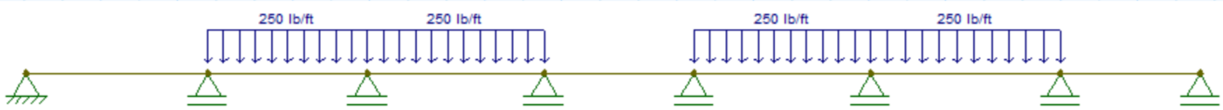


Figure 3.12 Distributed load for short-term deflection

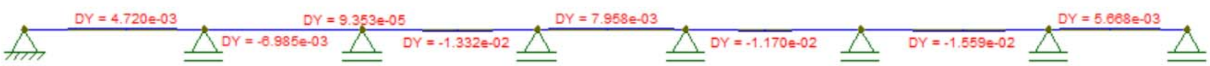


Figure 3.13 Diagram showing short-term deflection

check total deflection

...using equations

$$\delta_{lt} := \frac{5 \cdot w_{lt} \cdot l^4}{384 \cdot E \cdot I} = 0.116\ in \qquad \delta_{tot} := 1.5 \cdot \delta_{lt} + \delta_{st} = 0.235\ in \qquad \Delta_{tot} := \frac{l}{240} = 0.475\ in$$

short-term deflection from ftool $\qquad$ $\delta_{st} := 0.01559\ in$

...using load analysis program (ftool)

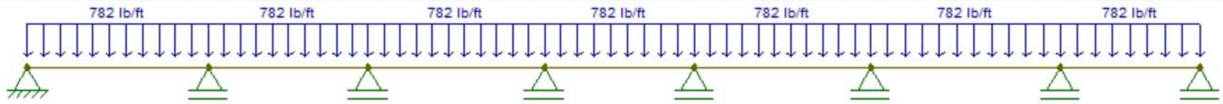


Figure 3.14 Distributed loading for total deflection

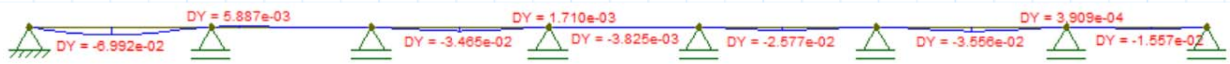


Figure 3.15 Deflection diagram for total deflection

$\delta_{tot} := 0.06992\ in$

Design Equation: $\Delta_{st} \geq \delta_{st} = 1$ deflection requirements check out

DCR: $\dfrac{\delta_{st}}{\Delta_{st}} = 0.049$

Design Equation: $\Delta_{tot} \geq \delta_{tot} = 1$ deflection requirements check out

DCR: $\dfrac{\delta_{tot}}{\Delta_{tot}} = 0.147$

Bearing Design

bearing length parallel to grain $l_b := 6\ in$

bearing breadth perpendicular to grain $b_b := b = 8\ in$

define bearing area $A_b := l_b \cdot b_b = 48\ in^2$

define bearing area factor $C_b := \dfrac{l_b + 0.375\ in}{l_b}$

determine reaction force at column P

length of longest span beam $l = 9.5\ ft$

$P := 0.5 \cdot l \cdot w_{floors} = \left(3.441 \cdot 10^3\right)\ lbf$

$f_{c\_p} := \frac{P}{A_b} = (1.032 \cdot 10^4)\ psf$

apply appropriate adjustment factors to design values

$F_{c\_p} = 425\ psi$

$F_{c\_p}' := F_{c\_p} \cdot C_M \cdot C_t \cdot C_i \cdot C_b = (6.503 \cdot 10^4)\ psf$

ensure bearing is satisfied

Design Equation: $F_{c\_p}' \ge f_{c\_p} = 1$

DCR: $\frac{f_{c\_p}}{F_{c\_p}'} = 0.159$

<u>Design Summary</u>

The continuous beam was analyzed to resist moment, shear, deflection, and bearing; the corresponding design equations are shown below:

moment $F_b' = (1.257 \cdot 10^5)\ psf$ $f_b = (9.058 \cdot 10^4)\ psf$

$DCR := \frac{f_b}{F_b'} = 0.721$

shear $F_v' = (1.944 \cdot 10^4)\ psf$ $f_v = (3.353 \cdot 10^3)\ psf$

$DCR := \frac{f_v}{F_v'} = 0.172$

deflection $\Delta_{st} = 0.317\ in$ $\delta_{st} = 0.016\ in$

$DCR := \frac{\delta_{st}}{\Delta_{st}} = 0.049$

$\Delta_{tot} = 0.475\ in$ $\delta_{tot} = 0.07\ in$

$DCR := \frac{\delta_{tot}}{\Delta_{tot}} = 0.147$

bearing $F_{c\_p}' = (6.503 \cdot 10^4)\ psf$ $f_{c\_p} = (1.032 \cdot 10^4)\ psf$

$DCR := \frac{f_{c\_p}}{F_{c\_p}'} = 0.159$

V. Bearing Wall Analysis

Assumptions
-12" clay wythe bearing walls (use ASCE 7-16 for dead load)
-first floor system rests on foundation wall
-loads acting on bearing wall include:
- --roof loads
- --residential floor live and dead loads
- --dead load due to self weight of bearing wall

-maximum unsupported height to thickness ratio (h/t) = 10

Define Variables

dead load from residential floor system $DL_{floors} = 16.92\ psf$

live load from residential floor system $LL_{floors} = 40\ psf$

dead load from self-weight of brick bearing walls $DL_{brick} := 115\ psf$

thickness of wall $t_{brick} := 12\ in$

height of wall $h_{brick} := 14\ ft$

Design Calculations

Determine axial compressive load for bearing wall analysis

calculate tributary area for bearing wall anaylsis

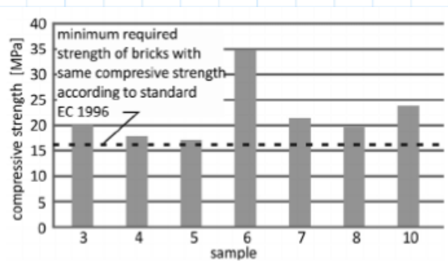
tributary width $t_b := 25\ ft$

combine loads into a continuous axial compressive force

$$P := \left(DL_{floors} + LL_{floors} + DL_{brick}\right) \cdot t_b = \left(4.298 \cdot 10^3\right) \frac{lbf}{ft}$$

determine material strength of brick bearing wall

ultimate compressive strength of common building brick $f_m' := 17\ MPa = \left(2.466 \cdot 10^3\right)\ psi$



*Note: this value is a conservative result from research conducted by Jiri Witzany, Tomas Cejka, Miroslav Sykora, and Milan Holicky. It was taken from their article, "Strength Assessment of Historic Brick Masonry" published in the Journal of Civil Engineering and Management 08 Dec. 2015.

use equation from table 7-1 in Reinforced Masonry Design to compute maximum allowable working stress in clay brick

max allowable stress $F_c' := f_m' \cdot 0.2 \cdot \left(1 - \left(\frac{h_{brick}}{40 \cdot t_{brick}}\right)^3\right) = 471.985 \ psi$

determine applied stress due to load combination

max applied stress due to floor system $f_c := \frac{P}{t_{brick}} = 29.847 \ psi$

use design equation to determine whether max applied stress exceeds max allowable stress

design equation $F_c' \geq f_c = 1$

demand-capacity ratio $DCR := \frac{f_c}{F_c'} = 0.063$

determine h/t ratio satisfies building code requirements

$$\frac{h_{brick}}{t_{brick}} = 14$$

Design Summary

The clay brick masonry bearing walls were analyzed to resist the compressive forces from the roof and residential floor loads. The design values and demand capacity ratio are as follows:

$F_c' = 471.985 \ psi$ $f_c = 29.847 \ psi$

$$DCR := \frac{f_c}{F_c'} = 0.063$$

VI. Design of Stair System

Assumptions
-all members are No. 2 SP-F
-floor system is continuously supported by wall framing
-must design handrail for uniform load of 50 lbf/ft and single concentrated load of 200 lbf
-longest flight is 15 steps

Define Variables

VI-i Stringer Design

reference design values

specific gravity of SPF	$SG := 0.42$
modulus of elasticity	$E := 1400000\ psi$
	$E_{min} := 510000\ psi$
bending strength	$F_b := 875\ psi$
tension parallel to grain	$F_t := 450\ psi$
shear parallel to grain	$F_v := 135\ psi$
compression perpendicular to grain	$F_{c\_p} := 425\ psi$
compression parallel to grain	$F_c := 1150\ psi$

section properties (use 3x14)

depth	$d_{string} := 13.25\ in$
effective depth of stringers	$d_{eff} := 7.25\ in$
breadth	$b_{string} := 2.5\ in$

cross-sectional area of stringers

$$A_{string} := d_{eff} \cdot b_{string} = 18.125\ in^2$$

section modulus	$S := 21.9\ in^3$
moment of inertia	$I := 79.39\ in^4$

determine max length of stringers

number of steps in largest flight $N_{steps} := 15$

height of risers $h := 7\ in$

length of treads $l := 11\ in$

width of treads/risers $b := 36\ in$

max length of stringers $L := \sqrt[2]{\left(l \cdot N_{steps}\right)^2 + \left(h \cdot N_{steps}\right)^2} = 16.298\ ft$

*assume max stringer length of 16' 4"

tributary width of stringers $t_b := \frac{b}{3} = 1\ ft$

select 1x12 plywood for treads

depth of tread $d_{tread} := 1\ in$

breadth of tread $b_{tread} := 12\ in$

cross sectional area of treads $A_{tread} := d_{tread} \cdot b_{tread} = 12\ in^2$

Design Calculations

determine appropriate dead and live loads

dead load due to self-weight of stringers $DL_{string} := \gamma_w \cdot SG \cdot A_{string} = 3.299\ plf$

dead load due to self-weight of treads $DL_{treads} := \gamma_w \cdot SG \cdot A_{tread} = 2.184\ plf$

dead load of stair cover $DL_{cover} := \left(DL_{cover} - 11\ psf\right) \cdot t_b = 0.8\ plf$ *subtract soundboard and HVAC weight from cover calculations

stair dead load $DL_{stairs} := DL_{string} + DL_{treads} + DL_{cover} = 6.283\ plf$

stair live load $LL_{stairs} := 40\ psf$

determine applicable load combinations

use load combo 2 $w_{stairs} := \left(LL_{stairs}\right) \cdot t_b + DL_{stairs} = 46.283\ plf$

determine applicable adjustment factors

load duration factor $C_D := 1.0$

wet service factor $C_M := 1.0$

temperature factor $C_t := 1.0$

size factor...

