Dutton Christian Elementary School Addition

Dutton Christian Elementary School

ADDITION AMBITION

Team #9

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Engineering 340-Senior Design Project

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Executive Summary

ADDITION AMBITION has been given the opportunity to help Dutton Christian Elementary School plan its next Addition. Dutton Elementary is a growing school and its needs include several classrooms, a large group gathering room, and a new teacher’s lounge (as well as a few other spaces). ADDITION AMBITION has designed an addition that meets these needs and created a scale model and walkthrough to present the design, as well as a plan set that included all the necessary architectural and structural drawings/details. The team focused on creating a building that was versatile, structurally safe, and economical to build.
I. Introduction

Dutton Christian Elementary school is a small Christian school located in Dutton, Michigan. The enrollment of the school has been increasing so an addition is desired. The objective of ADDITION AMBITION is to provide the school with a working set of structural and architectural drawings along with promotional material for the proposed addition of Dutton Christian Elementary. The addition has been designed to meet the needs of the current elementary school while promotional materials make it possible for the school to begin fund raising for the addition. It has been ADDITION AMBITION’S privilege to be involved with Dutton Christian and have the opportunity to use our gifts to benefit others.

II. Presenting the Challenge

A. Problem Statement

Currently Dutton Christian Elementary School is at maximum capacity. Growth of the school has forced classes to be held in two portables located off the east wing of the elementary building. With recent growth of the elementary school the portables can no longer accommodate the increase of students. Because the portables are separated from the rest of the building the student body seems to be split up and not as unified as it could be. Not only do the portables separate the student body, they require students and faculty to walk outdoors to get to and from the elementary building. For these reasons it has been decided to investigate the possibility of constructing an addition to the elementary school building. The addition would allow for the abandonment of the portables and the increasing amount of students.
B. Project Objective

The objective of the project is to provide the school with a working set of structural and architectural drawings along with promotional material for the proposed addition of Dutton Christian Elementary. The first objective includes a plan set containing Civil, Architectural, and Structural drawings. These drawings have been completed to a level which these elements of the building may be constructed if brought to the contractor.

The second objective involves the promotional aspects of the addition. Dutton Christian School celebrated its 50th year anniversary in April of 2005 and had requested that promotional materials be present at the celebration. At that time many of the people involved with the school were to be present. Dutton Christian School will need to raise a sufficient amount of money in order to build and furnish this addition. Advertising to the local community and parents of the school is a crucial step in raising enough funds to construct the addition. Advertising is also used to gain the support of the community involved with Dutton Christian School. Letting the public know the future plans of the school will motivate the local community and build support for construction. The promotional deliverables completed include a computer generated 3-D walkthrough, a poster, and a miniature model of the school including the addition.

A computerized 3-D walkthrough of the school provides a realistic feel for the size and design of the addition. This advertising technique will be very helpful to those who may not find visualizing 2-D drawings into 3-D objects. A miniature model of the school has been made of balsa wood and differentiates between the existing building and the addition. The model shows how the addition not only benefits but also complements
the existing building aesthetically. Posters are a silent advertisement and were intended
to grab the attention of the stakeholders of Dutton Christian at the 50th anniversary
celebration. The purpose of the poster is to grab people’s attention and draw them in
with the hope of sparking their interest and fueling motivation to become involved.

C. Design Norms Considered

Stewardship, caring, and trust are the three design norms which seemed to be
most pertinent to this project. When considering the design norm of stewardship our
team focused on keeping the design of the addition simple and far from extravagant. To
do this we focused on not using any excess materials such as over elaborate architectural
design. We also focused on designing the addition so that all the square footage of the
building would be versatile and put to good use. To do this we concentrated on designing
the structural components of the building so that the majority of the interior walls could
be relocated in the future, and not incorporating any entryways or spaces whose purpose
is not crucial.

The second design norm which we have followed is caring. Designing for this
design norm required our building layout to be environmentally friendly for its occupants
and the design must also promote safety. To design with the occupant in mind we
incorporated large windows in every one of the classrooms and teachers lounge. The
natural light will be less staining on student and teacher’s eyes as compared to depending
on 100% florescent lighting. To promote safety two fire exits are visible from the
majority of the multi-purpose room.