...for bending $C_{F\_b} := 1.0$ ...for tension $C_{F\_t} := 1.0$ ...for compression (parallel to grain) $C_{F\_c} := 1.0$

flat use factor $C_{fu} := 1.0$

incising factor $C_i := 1.0$

repetitive member factor $C_r := 1.0$

column stability factor $C_p := 1.0$

buckling stiffness factor $C_T := 1.0$

beam stability factor (CL)

determine effective length of joist $\frac{L}{d_{eff}} = 26.976$ $\frac{L}{d_{eff}} \geq 7 = 1$

$L_e := 1.63 \cdot L + 3 \cdot d_{eff} = 28.378\ ft$

calculate slenderness ratio

$R_B := \sqrt[2]{\frac{L_e \cdot d_{eff}}{{b_{string}}^2}} = 19.875$ $R_B < 50 = 1$

calculate reference bending design value

$F_{b\#} := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_{F\_b} \cdot C_i \cdot C_r$ $E_{min}' := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T = \left(5.1 \cdot 10^5\right)\ psi$

$$F_{bE} := \frac{1.2 \cdot E_{min}'}{{R_B}^2} = \left(2.231 \cdot 10^5\right)\ psf$$

calculate beam stability factor

$$C_L := \frac{1 + \left(\frac{F_{bE}}{F_{b\#}}\right)}{1.9} - \sqrt[2]{\left(\frac{1 + \left(\frac{F_{bE}}{F_{b\#}}\right)}{1.9}\right)^2 - \frac{\left(\frac{F_{bE}}{F_{b\#}}\right)}{0.95}} = 0.946$$

determine column stability factor

find column effective length $L_e := \frac{1}{15} \cdot L = 1.087\ ft$ (lateral bracing from risers)

calculate reference compession design value (for column stability factor)

$$F_{c\#} := F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_{F\_c} \cdot C_i$$

$$E_{min}' = \left(5.1 \cdot 10^5\right)\ psi$$

$$F_{cE} := \frac{0.822 \cdot E_{min}'}{\left(\frac{L_e}{b_{string}}\right)^2} = \left(1.541 \cdot 10^4\right)\ psi$$

$c := 0.8$ (for sawn lumber)

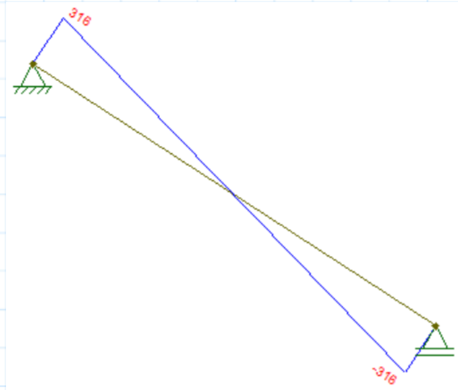
calculate column stability factor

$$C_P := \frac{1 + \left(\frac{F_{cE}}{F_{c\#}}\right)}{2 \cdot c} - \sqrt[2]{\left(\frac{1 + \left(\frac{F_{cE}}{F_{c\#}}\right)}{2 \cdot c}\right)^2 - \frac{\left(\frac{F_{cE}}{F_{c\#}}\right)}{c}} = 0.984$$

Tension Design

calculate reference tension design value

$F_t' := F_t \cdot C_D \cdot C_M \cdot C_t \cdot C_{F\_t} \cdot C_i = 450\ psi$



calculate maximum applied tension stress

max tensile force $T := 205\ lbf$

max tensile stress $f_t := \frac{T}{A_{string}} = 11.31\ psi$

Compression Design

calculate reference compression design value

$F_c' := F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_{F\_c} \cdot C_i \cdot C_p = \left(1.15 \cdot 10^3\right)\ psi$

calculate maximum applied tension stress

max tensile force $C := 205\ lbf$

max tensile stress $f_c := \frac{C}{A_{string}} = 11.31\ psi$

Moment Design

determine Fb* to check combined bending and tension

$$F_{b\#} := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_{F\_b} \cdot C_{fu} \cdot C_i = 875\ psi$$

estimate Fb' using applicable adjustment factors

$$F_{b\_i}' := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_{F\_b} \cdot C_{fu} \cdot C_i = 827.554\ psi$$

apply loads to stringer using structural analysis program (ftool)

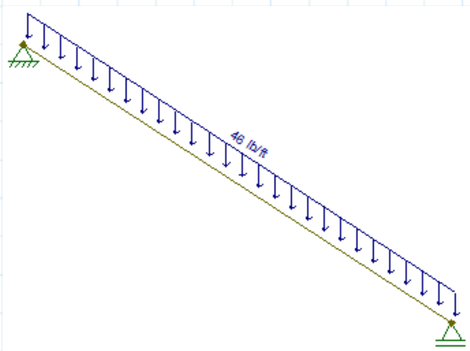


Figure 6.1.1 Distributed load for stair stringer

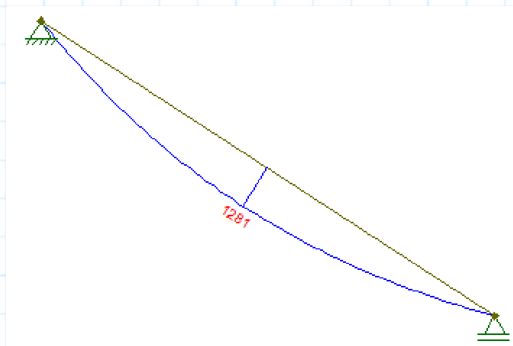


Figure 6.1.2 Bending moment diagram for stair stringer

max bending moment developed in stringer $M := 1281\ lbf \cdot ft$

determine maximum applied bending stress

$$f_{b\_i} := \frac{M}{S} = 701.918\ psi$$

compare applied bending stress to bending strength of material

Design Equation: $F_{b\_i}' \geq f_{b\_i} = 1$

$$DCR_{b\_i} := \frac{f_{b\_i}}{F_{b\_i}'} = 0.848$$ section is acceptable for design

check for combined bending plus axial tension

$$DCR_{b\_t} := \frac{f_t}{F_t'} + \frac{f_b}{F_{b\#}} = 0.744$$ bending plus axial tension is satisfied

check for combined bending plus axial compression (moment amplification)

calculate amplification factor $B_1 := \left(1 - \frac{f_c}{F_{cE}}\right)^{-1} = 1.001$

$DCR_{b\_c} := \left(\frac{f_c}{F_c'}\right)^2 + B_1 \cdot \left(\frac{f_b}{F_b'}\right) = 0.721$ bending plus axial compression is satisfied

Shear Design

estimate Fv' using applicable adjustment factors

$F_{v\_i}' := F_v \cdot C_D \cdot C_M \cdot C_t \cdot C_i = \left(1.944 \cdot 10^4\right)$ ***psf***

determine applied shear stress fv

$w_{stairs} = 46.283$ ***plf***

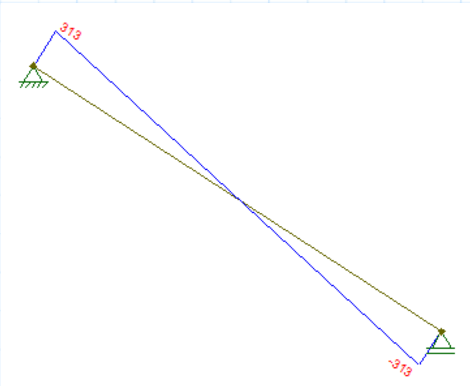


Figure 6.1.3 Shear force diagram for stringers

$V := 313$ ***lbf***

determine applied shear stress parallel to grain

$f_{v\_i} := \frac{3 \cdot V}{2 \cdot b_{string} \cdot d_{eff}} = \left(3.73 \cdot 10^3\right)$ ***psf***

Design Equation: $f_v \le F_v' = 1$

$DCR_{v\_i} := \frac{f_{v\_i}}{F_{v\_i}'} = 0.192$ section is acceptable for design

Deflection Design

determine appropriate short and long term loads

short term $w_{st} := \left(0.5 \cdot LL_{stairs}\right) \cdot t_b = 20$ ***plf***

long term $w_{lt} := DL_{stairs} + \left(0.5 \cdot LL_{stairs} \cdot t_b\right) = 26.283$ ***plf***

apply load combination for total deflection

$w_{tot} := 1.5 \cdot w_{lt} + 0.5 \cdot w_{st} = 49.424$ ***plf***

check short-term deflection...

...using equations

$$\delta_{st} := \frac{5 \cdot w_{st} \cdot L^4}{384 \cdot E \cdot I} = 0.286 \ \textit{in} \qquad \Delta_{st\_i} := \frac{L}{360} = 0.543 \ \textit{in}$$

...using load analysis program (ftool)

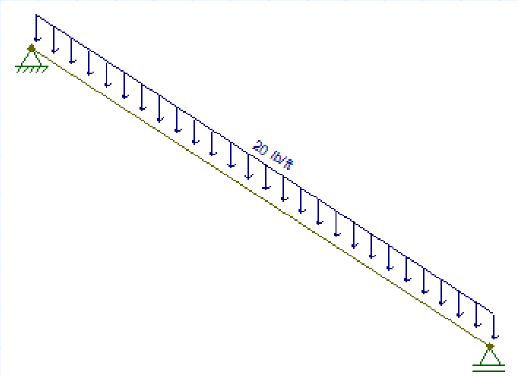


Figure 6.1.4 Distributed load for short-term deflection

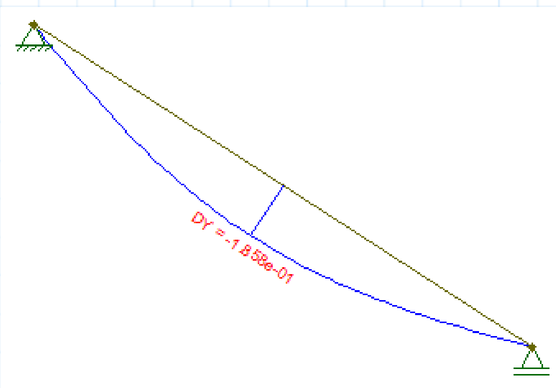


Figure 6.1.5 Diagram showing short-term deflection

short term deflection results from load analysis $\delta_{st\_i} := 0.186\ in$

Design Equation: $\Delta_{st} \ge \delta_{st} = 1$

$DCR_{st\_i} := \frac{\delta_{st\_i}}{\Delta_{st\_i}} = 0.342$ design is adequate for deflection

check total deflection

...using equations

$\delta_{lt} := \frac{5 \cdot w_{lt} \cdot L^4}{384 \cdot E \cdot I} = 0.375\ in$ $\delta_{tot} := 1.5 \cdot \delta_{lt} + \delta_{st} = 0.849\ in$ $\Delta_{tot} := \frac{L}{240} = 0.815\ in$