The third and final design norm considered in the design of the addition is trust.
Trust is crucial in the structural aspect of the design. The building needs to look and feel
sturdy. The factor of safety appropriate for an elementary school is greater than normal because children (who are our future) will be occupying the addition.

**D. Christian Perspective on Project**

Dutton Christian Elementary School addition is a project that is all about stewardship. Our group, ADDITION AMBITION, has used our engineering skills, skills that God has given us to help further his kingdom. We assisted a small Christian school in the design of a new addition which will save thousands of dollars. Also, as products of the Christian education system our group understands the building features that are valued in a Christian school. Mr. Terpstra says, “Form to follow function.” This means he wants a simple building that is cost effective (inexpensive) and yet is still aesthetically pleasing. Money is not given to Christian schools by the state, like public schools; therefore Christian schools must raise the money themselves, so a lower cost is extremely important to the success of the project.

It is important to integrate ones faith in ones work. Faith is not something that should be separated from everyday life and only thought about on Sunday. ADDITION AMBITION is attempting to integrate our faith in all aspects of our project. We wish to glorify God by being good stewards of our gifts and by helping to support Christian education which is extremely valuable.

**III. Present the Solution**

**A. Loads and Codes**

1. Introduction
The loads that have been used for the Dutton Christian Elementary School Addition are calculated from ASCE7 Minimum Design Loads for Buildings and Other Structures. The loads that have been taken into account are the dead, live, wind, and snow loads. These are the four loads that have a major impact on structures in West Michigan. The seismic load is not applicable in Michigan for it is not in a seismic area. It is also assumed that the rain load will be far less than the snow load because snow accumulates and rain does not (a rain load would be applicable in a state like Florida that does not have snow).

2. Dead Load

The dead loads used have been acquired from Design Works plan set. They used loads of 3 psf for membrane and insulation, 2 psf for steel decking, 2 psf for steel joist, 5 psf for mechanical and miscellaneous, and 3 psf for the ceiling and lights. This results in a total dead load of 15 psf. These can also be seen in Appendix A.1.

3. Live Load

The live load used has been calculated using formulas in the ASCE7 book. These calculations can be seen in Appendix A.2. The resulting live load used is 12 psf.

4. Snow Load

The snow load calculations can be seen in Appendix A.3. A few things of importance for the snow load calculations is that the ground snow load in West Michigan is 35 psf and that the addition has an importance factor of 1.1 because it is an elementary school. The snow load used is 26.95 psf.

The existing roof also has been analyzed. We needed to determine if the existing joists could support the additional snow load that the drifting due to the new addition
would create. It has been determined that the existing roof will experience an additional load of 421 pounds per foot of width along the edge of the existing and new addition. ADDITION AMBITION does not have the joist sizes of this section of the building and therefore is not able to determine if the existing joist will hold up the additional snow load. If the existing joists are going to fail, a plate can be welded onto the bottom of all the existing joists that will experience the additional snow load so that they will be strong enough.

5. Wind Load

The wind load calculations can be seen in Appendix A.4. There are many different wind load cases and the worst case scenario results in a wind load of -21.5 psf.

6. Total Factored Loads

The total factored load that the roof will experience is calculated in Appendix A.5. The loads above have been applied in all seven of the equations of LRFD. The greatest load is from equation three and it is 61.2 psf. We decided to use a total roof load of 65 psf for an additional factor of safety on top of the factor of safety from the load equations.

B. Joist Design

1. Joist Design Decisions

As with structural engineering in general, safety is paramount and we considered safety to be even more important because this is a school addition for children (who are our future). While code requirements basically require the building to be 10% stronger than the standard, our building exceeds this where appropriate and economically feasible. Minimum design loads were calculated (as outlined in another section of the report), but
by reverse engineering the old addition we found that it had been built with an even larger factor of safety which we then tried to match in our design.

While joist design is fairly standardized, there were some design decisions that differed from the norm. The joists that span the large group room in the center of the addition continue east over the hallway and storage area even though possible bearing walls exist to decrease the length of the span. The reasoning is as follows; first the increased span makes the whole center bay of the addition one joist type leading to quicker fabrication and construction. And second this increases the versatility of the space, the small group room could be removed and the storage room could be turned into an additional classroom, this would probably coincide with building a storage shed adjacent to the building. While these changes will likely lead to a marginal increase in materials cost, it should be offset by labor savings.

2. Joist Design

Joists have been sized to account for the weight of the rooftop units. This is done with the program from Vulcraft called “The Vulcraft Assistant.” The program knows the allowable shear envelope and moment capacity of the joists, and uses information given on the uniform load and point loads to determine the appropriate joist. These joist sizes were then used for the entire bay because of familiar reasons; the single joist type leads to quicker fabrication and construction, as well as increased safety because it is impossible to place the wrong joist beneath the rooftop units.

The roof is sloped at an angle of $\frac{1}{4}”$ per foot so as to ensure the water will flow towards the roof drains and reduce the risk of ponding. The elevations of the joists on
each end are designed to minimize the amount of built-up insulation required to achieve this $\frac{1}{4}$” per foot requirement.

Joist design is based on the loads calculated from ASCE7 and detailed in the “Loads and Codes” section of the report; the factored design load on the roof of the Dutton Christian Elementary School addition is 58 psf. The actual design load for the joists is less because of the differences between code requirements from ASCE7 and the Steel Joist Institute (SJI), making the design load about two thirds of the factored load. Our design exceeds these load requirements as stated in the design decisions section and the joist calculations can be seen in Appendix B.

C. Beam Design

The layout of the addition necessitated the use of a single I-beam in the hallway on the west side of the addition. The beam is sized based on the moment generated from a uniform load from the roof of 358 lb/ft and two 2.5 kip point loads caused by the two joists that rest on the beam. The maximum moment has been calculated as 21.7 ft-kips, and a W10x19 was sized from the LRFD Beam Design Moment tables for an un-braced length of 15 ft (the length of the beam). The detailed analysis of this beam can be found in Appendix C.

D. Bearing Plate Design

The bearing plates have been designed using the design procedure given in the LRFD code book. Due to the relatively small loads encountered because the spans are relatively small and the roof fairly light, most of the bearing plates were designed to meet practical requirements and not the code requirements of LRFD. The beam, for example, required an area of only 7 in$^2$ to bear safely on the CMU wall, this however, is obviously
not a practical area for the beam to be welded to and the plate be grouted into the wall. An 8x12 inch plate was called out on the plans because the 8 inch depth covers the whole CMU block, and the 12 inch width allows 4 inches on each side of the 4 inch wide beam for the ½ inch anchor bolts to fit with enough room for the steel worker to fasten the bolts. The joist bearing plates have been designed with a very similar procedure, though again practical requirements overshadowed code requirements. All of the bearing plate calculations can be seen in Appendix D.

**E. HVAC Rooftop Unit Considerations**

The size of the HVAC units is important for the sizing of the joists because a larger rooftop unit than anticipated would make the joists less safe. The joists were designed for rooftop units with a 7.5 ton (ton = a heat transfer unit) cooling capacity. Normally HVAC rooftop units would not have much of an effect on the joist sizes; however in our case much of the HVAC design was unknown so we have used a lot of worst case scenarios to design the joists to support the HVAC unit. Because of this the joist sizes are probably larger than necessary.

One possible way to reduce the cost of materials on the project is to not locate an HVAC unit on top of the large span above the group meeting room. This would reduce the size of the joists necessary and reduce costs. There would have to be two major considerations before considering this alternate. One is that the rooftop units should not be visible from a person walking relatively close to the building, and second that the cost of moving the rooftop unit and the extra ductwork that may require is not more expensive than simply increasing the size of the joists.