...using load analysis program (ftool)

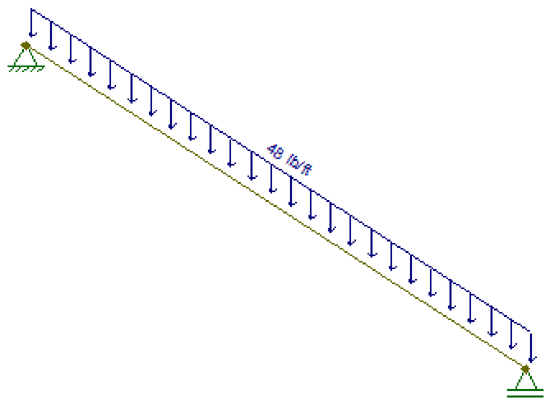


Figure 6.1.6 Distributed load for total deflection

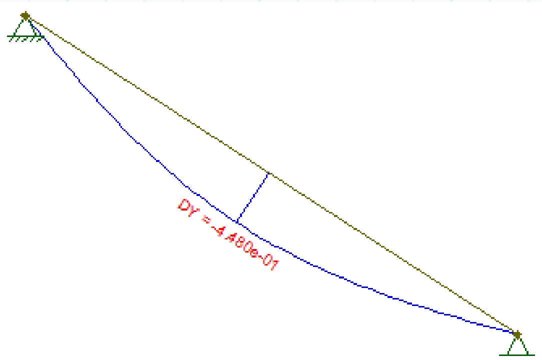


Figure 6.1.7 Diagram showing total deflection

$\delta_{tot} := 0.448\ in$

Design Equation: $\Delta_{tot} \geq \delta_{tot} = 1$

DCR: $DCR_{tot\_i} := \frac{\delta_{tot}}{\Delta_{tot}} = 0.55$ design is adequate for deflection

VI-ii Landing Joist Design

select 2x6 @16" O.C. for initial stud size

reference design values		section properties of 2x4 joists	
specific gravity of SPF	$SG := 0.42$	depth	$d_{joist} := 3.5\ in$
modulus of elasticity	$E := 1400000\ psi$	breadth	$b_{joist} := 1.5\ in$
	$E_{min} := 510000\ psi$	length of joists	$l := 3\ ft$
bending strength	$F_b := 875\ psi$	cross-sectional area of joists	
tension parallel to grain	$F_t := 450\ psi$	$A_{joist} := d_{joist} \cdot b_{joist} = 5.25\ in^2$	
shear parallel to grain	$F_v := 135\ psi$	section modulus	$S_j := 3.06\ in^3$
compression perpendicular to grain	$F_{c\_p} := 425\ psi$	moment of inertia	$I_j := 5.359\ in^4$
		tributary width of studs	$t_b := 16\ in$
compression parallel to grain	$F_c := 1150\ psi$		

determine appropriate dead and live loads

dead load due to self-weight of joists $DL_{joists} := \gamma_w \cdot SG \cdot A_{joist} = 0.956\ plf$

landing dead load $DL_{landing} := DL_{cover} + DL_{joists} = 1.756\ plf$

determine applicable load combination

use load combo 2 $w_{landing} := \left(LL_{stairs}\right) \cdot t_b + DL_{landing} = 55.089\ plf$

determine applicable adjustment factors

load duration factor $C_D := 1.0$

wet service factor $C_M := 1.0$

temperature factor $C_t := 1.0$

size factor...

...for bending $C_{F\_b} := 1.5$ ...for tension $C_{F\_t} := 1.5$ ...for compression (parallel to grain) $C_{F\_c} := 1.15$

flat use factor $C_{fu} := 1.0$

incising factor $C_i := 1.0$

repetitive member factor $C_r := 1.15$

column stability factor $C_p := 1.0$

buckling stiffness factor $C_T := 1.0$

bearing area factor $C_b := 1.0$

Bending Moment Design

bending strength of No. 2 SPF $F_b = 875\ psi$

calculate adjusted bending strength $F_{b\_ii}' := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_{F\_b} \cdot C_{fu} \cdot C_i \cdot C_r$

determine applied loads using load analysis program (ftool)

$w_{landing} = 55.089\ plf$

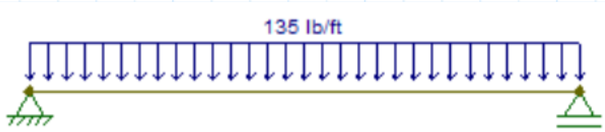


Figure 6.2.1 Distributed loading over joist

Figure 6.2.2 Bending moment diagram for joist

max moment in joist $M := 152\ lbf \cdot ft$

determine applied bending stress using section modulus $f_{b\_ii} := \frac{M}{S_j} = 596.078\ psi$

compare applied stress to joist strength $F_{b\_ii}' = \left(1.428 \cdot 10^3\right)\ psi$

$$DCR_{b\_ii} := \frac{f_{b\_ii}}{F_{b\_ii}'} = 0.418$$

Shear Design

estimate Fv' using applicable adjustment factors

$$F_{v\_ii}' := F_v \cdot C_D \cdot C_M \cdot C_t \cdot C_i = \left(1.944 \cdot 10^4\right)\ psf$$

determine applied shear stress fv

$$w_{landing} = 55.089\ plf$$

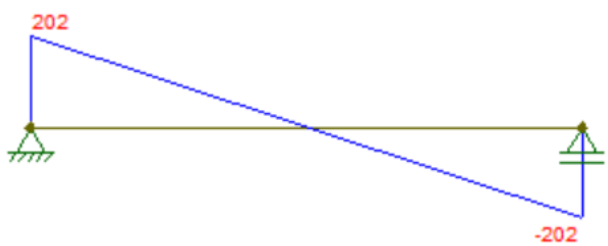


Figure 6.2.3 Shear force diagram for landing joists

$$V := 202\ lbf$$

determine applied shear stress parallel to grain

$$f_{v\_ii} := \frac{3 \cdot V}{2 \cdot b \cdot d} = 121.2\ psf$$

Design Equation: $f_v \le F_v{}' = 1$

$DCR_{v\_ii} := \frac{f_{v\_ii}}{F_{v\_ii}{}'} = 0.006$ joists satisfy shear design

Deflection Design

determine appropriate short and long term loads

short term deflection load $w_{st} := 0.5 \cdot (LL_{floors}) \cdot t_b = 26.667\ plf$

long term deflection load $w_{lt} := DL_{landing} + (0.5 \cdot LL_{floors})\ t_b = 28.422\ plf$

apply load combination for total deflection

$w_{tot} := 1.5 \cdot w_{lt} + 0.5 \cdot w_{st} = 55.967\ plf$

check short-term deflection...

...using equations

$\delta_{st} := \frac{5 \cdot w_{st} \cdot l^4}{384 \cdot E \cdot I} = (4.373 \cdot 10^{-4})\ in$ $\Delta_{st\_ii} := \frac{l}{360} = 0.1\ in$

...using load analysis program (ftool)

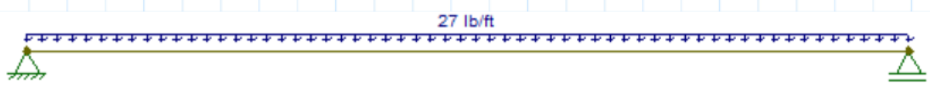


Figure 6.2.4 Distributed load for short-term deflection

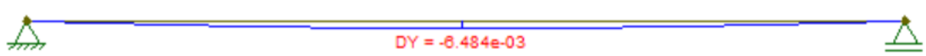


Figure 6.2.5 Diagram showing short-term deflection

short term deflection results from load analysis $\delta_{st\_ii} := 0.006484\ in$

$DCR_{st\_ii} := \frac{\delta_{st\_ii}}{\Delta_{st\_ii}} = 0.065$

check total deflection

...using equations

$\delta_{lt} := \frac{5 \cdot w_{lt} \cdot l^4}{384 \cdot E \cdot I} = \left(4.66 \cdot 10^{-4}\right)\ in$ $\delta_{tot} := 1.5 \cdot \delta_{lt} + \delta_{st\_ii} = 0.007\ in$ $\Delta_{tot\_ii} := \frac{l}{240} = 0.15\ in$

...using load analysis program (ftool)

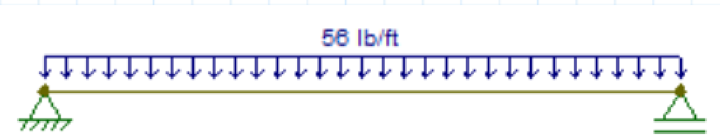


Figure 6.2.6 Distributed load for total deflection

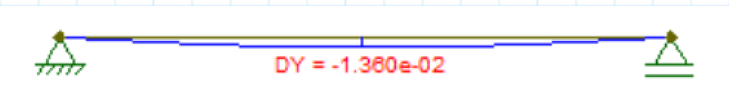


Figure 6.2.7 Diagram showing total deflection

$\delta_{tot\_ii} := 0.0136\ in$

Design Equation: $\Delta_{tot} \geq \delta_{tot} = 1$ deflection requirements check out

DCR: $DCR_{tot\_ii} := \frac{\delta_{tot\_ii}}{\Delta_{tot\_ii}} = 0.091$

Bearing Design

bearing length parallel to grain $l_b := 2\ in$

bearing breadth perpendicular to grain $b_b := b = 36\ in$

define bearing area $A_b := l_b \cdot b_b = 72\ in^2$

define bearing area factor $C_b := \frac{l_b + 0.375\ in}{l_b} = 1.188$

determine reaction force at column P

length of longest span beam $l=3\ ft$

$P:=0.5 \cdot l \cdot w_{floors}=\left(1.087 \cdot 10^{3}\right)\ lbf$

$f_{c\_pii}:=\frac{P}{A_b}=\left(2.173 \cdot 10^{3}\right)\ psf$

apply appropriate adjustment factors to design values

$F_{c\_p}=425\ psi$

$F_{c\_pii}':=F_{c\_p} \cdot C_M \cdot C_t \cdot C_i \cdot C_b=\left(7.268 \cdot 10^{4}\right)\ psf$

ensure bearing is satisfied

Design Equation: $F_{c\_pii}' \geq f_{c\_pii}=1$

DCR: $DCR_{c\_pii}:=\frac{f_{c\_pii}}{F_{c\_pii}'}=0.03$

VI-iii Landing Stud Design

section properties of 2x4 studs

depth $d_{stud}:=3.5\ in$

breadth $b_{stud}:=1.5\ in$

cross-sectional area of studs

$A_{stud}:=d_{stud} \cdot b_{stud}=5.25\ in^2$

section modulus $S:=3.06\ in^3$

moment of inertia $I:=5.359\ in^4$

tributary width of studs $t_b:=16\ in$

length of studs $l_{stud}:=6\ ft$

determine appropriate dead and live loads

dead load due to self-weight of studs $DL_{stud} := \gamma_w \cdot SG \cdot A_{stud} = 0.956\ plf$

dead load due to self-weight of joists $DL_{joists} := \gamma_w \cdot SG \cdot A_{joist} = 0.956\ plf$

landing dead load $DL_{landing} := DL_{cover} + DL_{joists} = 1.756\ plf$

point load from shear force of stringer $P_{stairs} := 316\ lbf$

determine applicable load combination

use load combo 2 $w_{landing} := \left(LL_{stairs}\right) \cdot t_b + DL_{landing} + DL_{stud} = 56.044\ plf$

determine applied axial compressive stress

$$P_{landing} := w_{landing} \cdot 3\ ft + P_{stairs} = 484.133\ lbf$$

$$f_{c\_iii} := \frac{P_{landing}}{A_{stud}} = 92.216\ psi$$

determine applicable adjustment factors

load duration factor $C_D := 1.0$

wet service factor $C_M := 1.0$

temperature factor $C_t := 1.0$

size factor...

...for bending $C_{F\_b} := 1.3$ ...for tension $C_{F\_t} := 1.3$ ...for compression (parallel to grain) $C_{F\_c} := 1.1$

flat use factor $C_{fu} := 1.0$

incising factor $C_i := 1.0$

repetitive member factor $C_r := 1.15$

column stability factor $C_p := 1.0$

buckling stiffness factor $C_T := 1.0$

bearing area factor $C_b := 1.0$

determine column stability factor

find column effective length $l_e := 1.0 \cdot l_{stud} = 6\ ft$

calculate reference compession design value (for column stability factor)

$F_{c\#} := F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_{F\_c} \cdot C_i$ $E_{min}' = \left(5.1 \cdot 10^5\right)\ psi$

$F_{cE} := \dfrac{0.822 \cdot E_{min}'}{\left(\dfrac{l_e}{b}\right)^2} = \left(1.048 \cdot 10^5\right)\ psi$ $c := 0.8$ (for sawn lumber)

calculate column stability factor

$$C_P := \frac{1+\left(\frac{F_{cE}}{F_{c\#}}\right)}{2 \cdot c} - \sqrt[2]{\left(\frac{1+\left(\frac{F_{cE}}{F_{c\#}}\right)}{2 \cdot c}\right)^2 - \frac{\left(\frac{F_{cE}}{F_{c\#}}\right)}{c}} = 0.998$$

determine axial compressive strength of material

$F_{c\_iii}' := F_{c\#} \cdot C_P = \left(1.262 \cdot 10^3\right)\ psi$

design equation: $F_c' \geq f_c = 1$

$DCR_{c\_iii} := \dfrac{f_{c\_iii}}{F_{c\_iii}'} = 0.073$

Design Summary

i. Stringer Design Results

moment	$DCR_{b\_i}=0.848$	deflection	$DCR_{st\_i}=0.342$
shear	$DCR_{v\_i}=0.192$		$DCR_{tot\_i}=0.55$

ii. Landing Joist Design

moment	$DCR_{b\_ii}=0.418$	deflection	$DCR_{st\_ii}=0.065$
shear	$DCR_{v\_ii}=0.006$		$DCR_{tot\_ii}=0.091$
		bearing	$DCR_{c\_pii}=0.03$

iii. Landing Stud Design

compression	$DCR_{c\_iii}=0.073$

VII. Balcony System Design

Assumptions
-all wood members are spruce-pine-fir (SPF) No. 2
-all floor joists are TJI 360 engineered wood products
-lateral support is provided to prevent LTB
-joists rest on brick bearing wall

Define Variables

length of balconies	$L_{balcony} := 6\ ft$
width of balconies	$b_{balcony} := 25\ ft$

VII-i. TJI Joist Design

use Weyerhaeuser TJI Joist design manual to find appropriate engineered wood beam size

*use TJI 360 with 16" depth @12" O.C.

reference design values

modulus of elasticity	$EI := 830000000\ in^2 \cdot lbf$
max resistive moment	$M_{all} := 8405\ lbf \cdot ft$
shear parallel to grain	$V_{all} := 2190\ lbf$
compression perpendicular to grain	$P_{all} := 1080\ lbf$

section properties

length	$l := 25\ ft$
depth	$d := 16\ in$
flange breadth	$b_f := \left(2 + \frac{5}{16}\right)\ in$
flange depth	$d_f := 1.375\ in$
web breadth	$b_w := \frac{3}{8}\ in$
tributary width	$t_b := 12\ in$
weight in plf	$w_{joists} := 3.5\ \frac{lbf}{ft}$

Design Calculations

determine applicable loads

Snow Loads

ground snow load $p_g := 25\ psf$

exposure factor $C_e := 1.2$

thermal condition $C_t := 1.0$

importance factor $I_s := 1.0$

determine appropriate slope factor $C_s := 1.0$

roof slope $s := 10$

calculate flat roof snow load $p_f := 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g = 21\ psf$

calculate balanced snow load $p_s := C_s \cdot p_f = 21\ psf$

snow density $\gamma_s := 0.13 \cdot (25) + 14 = 17.25$

$\gamma_s := 17.25\ pcf$

determine balanced snow height $h_b := \dfrac{p_s}{\gamma_s} = 1.217\ ft$

calculate drift height

clear distance from top of balanced snow load to top of obstruction...

...above insurance

parapet height $h_{ci\_para} := 14\ ft - h_b$

privacy wall height $h_{ci\_wall} := 12.83\ ft - h_b$

height to roof $h_{ci\_roof} := 12\ ft - h_b$

...above collector's shop

height to roof $h_{cc\_roof} := 13.5\ ft - h_b$

privacy wall height $h_{cc\_wall} := 10.5\ ft - h_b$

length of roof upwind of drift (in ft)...

...above insurance

upper roof $l_{ui\_up} := 68.5$

lower roof $l_{ui\_low} := 59$

...above collector's shop

upper roof $l_{uc\_up} := 57$

lower roof $l_{uc\_low} := 69$

determine drift height for balconies...

...above insurance

$$h_{di\_up} := 0.43 \cdot \left(\sqrt[3]{l_{ui\_up}} \cdot \sqrt[4]{25+10}\right) - 1.5 = 2.779$$

$$h_{di\_low} := 0.43 \cdot \left(\sqrt[3]{l_{ui\_low}} \cdot \sqrt[4]{25+10}\right) - 1.5 = 2.572$$

...above collector's shop

$$h_{dc\_up} := 0.43 \cdot \left(\sqrt[3]{l_{uc\_up}} \cdot \sqrt[4]{25+10}\right) - 1.5 = 2.525$$

$$h_{dc\_low} := 0.43 \cdot \left(\sqrt[3]{l_{uc\_low}} \cdot \sqrt[4]{25+10}\right) - 1.5 = 2.79$$

drift height for balconies $h_{d\_b} := 2.79\ ft$

calculate unbalanced load

$$p_{un\_b} := h_{d\_b} \cdot \gamma_s = 48.128\ psf$$

calculate width of unbalanced load

$$W_{un\_iNS} := \frac{8}{3} \cdot h_{d\_b} \cdot \sqrt[2]{s} = 23.527\ ft$$

convert loads into linearly distributed loads $DL_{cover} := 2.8\ \frac{lbf}{ft^2} \cdot t_b$

calculate dead load (plf) $w_{DL} := DL_{cover} + w_{joists} = 6.3\ \frac{lbf}{ft}$

calculate live load (plf) $w_{LL} := LL_{floors} \cdot t_b = 40\ \frac{lbf}{ft}$

calculate snow load (plf) $w_s := p_s \cdot t_b = 21\ \frac{lbf}{ft}$

calculate unbalanced snow load (plf) $w_{un\_b} := p_{un\_b} \cdot t_b = 48.128\ \frac{lbf}{ft}$

determine applied loads using load analysis program (ftool)

$w_{DL} = 6.3 \ \frac{lbf}{ft}$ $w_{LL} = 40 \ \frac{lbf}{ft}$

$w_{balconies} := w_{DL} + 0.75 \cdot w_{LL} + 0.75 \cdot (w_s + w_{un\_b}) = 88.146 \ \frac{lbf}{ft}$

Bending Moment Design

determine maximum applied bending moment

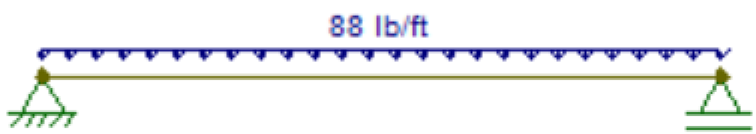


Figure 7.1.1 Distributed loading over joist

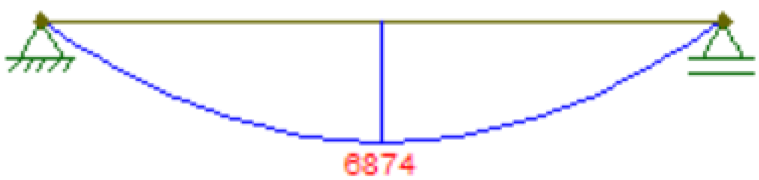


Figure 7.1.2 Bending moment diagram for joist

max moment developed in joist $M := 6874 \ lbf \cdot ft$

compare max moment developed from loading to max resistive moment in section

design equation $M_{all} \geq M = 1$

$DCR := \frac{M}{M_{all}} = 0.818$ joists satisfy moment design

Shear Design

determine applied vertical shear

$w_{balconies} = 88.146\ plf$ $l = 25\ ft$

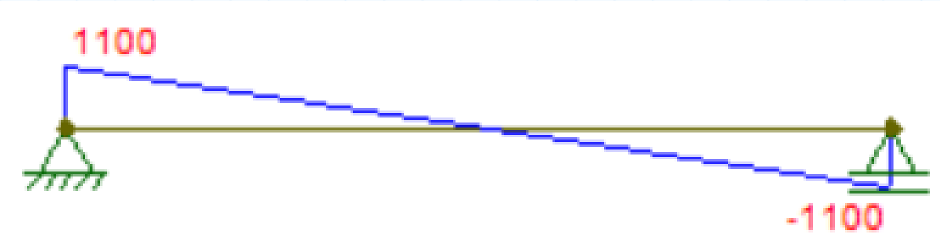


Figure 7.1.3 Shear force diagram for 25' joists

max shear developed in joists $V := 1100\ lbf$

compare max shear developed from loading to max resistive shear

Design Equation: $V_{all} \geq V = 1$

$DCR := \frac{V}{V_{all}} = 0.502$ joists satisfy shear design

Deflection Design

determine appropriate short and long term loads

short term load $w_{st} := 0.5 \cdot \left(LL_{floors} + p_s + p_{un\_b}\right) \cdot t_b = 54.564\ plf$

long term load $w_{lt} := w_{DL} + \left(0.5 \cdot \left(LL_{floors}\right)\right) \cdot t_b = 26.3\ plf$

apply load combination for total deflection

$w_{tot} := 1.5 \cdot w_{lt} + 0.5 \cdot w_{st} = 66.732\ plf$

check short-term deflection...