**F. Lintel Design**
1. Introduction

There are four different lintel types that are used for the Dutton Christian Elementary School Addition. Each lintel has been designed so that it will satisfy the Load and Resistance Factor Design manual. The base plates, supporting tubing that has been placed in the windows in order to decrease the lintel span and the channels have also been designed to satisfy the LRFD manual.

2. Design of Lintel Supports

The lintels over the windows require supporting tubing to decrease the span of the lintel. The teachers lounge windows span 25’4” and all the other windows span 10’0”. These spans are large and would require an enormous I-beam to support all the masonry above the windows if the lintel was forced to span the entire distance. Therefore, the window lintels have been designed with supporting tubing. The teachers lounge window has been designed with two HSS 2 ¼ X 2 ¼ X ¼ structural tubes. The spacing of the tubes is 8’0” from the edge of the window and 9’0” from each other. The details of the tubing can be seen in Appendix E. The classroom lintels have been designed with two HSS 2 X 2 X 1/8 structural tubes. The spacing of the tubes is 3’0” from the edge of the window and 3’0” from each other. The details of the tubing spacing can be seen in the attached plan set on page S6, D5.

3. Lintel Design

The lintels have been designed according to the deflection criteria because deflection criterion is the controlling element in the design. The deflection for glass or masonry must be less than L/600, where L is the length of the span. The addition has been designed with four different lintel types.
The first type of lintel is used for all of the windows in the school. It is made of a W8X10 I-beam, a ½” plate, and a C4X4.5 channel. All of these elements span the entire width of the window plus an addition 8” on each side of the window so that they bear on the masonry wall.

The second type of lintel is used for all of the interior doors located along the load bearing walls, as designated by the joist layout/foundation plan. This lintel is made of a W8X10 I-beam and a ½” base plate. These elements span the entire width of the door plus an additional 8” on each side of the door so that they bear on the masonry wall.

The third type of lintel is used for all of the doors that are not located along a load bearing wall and all of the closet doors. This lintel is made of two L3½X3 ½X¼ angle beams and a ¼” base plate. These elements span the entire width of the door plus an additional 8” on each side of the door so that they bear on the masonry wall.

The fourth type of lintel is used for the two exterior doors. This lintel is made of a W16X26 I-beam, a ½” base plate, and a C4X4.5 channel. These elements span the entire width of the door plus an additional 8” on each side of the door so that they bear on the masonry wall.

4. Lintel Channel Design Decision

The channel used in the lintel for the exterior doors and the windows has been designed so that it will fit in the gap of the exterior wall. The exterior wall has a 2” gap between the CMU block and the face brick. The proposed channel is 1.58” wide which means it is able to fit in the gap of the wall. The channel has been included in the design of the lintel because the bearing plate for a wall type like the proposed exterior wall have
G. Foundation Design

1. Introduction

There are three different foundation designs for the Dutton Christian Elementary School Addition. There is an exterior foundation, an interior load bearing wall foundation, and an interior non-load bearing wall foundation. All three of these foundations have been designed based on the loads calculated from ASCE7 which is detailed in the “Loads and Codes” section of this report and by the size of the joist members over each bay.

2. Footing Design

The footing width has been designed such that the load it distributes is less than the soil bearing capacity. The soil bearing capacity of 2250 lb/ft² has been acquired from Design Works plan set. If the load from the footing is greater than the soil bearing capacity, the soil will experience shear failure and the building will become unstable.

Three different footing widths have been used in the Dutton Christian Elementary School Addition. There is an exterior footing, an interior load bearing wall footing, and an interior non-load bearing wall footing. The loads taken into account for the foundations are the joist weight, the wall weight, the concrete block weight, and the roof load. The exterior wall has a maximum total factored load of 4.29 kip/ft. This leads to a required area of 2.04 ft²/ft of length. The footing width which has been used is 2’4”. This is larger than the required area so it will transfer a smaller load than the soil bearing capacity. Therefore the footing width is satisfactory. The reason the footing width is
much greater than the required width is that the footing must have a minimum of 6”
additional width on each side of the concrete block resting on it. This can be seen in
Appendix F and in the plan set on page S3, D3. There are two reasons the exterior walls
have only one foundation designed for them even though some of the exterior walls are
load bearing and some are not. The first reason is that the footing must have a minimum
of 6” of additional width extending past each side of the concrete block resting on it.
This would reduce the footing width by a maximum of 2” which is negligible for a
building of this small size. The second reason is for the ease of construction. Having
only one exterior foundation design speeds up the construction process for the workers do
not need to constantly adjust their foundation construction methods.