$w_{st} = 54.564 \frac{lbf}{ft}$

...using equations

$$\frac{22.5 \cdot 54.5 \cdot 25^4}{830000000} + \frac{2.67 \cdot 54.5 \cdot 25^2}{16 \cdot 10^5} = 0.634$$

$\delta_{st} := 0.634 \; in$

$\Delta_{st} := \frac{l}{360} = 0.833 \; in$

...using load analysis program (ftool)

Figure 7.1.4 Distributed load for short-term deflection

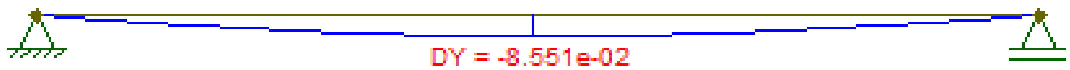


Figure 7.1.5 Diagram showing short-term deflection

short term deflection results
from load analysis

check total deflection

...using equations

$$\frac{22.5 \cdot 26.3 \cdot 25^4}{830000000} + \frac{2.67 \cdot 26.3 \cdot 25^2}{16 \cdot 10^5} = 0.306$$

$\delta_{lt} := 0.306 \; in$

$\delta_{tot} := 1.5 \cdot \delta_{lt} + \delta_{st} = 1.093 \; in$

$\Delta_{tot} := \frac{l}{240} = 1.25 \; in$

...using load analysis program (ftool)

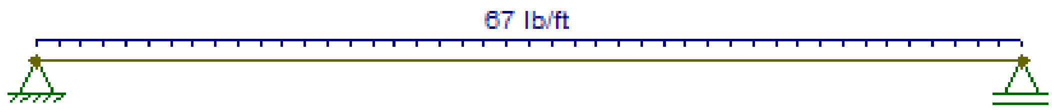


Figure 7.1.6 Distributed load for total deflection

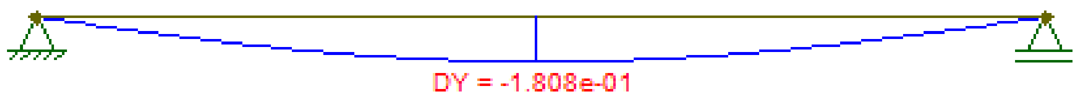


Figure 7.1.7 Diagram showing total deflection

Design Equation: $\Delta_{tot} \geq \delta_{tot} = 1$

DCR: $\frac{\delta_{tot}}{\Delta_{tot}} = 0.874$ deflection requirements check out

Bearing Design

bearing length parallel to grain $l_b := 1.75\ in$

bearing breadth perpendicular to grain $b_b := b_f = 2.313\ in$

define bearing area $A_b := l_b \cdot b_b = 4.047\ in^2$

define bearing area factor $C_b := \frac{l_b + 0.375\ in}{l_b} = 1.214$

determine reaction force at column P

length of longest span beam $l = 25\ ft$

$P := 0.5 \cdot l \cdot w_{balconies} = \left(1.102 \cdot 10^3\right)\ lbf$

Design Equation: $P_{all} \geq P = 0$

$DCR := \frac{P}{P_{all}} = 1.02$ joists satisfy minimum bearing design

Design Summary

Joists were designed to resist moment, shear, and deflection; the corresponding design equations are shown below:

moment $M_{all} = \left(8.405 \cdot 10^3\right)\ lbf \cdot ft$ $M = \left(6.874 \cdot 10^3\right)\ lbf \cdot ft$

$$DCR := \frac{M}{M_{all}} = 0.818$$

shear $V_{all} = \left(2.19 \cdot 10^3\right)\ lbf$ $V = \left(1.1 \cdot 10^3\right)\ lbf$

$$DCR := \frac{V}{V_{all}} = 0.502$$

deflection $\Delta_{st} = 0.833\ in$ $\delta_{st} = 0.634\ in$

$$DCR := \frac{\delta_{st}}{\Delta_{st}} = 0.761$$

$\Delta_{tot} = 1.25\ in$ $\delta_{tot} = 1.093\ in$

$$DCR := \frac{\delta_{tot}}{\Delta_{tot}} = 0.874$$

bearing $P_{all} = \left(1.08 \cdot 10^3\right)\ ft^2 \cdot psf$ $P = \left(1.102 \cdot 10^3\right)\ ft^2 \cdot psf$

$$DCR := \frac{P}{P_{all}} = 1.02$$

VII-ii. Built-up Beam Design

reference design values

specific gravity of SPF	$SG := 0.42$
modulus of elasticity	$E := 1400000\ psi$
	$E_{min} := 510000\ psi$
bending strength	$F_b := 875\ psi$
tension parallel to grain	$F_t := 450\ psi$
shear parallel to grain	$F_v := 135\ psi$
compression perpendicular to grain	$F_{c\_p} := 425\ psi$
compression parallel to grain	$F_c := 1150\ psi$

section properties of (3) 3x14 timbers

length	$l := 25\ ft$
depth	$d := 13.25\ in$
breadth	$b := 2.5\ in$
cross sectional area	$A := d \cdot b = 33.125\ in^2$
tributary width	$t_b := 1\ ft$
section modulus	$S := 73.15\ in^3$
moment of inertia	$I := 484.6\ in^4$
weight in plf	$w_{beams} := 6\ \frac{lbf}{ft}$

determine applicable adjustment factors

load duration factor $C_D := 0.9$

wet service factor $C_M := 1.0$

temperature factor $C_t := 1.0$

size factor...

...for bending $C_{F\_b} := 0.9$ ...for tension $C_{F\_t} := 0.9$ ...for compression (parallel to grain) $C_{F\_c} := 0.9$

flat use factor $C_{fu} := 1.0$

incising factor $C_i := 1.0$

repetitive member factor $C_r := 1.0$

buckling stiffness factor $C_T := 1.0$

bearing area factor $C_b := 1.0$

beam stability factor (CL)

determine effective length of joist $\frac{l}{d} = 22.642$ $\frac{l}{d} \ge 7 = 1$

$l_e := 1.63 \cdot l + 3 \cdot d = 44.063 \; ft$

calculate slenderness ratio

$R_B := \sqrt[2]{\frac{l_e \cdot d}{b^2}} = 33.481$ $R_B < 50 = 1$

calculate reference bending design value

$F_{b\#} := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_{F\_b} \cdot C_i \cdot C_r$ $E_{min}' := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T = \left(5.1 \cdot 10^5\right) \; psi$

$F_{bE} := \frac{1.2 \cdot E_{min}'}{{R_B}^2} = \left(7.862 \cdot 10^4\right) \; psf$

calculate beam stability factor

$$C_L := \frac{1 + \left(\frac{F_{bE}}{F_{b\#}}\right)}{1.9} - \sqrt[2]{\left(\frac{1 + \left(\frac{F_{bE}}{F_{b\#}}\right)}{1.9}\right)^2 - \frac{\left(\frac{F_{bE}}{F_{b\#}}\right)}{0.95}} = 0.692 \qquad C_L := 1.0$$

Bending Moment Design

bending strength of No. 2 SPF $F_b = 875 \; psi$

calculate adjusted bending strength $F_b' := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_{F\_b} \cdot C_{fu} \cdot C_i \cdot C_r$

convert loads into linearly distributed loads

calculate wall load (plf) $w_{wall} := DL_{brick} \cdot t_b = 115 \; \frac{lbf}{ft}$

calculate dead load (plf) $w_{DL} := w_{wall} + 3 \cdot w_{beams} = 133 \; \frac{lbf}{ft}$

*note: this beam only supports weight of brick wall; use load combination 1

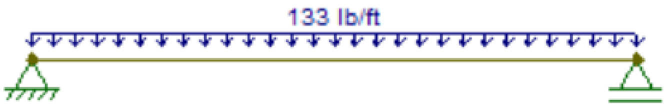


Figure 7.2.1 Distributed loading over joist

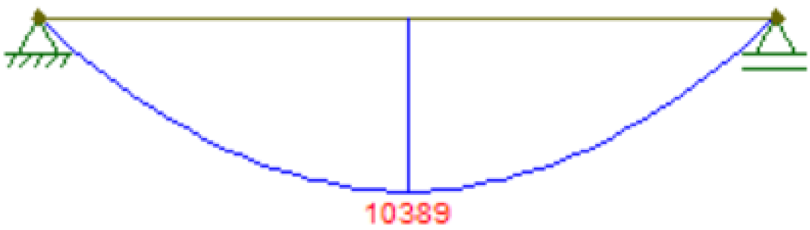


Figure 7.2.2 Bending moment diagram for joist

max moment in joist $M := 10389\ lbf \cdot ft$

distribute moment over 3 members $M := \frac{M}{3} = \left(3.463 \cdot 10^{3}\right)\ lbf \cdot ft$

determine applied bending stress using section modulus $f_b := \frac{M}{S} = 568.093\ psi$

compare applied stress to beam strength $F_b' = 708.75\ psi$

$DCR := \frac{f_b}{F_b'} = 0.802$ beam design checks out

Shear Design

estimate Fv' using applicable adjustment factors

$$F_v' := F_v \cdot C_D \cdot C_M \cdot C_t \cdot C_i = \left(1.75 \cdot 10^{4}\right)\ psf$$

determine applied shear stress fv

$w_{balconies} = 88.146\ plf$ $l = 25\ ft$

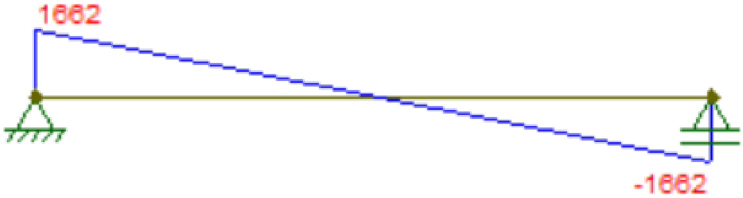


Figure 7.2.3 Shear force diagram for 25' beams

$V := 1662\ lbf$ $V := \frac{V}{3} = 554\ lbf$

determine applied shear stress parallel to grain

$f_v := \frac{3 \cdot V}{2 \cdot b \cdot d} = \left(3.612 \cdot 10^3\right)\ psf$