The interior load bearing walls have a maximum total factored load of 4.69 kip/ft.
This leads to a required area of 2.234 ft²/ft of length. The footing width which has been
used is 2’4”. This is larger than the required area so it will transfer a smaller load than
the soil bearing capacity. Therefore the footing width is satisfactory. The designed
interior load bearing wall footing has 10” addition width on each side of the concrete
block resting on it. This is greater than the required 6”. This can be seen in Appendix F
and in the plan set on page S3, D4. The reason the interior load bearing walls have only
one foundation designed for it is for ease of construction like the exterior walls.

The interior non-load bearing walls have a much lower total factored load and
therefore the foundation design is quite different. The standard foundation design, which
can be seen in the plan set on page S3-D2, has been used.

3. Reinforcing Bars
All of the footings have been reinforced with 2 #5 bars. The exterior walls and the interior load bearing walls use a spacing of 1’4” O.C. The interior non-load bearing walls use a spacing of 1’2” O.C.

The concrete block in the interior load bearing walls and the exterior walls has also been reinforced. The exterior foundation has been designed with 4 #5 bars 10” O.C. Two bars are in the top of the block and two bars are in the bottom of the block. The interior load bearing wall foundations have been designed with 4 #5 bars 5” O.C. Two bars are in the top of the block and two bars are in the bottom of the block.

All of the footings for the exterior walls and the interior load bearing walls have been connected to the concrete block by means of dowels. The dowels used are #4 dowels which are 1’4” long and 24” O.C. The masonry walls are all connected to the foundations/concrete blocks by means of dowels as well. The dowels used for this are #4 dowels which are 40” long. These dowels are spaced to match the wall reinforcement.

**H. Masonry Design**

1. **Design Decisions**

   The masonry design for Dutton Christian School is calculated mainly by using software “NCMA Masonry Design” and the calculations can be seen in Appendix G. The main consideration of designing masonry is the out of plane loads on the wall. These loads can include the dead, live, soil, fluid, wind, seismic, roof, rain, and snow load. The masonry design guideline for this project is based on 2002 MSJC Allowable Stress Design Code. The design requirement on this design code is compatible with both IBC and ASCE 7-98. Both UBC and MSJC codes have restriction and quality standards on
material, design, and construction. They establish limits on the type of units, control reinforcement, connectors, mortar, and grout types.

At the design section, the forces are calculated using the load data (wind load and dead load). Our team decided to use a reinforced masonry wall with a simply supported wall. The reinforced masonry wall is preferred to the un-reinforced wall as the compressive, flexural, and shearing stresses are quite high in our building design. The combination of size and placement of steel reinforcement is determined to give the most economical value. The steel reinforcement with either the close or wide spacing provide the same result to hold the load. However, the steel spaced too closely will slow the construction, and be more expensive than in the wider space as it requires more labor expense. The number 6 bars (Area = 0.44 square inch, Diameter = 0.75 inch) with 48 inches on the center appear to give the most economical value for this building construction. The steel grade 60 (fy = 60 ksi) is suitable for the design of this steel reinforcement.

2. Construction Design

The standard requirement in wall thickness for a one story building is 6 inches wide. To give a more solid and safe design, the wall thickness is designed to be 7.625 inches. The height of the wall is designed to be 208 inches which follows the wall’s height from the recent addition completed just over a year ago. 8 inches of Concrete Masonry Units (CMU) are used as the block for this building. The construction of the wall should be full grout with running bond. Running bond is preferred to stack bond as it gives more strength to the masonry design.