Design Equation: $f_v \le F_v' = 1$

DCR: $\frac{f_v}{F_v'} = 0.206$ joists satisfy shear design

Deflection Design

determine appropriate short and long term loads

*no short-term loading $w_{st} := 0\ plf$

long term load $w_{lt} := w_{DL} = 133\ plf$

apply load combination for total deflection

$w_{tot} := 1.5 \cdot w_{lt} + 0.5 \cdot w_{st} = 199.5\ plf$ $\frac{w_{tot}}{3} = 66.5\ plf$

check total deflection

...using equations

$\delta_{lt} := \frac{5 \cdot w_{lt} \cdot l^4}{384 \cdot E \cdot I} = 1.723\ in$ $\delta_{tot} := 1.5 \cdot \delta_{lt} = 2.584\ in$ $\Delta_{tot} := \frac{l}{240} = 1.25\ in$

...using load analysis program (ftool)

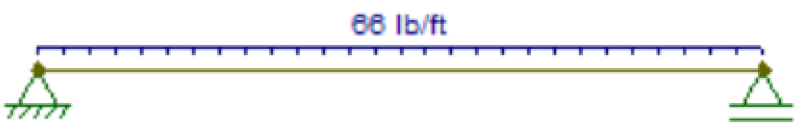


Figure 7.2.4. Distributed load for total deflection

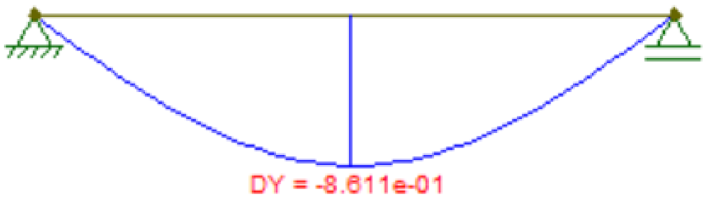


Figure 7.2.5 Diagram showing total deflection

$\delta_{tot} := 0.8611\ in$

Design Equation: $\Delta_{tot} \geq \delta_{tot} = 1$ deflection requirements check out

DCR: $\frac{\delta_{tot}}{\Delta_{tot}} = 0.689$

Bearing Design

bearing length parallel to grain $l_b := 4\ in$

bearing breadth perpendicular to grain $b_b := b = 2.5\ in$

define bearing area $A_b := l_b \cdot b_b = 10\ in^2$

define bearing area factor $C_b := \frac{l_b + 0.375\ in}{l_b} = 1.094$

determine reaction force at column P

length of longest span beam $l = 25\ ft$

$P := 0.5 \cdot l \cdot w_{DL} = \left(1.663 \cdot 10^3\right)\ lbf$

$f_{c\_p} := \frac{P}{A_b} = \left(2.394 \cdot 10^4\right)\ psf$

apply appropriate adjustment factors to design values

$F_{c\_p} = 425\ psi$

$F_{c\_p}' := F_{c\_p} \cdot C_M \cdot C_t \cdot C_i \cdot C_b = \left(6.694 \cdot 10^4\right)\ psf$

ensure bearing is satisfied

Design Equation: $F_{c\_p}' \geq f_{c\_p} = 1$

$DCR := \frac{f_{c\_p}}{F_{c\_p}'} = 0.358$ bearing satisfies design

Design Summary

Joists were designed to resist moment, shear, and deflection; the corresponding design equations are shown below:

moment $F_b' = (1.021 \cdot 10^5)\ psf$ $f_b = (8.181 \cdot 10^4)\ psf$

$DCR := \frac{f_b}{F_b'} = 0.802$

shear $F_v' = (1.75 \cdot 10^4)\ psf$ $f_v = (3.612 \cdot 10^3)\ psf$

$DCR := \frac{f_v}{F_v'} = 0.206$

deflection $\Delta_{st} = 0.833\ in$ $\delta_{st} = 0.634\ in$

$DCR := \frac{\delta_{st}}{\Delta_{st}} = 0.761$

$\Delta_{tot} = 1.25\ in$ $\delta_{tot} = 0.861\ in$

$DCR := \frac{\delta_{tot}}{\Delta_{tot}} = 0.689$

bearing $F_{c\_p}' = (6.694 \cdot 10^4)\ psf$ $f_{c\_p} = (2.394 \cdot 10^4)\ psf$

$DCR := \frac{f_{c\_p}}{F_{c\_p}'} = 0.358$

VIII. Garage Design

VIII-i. Header Design

reference design values for 2.0E grade LVL header

specific gravity of SPF $SG := 0.5$

modulus of elasticity $E := 2000000$

$E_{min} := 1016535\ psi$

bending strength $F_b := 2600\ psi$

tension parallel to grain $F_t := 1895\ psi$

shear parallel to grain $F_v := 285\ psi$

compression perpendicular to grain $F_{c\_p} := 750\ psi$

compression parallel to grain $F_c := 2510\ psi$

section properties of LVL header

length $l := 19\ ft$

depth $d := 9.25\ in$

breadth $b := 3.5\ in$

cross sectional area $A := d \cdot b = 32.375\ in^2$

tributary width $t_b := 1\ ft$

section modulus $S := \frac{b \cdot d^2}{6} = 49.911\ in^3$

moment of inertia $I := 484.6\ in^4$

weight in plf $w_{head} := 5.7\ \frac{lbf}{ft}$

Allowable design properties

moment $M_{all} := 8070\ \frac{lbf}{ft}$ shear $V_{all} := 3740\ lbf$

determine applicable adjustment factors

load duration factor $C_D := 0.9$

wet service factor $C_M := 1.0$

temperature factor $C_t := 1.0$

size factor...

...for bending $C_{F\_b} := 1.0$ ...for tension $C_{F\_t} := 1.0$ ...for compression (parallel to grain) $C_{F\_c} := 1.0$

flat use factor $C_{fu} := 1.0$

incising factor $C_i := 1.0$

repetitive member factor $C_r := 1.0$

buckling stiffness factor $C_T := 1.0$

bearing area factor $C_b := 1.0$

beam stability factor (CL)

determine effective length of joist $\frac{l}{d} = 24.649$ $\frac{l}{d} \ge 7 = 1$

$l_e := 1.63 \cdot l + 3 \cdot d = 33.283 \ ft$

calculate slenderness ratio

$R_B := \sqrt[2]{\frac{l_e \cdot d}{b^2}} = 17.366$ $R_B < 50 = 1$

calculate reference bending design value

$F_{b\#} := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_{F\_b} \cdot C_i \cdot C_r$ $E_{min}' := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T = \left(1.017 \cdot 10^6\right) \ psi$

$F_{bE} := \frac{1.2 \cdot E_{min}'}{{R_B}^2} = \left(5.825 \cdot 10^5\right) \ psf$

calculate beam stability factor

$$C_L := \frac{1 + \left(\frac{F_{bE}}{F_{b\#}}\right)}{1.9} - \sqrt[2]{\left(\frac{1 + \left(\frac{F_{bE}}{F_{b\#}}\right)}{1.9}\right)^2 - \frac{\left(\frac{F_{bE}}{F_{b\#}}\right)}{0.95}} = 0.943$$

Bending Moment Design

bending strength of No. 2 SPF $F_b = \left(2.6 \cdot 10^3\right) \ psi$

calculate adjusted bending strength $F_b' := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_{F\_b} \cdot C_{fu} \cdot C_i \cdot C_r$

convert loads into linearly distributed loads

calculate wall load (plf) $w_{wall} := DL_{brick} \cdot t_b = 115 \ \frac{lbf}{ft}$

calculate dead load (plf) $w_{DL} := w_{wall} + w_{head} = 120.7 \frac{lbf}{ft}$

*note: this beam only supports weight of brick wall; use load combination 1

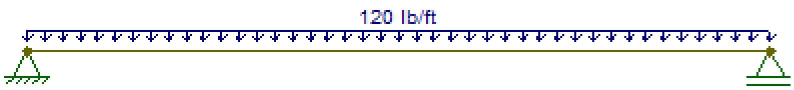


Figure 7.2.1 Distributed loading over header

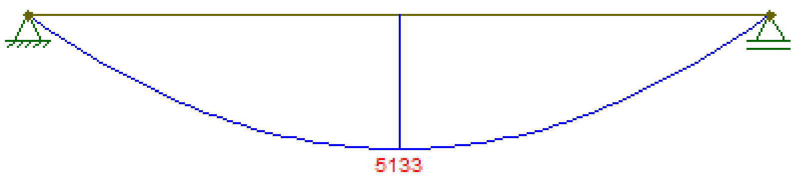


Figure 7.2.2 Bending moment diagram for header

max moment in headers $M := 5133\ lbf \cdot ft$

determine applied bending stress using section modulus $f_b := \frac{M}{S} = \left(1.234 \cdot 10^3\right)\ psi$

compare applied stress to beam strength $F_b{}' = \left(2.207 \cdot 10^3\right)\ psi$

$DCR := \frac{f_b}{F_b{}'} = 0.559$ beam design checks out

Shear Design

estimate Fv' using applicable adjustment factors

$F_v{}' := F_v \cdot C_D \cdot C_M \cdot C_t \cdot C_i = \left(3.694 \cdot 10^4\right)\ psf$

determine applied shear stress fv

$w_{balconies} = 88.146\ plf$ $l = 19\ ft$

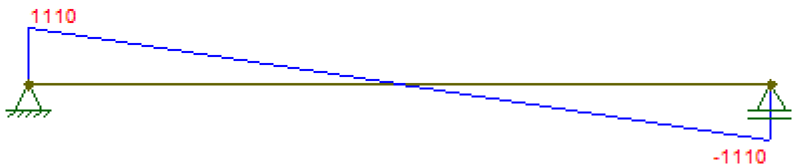


Figure 7.2.3 Shear force diagram for 19' headers

$V := 1110\ lbf$

determine applied shear stress parallel to grain

$$f_v := \frac{3 \cdot V}{2 \cdot b \cdot d} = \left(7.406 \cdot 10^3\right)\ psf$$