3. Masonry Properties
The CMU density is designed to be 115 pcf. The types of principal components of masonry mortar and grout are cement, lime, sand, and water. The mortar used for this building is Portland cement lime. Although this is a lengthy and complicated procedure, it gives more strength than using the other technique. The type N mortar is selected as it gives the best of compressive and flexural strength, workability, and economy. This type of mortar is usually known as a “medium strength” mortar.

The grout’s characteristic for this building is fine grout. The size of this type of grout is usually less than 3/8 inches. The calculation method for masonry strength is the prism test method. This method meets the purpose of our design to yield higher values and more economical design.

4. Load Design

There are two major loads that need to be considered for designing the masonry: the roof dead load and the wind load. Included in the roof dead load are the membrane, insulation, steel deck, steel joist, mechanical, ceiling, and lights for a total of 15 psf. The complete axial load calculation can be seen in Appendix G. The wind load is defined as the lateral load that spans at the wall height of 208 inches. As shown in the Appendix A.4, the designed wind load for this building is 15 psf.

5. Control Joint Locations

The most effective locations to put control joints are in the openings, offsets, and intersections. As our project has been designed to have concrete masonry walls, the control joints are required at the points of weakness such as abrupt changes in wall height and thickness, door or window openings, and uninterrupted walls. As our proposed design has many windows but no abrupt change on the walls’ height and thickness, the
control joints are mainly located on the edges of doors or window openings. The control joints are also placed periodically in uninterrupted walls. The standard requirement of 20 feet spacing between control joints is followed on this project.

I. Plan Set

1. Civil Design-Site Plan

To begin this project a site plan which shows existing contours, elevations, and services was needed. Exxel engineering had done previous work for Dutton Christian and had in their possession a plan showing these items. The site plan was obtained electronically and used to build a site plan for the proposed addition.

On the site plan, demolition of existing structures and the construction of proposed structures can be found. Ideally the plan would also show the location of the sanitary sewer tie in and size along with the location of the water service but this information is not available to our team. The location and elevation of the storm line is available to our team so a proposed storm line and tie in location are specified on Sheet C1.

2. Architectural Design

a. Demolition Plan

On sheet A1 the demolition plan for the proposed location of the addition can be found. This sheet has been constructed to give a clear message to the contractor as to what needs to be removed from the existing site to accommodate the proposed addition.

b. Elevations

Sheet A2 is constructed to give a clear view of the proposed addition from the north, east, south, and west. It is on this sheet where the control joints for the masonry
block are called out. It is important for the contractor to see the exact location of these
control joints as they are crucial in the masonry block design. A detail of the window and
door types can also be found on this sheet which aids in the masonry block wall opening
dimensions.

c. Floor Plan

Determining the size and design of the floor plan was the most daunting task for
our team. To finally settle on a layout many meetings were scheduled between the client
and ADDITION AMBITION. Because the floor plan was changing very often this sheet
has been redrawn many times. It is on Sheet A3 that room dimensions, window
locations, wall types and various other architectural criteria are called out. Four wall
types are used in the design of this addition. The first which is labeled 1A consists of
materials needed for the exterior walls of the building. These materials not only provide
insulation but are also for aesthetics. The second wall style is labeled B and is consistent
with the interior CMU block wall type. The third wall type was used at an interior wall
for the perimeter of the music room. This wall type includes insulation and drywall for
the purposes of sound deadening. The final wall type is similar to the exterior wall style,
but accommodates the interior characteristics of the music room.

d. Roof Plan

Sheet A4 shows a simple schematic of the slope of the roof to each roof drain.
For every roof drain there is an overflow drain. The number of roof drains required was
estimated using the existing building square footage divided by the number of existing
roof drains and then comparing that to our square footage. These roof drains collect rain
water and deliver it to the east side of the building where the storm sewer line ties in.
e. Miscellaneous Architectural Sheets

Sheet A5 contains details which show sections through the addition and sheets A6, A7, and A8 provide a floor, wall, and ceiling finish plan. Overall the finishes have been chosen to match the existing school. The carpet has been chosen to be a removable tile carpet such that if there were a spill or heavily trafficked area the carpet could be replaced locally.