Design Equation: $f_v \le F_v{}' = 1$

DCR: $\frac{f_v}{F_v{}'} = 0.201$ joists satisfy shear design

Deflection Design

determine appropriate short and long term loads

*no short-term loading $w_{st} := 0\ plf$

long term load $w_{lt} := w_{DL} = 120.7\ plf$

apply load combination for total deflection

$$w_{tot} := 1.5 \cdot w_{lt} + 0.5 \cdot w_{st} = 181.05\ plf$$

check total deflection

...using equations

$$\delta_{tot} := \frac{270 \cdot 120.7 \cdot 18.5^4}{2000000 \cdot 3.5 \cdot 11.25^3} + \frac{28.8 \cdot 120.7 \cdot 18.5^2}{2000000 \cdot 3.5 \cdot 11.25} = 0.398 \qquad \Delta_{tot} := \frac{l}{240} = 0.95\ in$$

$\delta_{tot} := 0.398\ in$

...using load analysis program (ftool)

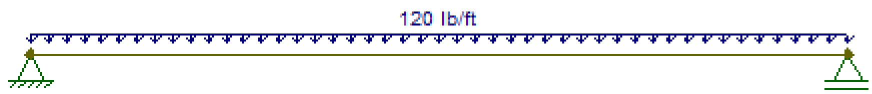


Figure 7.2.4. Distributed load for total deflection

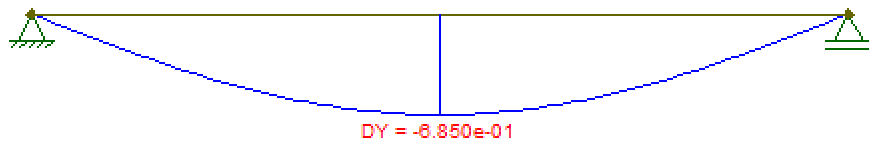


Figure 7.2.5 Diagram showing total deflection

$\delta_{tot} := 0.685\ in$

Design Equation: $\Delta_{tot} \geq \delta_{tot} = 1$ deflection requirements check out

DCR: $\frac{\delta_{tot}}{\Delta_{tot}} = 0.721$

Bearing Design

bearing length parallel to grain $l_b := 3\ in$

bearing breadth perpendicular to grain $b_b := b = 3.5\ in$

define bearing area $A_b := l_b \cdot b_b = 10.5\ in^2$

define bearing area factor $C_b := \frac{l_b + 0.375\ in}{l_b} = 1.125$

determine reaction force at column P

length of longest span beam $l = 19\ ft$

$P := 0.5 \cdot l \cdot w_{DL} = \left(1.147 \cdot 10^3\right)\ lbf$

$f_{c\_p} := \frac{P}{A_b} = 109.205\ psi$

apply appropriate adjustment factors to design values

$F_{c\_p} = 750\ psi$

$F_{c\_p}' := F_{c\_p} \cdot C_M \cdot C_t \cdot C_i \cdot C_b = 843.75\ psi$

ensure bearing is satisfied

Design Equation: $F_{c\_p}' \geq f_{c\_p} = 1$

$DCR := \dfrac{f_{c\_p}}{F_{c\_p}'} = 0.129$ bearing area satisfies design

Design Summary

Headers were designed to resist moment, shear, and deflection; the corresponding design equations are shown below:

moment $F_b' = \left(3.179 \cdot 10^5\right)\ psf$ $f_b = \left(1.777 \cdot 10^5\right)\ psf$

$DCR := \dfrac{f_b}{F_b'} = 0.559$

shear $F_v' = \left(3.694 \cdot 10^4\right)\ psf$ $f_v = \left(7.406 \cdot 10^3\right)\ psf$

$DCR := \dfrac{f_v}{F_v'} = 0.201$

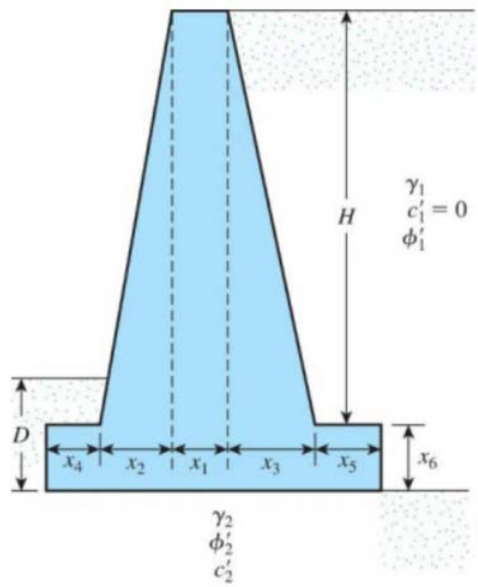
deflection $\Delta_{tot} = 0.95\ in$ $\delta_{tot} = 0.685\ in$

$DCR := \dfrac{\delta_{tot}}{\Delta_{tot}} = 0.721$

bearing $F_{c\_p}' = \left(1.215 \cdot 10^5\right)\ psf$ $f_{c\_p} = \left(1.573 \cdot 10^4\right)\ psf$

$DCR := \dfrac{f_{c\_p}}{F_{c\_p}'} = 0.129$

VIII-ii. Foundation Retaining Wall Design



Assumptions
-car load on concrete slab is a uniform 40 psf
-in situ soil has unit weight of 130 pcf
-active earth pressure coefficient for backfill Ka=0.33
-allowable bearing pressure $q_{all} := 1500\ psf \cdot \frac{1}{1\ ft}$

Define Variables

unit weight of concrete $\gamma_c := 150\ pcf$

wall dimensions

$H := 5\ ft + 10\ in$ $x_1 := 1\ ft$ $x_2 := 1\ ft$ $x_3 := 1\ ft$

$x_4 := 1\ ft$ $x_5 := 1\ ft$ $x_6 := 1\ ft$

$D := 0\ ft$ $\beta := 0\ deg$ $B := x_4 + x_2 + x_1 + x_3 + x_5 = 5\ ft$

slab dimensions $w_{slab} := 24\ ft$ $l_{slab} := 24\ ft$ $t_{slab} := 4\ in$

surcharge loads $q_{car} := 40\ psf$ $q_{slab} := t_{slab} \cdot \gamma_c = 50\ psf$

$w_{surcharge} := (q_{slab} + q_{car}) \cdot 1\ ft = 90\ psf \cdot ft$

soil properties

in situ soil $\gamma_s := 130\ pcf$ $\gamma_{sat} := 135\ pcf$ $\mu_s := 0.35$ $c_a := 130\ psf$

lateral earth pressure $\sigma_p' := 100\ psf$

backfill material (granular) $\gamma_b := 120\ pcf$ $K_a := 0.33$ $\delta_b := 25\ deg$

distance to ground water table $z_w := 3.5\ ft$

metallic strip properties

vertical spacing $S_v := 1.5\ ft$ horizontal spacing $S_h := 3\ ft$

strip width $w := 6\ in$ strength of steel $F_y := 36\ ksi$

Design Calculations

$H_w := x_6 + H = 6.833\ ft$ $\alpha := 0 = 0\ deg$

determine active earth pressure using Coulomb's theory

$K_a = 0.33$

$P_a := \left(\frac{1}{2} \cdot \gamma_b \cdot H_w^{\ 2} + w_{surcharge}\right) \cdot K_a = 0.954\ \frac{kip}{ft}$ $z_a := \frac{H_w}{3} = 2.278\ ft$

find x and y components of active earth pressure Pa

$P_h := P_a = 0.954\ \frac{kip}{ft}$

determine passive earth pressure using Coulomb's theory

$K_p := 0.6$

$P_p := \frac{1}{2} \cdot \sigma_p{}' \cdot x_6 = 0.05\ \frac{kip}{ft}$

determine weights and moment arms

footing $w_1 := B \cdot x_6 \cdot \gamma_c = 0.75\ \frac{kip}{ft}$ $x\_1 := \frac{B}{2} = 2.5\ ft$

backfill load resting on footing $w_2 := x_5 \cdot H \cdot \gamma_b = 0.7\ \frac{kip}{ft}$ $x\_2 := B - \frac{x_5}{2} = 4.5\ ft$

backfill load resting on cambered wall $w_3 := \frac{1}{2} \cdot x_3 \cdot H \cdot \gamma_b = 0.35\ \frac{kip}{ft}$ $x\_3 := B - x_5 - \left(\frac{1}{3} \cdot x_3\right) = 3.667\ ft$

weight of cambered section of wall $w_4 := 0.5 \cdot x_3 \cdot H \cdot \gamma_c = 0.438\ \frac{kip}{ft}$ $x\_4 := B - x_5 - \left(\frac{2}{3} \cdot x_3\right) = 3.333\ ft$

weight of rectangular section of wall $w_5 := x_1 \cdot H \cdot \gamma_c = 0.875 \ \frac{kip}{ft}$ $x_{\_5} := x_4 + x_2 + 0.5 \cdot x_1 = 2.5 \ ft$

weight of cambered section facing away from backfill $w_6 := 0.5 \cdot x_2 \cdot H \cdot \gamma_c = 0.438 \ \frac{kip}{ft}$ $x_{\_6} := x_4 + \frac{2}{3} \cdot x_2 = 1.667 \ ft$

determine FSo (overturning)

$M_r := w_1 \cdot x_{\_1} + w_2 \cdot x_{\_2} + w_3 \cdot x_{\_3} + w_4 \cdot x_{\_4} + w_5 \cdot x_{\_5} + w_6 \cdot x_{\_6} = 10.683 \ kip$

$M_o := P_h \cdot z_a = 2.174 \ kip$

$FS_o := \frac{M_r}{M_o} = 4.915$ okay (FS>=2)

check for uplift

$\Sigma P := w_1 + w_2 + w_3 + w_4 + w_5 + w_6 = 3.55 \ \frac{kip}{ft}$

$M_{net} := M_r - M_o = 8.51 \ \frac{kip \cdot ft}{ft}$ $x_{\_R} := \frac{M_{net}}{\Sigma P} = 2.397 \ ft$

$e_{cc} := \left| \frac{B}{2} - x_{\_R} \right| = 0.103 \ ft$ $\frac{B}{6} > e_{cc} = 1$ no uplift

check for bearing capacity

determine applied stress due to gravity wall

$q_P := \frac{\Sigma P}{B \cdot 1 \ ft} = 710 \ psf \cdot \frac{1}{ft}$

determine stress due to eccentrically loaded footing

determine distance from neutral axis to farthest point $c := \frac{B}{2} = 2.5 \ ft$

determine moment of inertia of section $I := \frac{1 \ ft \cdot B^3}{12} = 10.417 \ ft^4$

determine max stress due to overturning moment

$$q_{ecc} := \frac{M_o \cdot c}{I} = 521.657\ psf \cdot \frac{1}{ft}$$

determine maximum applied bearing stress

$$q_{max} := q_P + q_{ecc} = \left(1.232 \cdot 10^3\right)\ psf \cdot \frac{1}{ft}$$

determine required dimensions of metallic strip reinforcement

determine maximum active earth pressure

$$\sigma_{a\_max} := \left(\left(5 \cdot S_v + 0.25\ ft\right) \cdot \gamma_b\right) \cdot K_a = 306.9\ psf$$

calculate thickness of steel strip t

$$T_z := \sigma_{a\_max} \cdot S_v \cdot S_h = 1.381\ kip \qquad FS_B := 1$$

$$T_{all} := T_z \cdot FS_B = 1.381\ kip$$

determine reduction factors

corrision

$$RF_{corrosion} := 1.1$$

steel

$$RF_{steel} := 1.67$$

$$t := \frac{T_{all} \cdot RF_{corrosion} \cdot RF_{steel}}{w \cdot F_y} = 0.012\ in$$

minimum thickness

calculate effective length of steel strip L

length is constant over all the layers, thus use stress at greatest depth z=H

$$FS_P := 2 \qquad z := H$$

$$c_a = 130\ psf$$

Guess Values

$L := 12\ ft$

Constraints

$$FS_P = \frac{2 \cdot L \cdot w \cdot \sigma_{a\_max} \cdot tan(\delta_b)}{T_z}$$

Solver

$solve := \mathbf{find}(L) = 19.301\ ft$

$L := 19\ ft + 4\ in$ minimum length

calculate shear stress in wall

$F_{max} := (B + L) \cdot c_a + P_p = 3.213\ \frac{kip}{ft}$ $P_h = 0.954\ \frac{kip}{ft}$

$$FS_v := \frac{F_{max}}{P_h} = 3.367$$

Design Summary

Factors of Safety against overturning, sliding, and bearing capacity are, respectively:

$FS_o := \frac{M_r}{M_o} = 4.915$ $FS_o \geq 3 = 1$ okay

$FS_v := \frac{F_{max}}{P_h} = 3.367$ $FS_v \geq 2 = 1$ okay

$FS_q := \frac{q_{all}}{q_{max}} = 1.218$ $FS_q \geq 1 = 1$ okay

IX. APPENDIX B: DETAILED COST ESTIMATE

239 5th Ave. S., Clinton, IA 52732 Building Rehabilitation Cost Estimate						
Earthwork						
Description of Work	Amount	Units	Material	Labor	Equipment	Total
Trench Excavation	16	LF	$ -	$ 1.61	$ 0.54	$ 34.40
Backfill Trenches	246.6	CY	$ -	$ 10.20	$ -	$ 2,515.32
Earthwork Total Estimate						$2,500.00
Existing Conditions						
Description of Work	Amount	Units	Material	Labor	Equipment	Total
Removal of Existing Doors in Masonry	3	EA	$ -	$ 93.30	$ -	$ 279.90
Removal of Existing Doors in a Wood Frame	11	SF	$ -	$ 20.40	$ -	$ 224.40
Window Removal	431.85	SF	$ -	$ 3.54	$ -	$ 1,528.75
Partition Wall Demolition	1692.61	SF	$ -	$ 1.14	$ -	$ 1,929.58
Masonry Wall Demolition	557.43	SF	$ -	$ 1.63	$ 1.15	$ 1,549.66
Floor Joist Removal (2 x 12)	1512	SF	$ -	$ 0.93	$ -	$ 1,406.16
Floor Joist Removal (3 x 12)	288	SF	$ -	$ 0.93	$ -	$ 361.58
Column and Footing Demolition	13.5	CF	$ -	$ 8.28	$ 4.49	$ 172.40
Wood Column Removal	54	SF	$ -	$ 5.00	$ -	$ 378.00
Asbestos removal, subcontract		LS	$ -	$ -	$ -	$ 2,016.00
Debris Removal - Trash chutes 36" diameter		LS	$ -	$ -	$ -	$ 1,735.00
Removal of Boilers	2	EA	$ -	$ 3,125.00	$ -	$ 6,250.00
Mold/Mildew Cleanup		LS				$ 2,226.00
General Trash/Debris Clean-up	8409.3	SF	$ -	$ 0.70	$ -	$ 5,886.51
Existing Conditions Total Estimate						$26,000.00
Concrete						
Description of Work	Amount	Units	Material	Labor	Equipment	Total
Slab on Grade Concrete	33.64	CY	$ 122.00	$ 17.50	$ -	$ 4,692.37
Slab on Grade Assembly	33.64	CY	$ 236.00	$ 433.00	$ -	$ 22,503.17
Foundation Wall Cast-in-Place	0	LS	$ -	$ -	$ -	$ 1,749.80
Concrete Fill (4000 psi)	2.3	CY	$ 129.00	$ -	$ -	$ 296.70
Steel Wire Mesh 6" x 6" W2.9 x W2.9	936	SF	$ 0.50	$ 0.16	$ -	$ 617.76
Precast Wall	928.04	SF	$ 22.32	$ -	$ -	$ 20,713.85
Concrete Total Estimate						$60,000.00

Masonry						
Description of Work	Amount	Units	Material	Labor	Equipment	Total
Heavy brushing of Bricks	8823.63	SF	$ 0.07	$ 0.94	$ -	$ 8,911.87
Sandblasting	8823.63	SF	$ 0.42	$ 1.18	$ 0.51	$ 18,617.86
Waterblasting	8823.63	SF	$ -	$ 0.24	$ 0.03	$ 2,382.38
Repointing Bricks	2205.9075	SF	$ 0.10	$ 1.56	$ -	$ 3,661.81
Repointing Limestone	938.85	SF	$ 0.11	$ 1.56	$ -	$ 1,569.76
Cleaning of Heavily Carbonated Limestone	938.85	SF	$ 0.59	$ 1.84	$ -	$ 2,285.63
Exterior Material Removal from Collector's Shop	32.58	SF	$ -	$ 5.00	$ -	$ 244.35
Masonry Privacy Walls	131.079	SF	$ 6.05	$ 6.90	$ -	$ 2,546.21
Masonry Total Estimate						$40,200.00
Wood and Composites						
Description of Work	Amount	Units	Material	Labor	Equipment	Total
Douglas Fir 1" x 3" Flooring w/ Finish	4875	SF	$ 4.21	$ 3.60	$ -	$ 38,073.75
16" TJI/35 Floor Joists	312	SF	$ 4.67	$ 0.60	$ -	$ 1,644.24
(3) 3" x 14" Timbers	50	LF	$ 17.28	$ -	$ -	$ 864.00
1/2" OSB Structural Sheathing	0.312	MSF	$ 816.00	$ -	$ -	$ 254.59
Neoprene w/ Adhesive	3.12	SQ	$ 732.00	$ 103.00	$ -	$ 2,605.20
Finished Wood Flooring	312	SF	$ 0.29	$ 2.66	$ -	$ 920.40
Stairs 10 risers w/ handrail	1	EA	$ 1,890.00	$ 402.00	$ -	$ 2,292.00
Stairs 15 risers w/handrail	1	EA	$ 2,880.00	$ 469.00	$ -	$ 3,349.00
Landing Douglas Fir	13.75	SF	$ 4.02	$ 9.05	$ -	$ 179.71
Boxed Stair 3'0" wide	8	EA	$ 136.00	$ 10.90	$ -	$ 1,175.20
Hand rail	2	EA	$ 69.80	$ 8.61	$ -	$ 156.82
Deck Railing (Pressure-treated Add 50% forsizing and different lumber)	25	LF	$ 6.99	$ 11.20	$ -	$ 454.75
Laminate Flooring	410	SF	$ 0.25	$ 2.00	$ -	$ 922.50
Wood and Composites Total Estimate						$53,000.00
Thermal and Moisture Protection						
Description of Work	Amount	Units	Material	Labor	Equipment	Total
Insulation Board, 1" thick, R-5.0	1792.94	SF	$ 0.64	$ 1.07	$ -	$ 3,065.93
Thermal and Moisture Protection Total Estimate						$3,075.00

Openings						
Description of Work	Amount	Units	Material	Labor	Equipment	Total
15-Lite 3'0" x 7'0" Balcony Door	2	EA	$ 632.00	$ 37.00	$ -	$ 46,768.00
3'0" x 7'0" Exterior Metal Door	6	EA	$ 494.00	$ 35.30	$ -	$ 3,175.80
8'0" x 7'0" Raised Steel Panel Garage Door	2	EA	$ 1,723.40	$ 360.00	$ -	$ 4,166.80
3'0" x 7'0" Hardboard Face Flush Interior Door	3	EA	$ 90.84	$ 45.80	$ -	$ 409.92
2'9" x 7'0" Hardboard Face Flush Interior Door	2	EA	$ 94.63	$ 45.80	$ -	$ 280.85
3'0" x 7'0" 4-Panel Wood Interior Door	19	EA	$ 370.80	$ 42.60	$ -	$ 7,854.60
3'0" x 7'0" Double Sliding Closet Door	6	EA	$ 197.17	$ 38.86	$ -	$ 1,416.19
3'6" x 7'0" Doorway Opening	1	EA	$ 43.68	$ 37.20	$ -	$ 80.88
2'8" x 9'0" Single Hung Insulated Low-E Windows	3	EA	$ 206.70	$ 50.30	$ -	$ 771.00
3'6" x 9'0" Single Hung Insulated Low-E Windows	3	EA	$ 238.50	$ 50.30	$ -	$ 866.40
3'-3" x 9'0" Single Hung Insulated Low-E Windows	1	EA	$ 230.55	$ 50.30	$ -	$ 280.85
2'-9" x 4'0" Single Hung Insulated Low-E Windows	5	EA	$ 198.75	$ 50.30	$ -	$ 1,245.25
2'9" x 5'0" Single Hung Insulated Low-E Windows	3	EA	$ 182.85	$ 50.30	$ -	$ 699.45
3'0" x 3'0" Fixed Low-E Insulating glass windows	3	EA	$ 157.50	$ 18.50	$ -	$ 528.00
7'3" x 9'0" Window Casement Triple Transom	1	EA	$ 1,708.00	$ 134.00	$ -	$ 1,842.00
7'6" x 9'0" Window Casement Triple Transom.	1	EA	$ 1,769.00	$ 134.00	$ -	$ 1,903.00
Openings Total Estimate						$72,500.00
Plumbing Total Estimate						$50,000.00
HVAC Total Estimate						$43,000.00
Electrical Total Estimate						$73,000.00
Indirect Overhead Estimate						$32,000.00
Direct Overhead Estimate						$28,000.00
Building Inspection Total Estimate						$3,000.00
Total Construction Cost	$426,500.00					
Overhead (53% Indirect; 47% Direct)	$60,000.00					
Profit (Rate is 7.5%)	$32,000.00					
Contingencies (10%)	$42,650.00					
Total Estimate	$561,000.00					

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