3. Structural Design

Sheets S1 through S6 present the design criteria resultant of the technical data calculated in sections III – A through III – H of this report. These sheets include a foundation plan and foundation details, a lintel plan and lintel details, a roof framing plan and roof framing plan details, and a detail showing the reinforcement between the windows. Found on most of these sheets are a series of notes and design criteria. Calling out various structural components was sometimes original but often mimicked from various other plan set callouts. These sheets have the intention of being as clear and straightforward as possible. They are designed to be used during construction and convey a message to the contractor in a very precise and clear manner.

J. Promotional Material

1. Walkthrough

While not necessarily a “hard engineering” task, our team thought it was appropriate to create a three dimensional walkthrough of the addition so that non-technical people could have a good grasp about the size and layout of the addition. This is especially important when considering that a major component of the purpose of our
design project was simply to get the community that supports Dutton Christian Elementary excited about the project.

The 3-D walkthrough was created in two steps; first the geometry of the building (windows, doors, furniture etc.) was created in Autodesk Architectural Desktop. This streamlines the process of creating the more intricate geometry (like furniture) because Architectural Desktop contains a large library of pre-made parts. This geometry is then imported into a sister program to Architectural Desktop, VIZ Render, which is basically a modified version of 3D Studio Max. Once the geometry is put into VIZ materials and lights can be added and cameras can be controlled to run through the scene, creating the 3-D walkthroughs.

2. Miniature Model

The miniature model is used as a promotional material to help raise the necessary funding for this project. With this miniature, many non-technical people and children can visualize and get excited about what the proposed addition looks like. The architect, Calvin Jen, along with Rama’s friend who is currently studying architecture at University of Southern California, was extremely helpful in giving the input to build the model.

The scale of this miniature is 1’= 1/8”. The major components to build the miniature are balsa wood, plywood, Plexiglas, Claycrete, foam board, an exacto knife, oil paint and green turf. A major obstacle to building this miniature is determining how detailed the model should be. The more detailed the model, the more representative the miniature will be of the proposed design, but more time is needed to construct it.

Our solution to this problem is by starting early to build this miniature. ADDITION AMBITION decided to build the miniature that consists not only of the
proposed design, but also the existing building. This decision is made so that people can see the complete idea of how the Dutton Christian School looks like in the future. In approximately 50 hours of work in two months, one of our team members could get the miniature model done with enough detail to represent the school well. This model will be kept by Dutton Christian School for display.

**K. Budget Estimation**

Dutton Christian Elementary is a private school and therefore does not receive any funding from the government; this means that any building costs will have to be absorbed by the community of people that supports the school. Because of this, one of the project goals was to create an accurate cost estimate for the school so they would have some idea about the funding that they would have to raise.

The cost estimate is based on figures given for Elementary Schools in the book RSMeans 2004, a fairly standard estimating tool in the construction industry. The final estimate of $862,606 is based on an all inclusive (general conditions, architect, materials, labor etc.) estimate of $90.33/SF. This square foot cost has been slightly modified from the $96.45 given for elementary schools built with bearing walls and decorative block, by several factors that were unique to this building. First the RSMeans data is from 2004, and as the estimate has been done in 2005 a 6% increase in square feet cost was allowed for general inflation and the unusually large increases in material costs since 2004. The cost of building in Grand Rapids, Michigan is lower than the national average so a location factor of 0.85 (given by RSMeans) modified the square feet cost. Also, since the addition is 4 feet taller than a normal elementary school another $3.80/SF was added to the construction cost. All these factors together lead to the final estimate of $90.33/SF.
We believe that the building can actually be built for less than the $850,000 estimate because of how previous additions at Dutton Elementary have been built. There is a large amount of donated time and material for these types of projects – construction was referred to as a real “barn raising” event by Principle Terpstra. This should drive costs down and require much less than the actual $850,000 estimate.

If it becomes apparent that this price is more than the school can afford, there is always the option of shrinking the addition down slightly to the size that it was in the second to last floor plan and can be seen in Appendix H. This design is a little more compact and because of the layout the classrooms are closer to the sizes of the existing classrooms.

L. Project’s Schedule

Project scheduling is a crucial part of any large or small scale project. When done correctly project scheduling is updated often and tasks marked completed. By keeping an up to date project schedule the completion of a project is easily gauged and one is able to determine if the project will be completed on time.

ADDITION AMBITION found the recommended project scheduling method, Microsoft Office Project, to not be an efficient way for them to plan the time frame of the individual tasks involved in their project. The main difficulty was determining how long a single task would take. Because we are inexperienced students, gauging the time the individual components of a typical real world project would take was daunting. More time was spent revising the project schedule than what we could determine as efficient time use and this scheduling method was abandoned very quickly at the start of spring semester 2005.
Our team found a very efficient and effective way of communicating tasks and staying on schedule. The method which we used throughout the remainder of the semester involved making a list of duties in which one saw themselves capable of completing with the most effectiveness. These lists were compiled into a “mother list” to make sure that tasks were not overlapping between individuals. It was at this time that ADDITION AMBITION was functioning at its fullest potential. Each team member knew what they had to get done and how long they had to complete it. Throughout the semester each individual task list changed slightly but overall we stayed within our original duties. In general each team member was responsible for (but not limited to) one or two main tasks. Koning was responsible for the walkthrough, the roof plan, and the budget. Suparta took care of the miniature model and the masonry design. DeJong found himself climbing things amid designing the foundation, the lintels, and doing the load and code calculations. Kooy was responsible for the compilation and organization of the plan set. Paper work and assignments were divided equally throughout the team.

For a project of this magnitude and team of four we believe our method of scheduling was efficient. On a project of a much larger scale with many more individuals involved this method of scheduling would not be affective. In a professional environment the time allotted for a task would be better known due to prior experience with similar projects. Our team could have started communicating and delegated tasks much earlier on in the school year to maximize our efficiency.

IV. Conclusion and Summary
Addition Ambition believes the plan set that it created for Dutton Christian Elementary School embodies the design norms that it set out to fulfill in its project. The architectural design of the building, including the decisions to increase the roof height and to use split face block with courses of soldiered brick, creates symmetry between the old and new additions that is aesthetically pleasing. The use of very large windows increases the amount of structural design, but it gives the classrooms an ample amount of natural light and a strong connection to the outdoors; this should promote the psychological health of the building occupants.

The structural design of the building represents the importance of keeping children (who are our future) safe. Emphasis is placed on not only designing a building that was safe, but that would also feel safe to the building occupants. This necessitated over-designing some of the building components to ensure that the masonry walls did not crack – which often happens in buildings that are still structurally sound, but it may still give the occupants the impression that the building is not as safe as it should be.

Several decisions effected the expected budget of the school; one of our most common design changes was to simplify the design to reduce construction time/cost. Increasing the span over the storage room not only allows that room layout to one day be changed into a classroom (which would coincide w/ the building of a storage shed outside), but also makes the fabrication and installation of the joists simpler, leading to reduced construction costs.

Addition Ambition believes that the scale model, 3-D walkthrough, construction plan set, and the calculations supporting the structural design represent a very effective
way to meet the needs of Dutton Christian Elementary and to incorporate the design norms that were an important part of our project.

V. Recommendations

The Dutton Christian Elementary School Addition is a large project with many details throughout it. There are many little things that could be changed to improve our project. The first thing that could be done is to minimize the material used. For instance, the foundation has been designed for three different walls which are the exterior walls, the interior load bearing walls, and the interior non-load bearing walls. The walls in each category do not all experience the same load and therefore do not require the same required footing area. So, a few more foundations could be designed to minimize the materials used. Another modification that could occur is to shrink down some of the lintel beam sizes. A W8X10 is used throughout most of the lintel design for consistency, but there are a few areas where the beam size could be decreased.

Another remaining design consideration requires mechanical engineers. They could design all of the duct work and they could also precisely size and place the rooftop units. Another possibility is for the expansion of the boiler. This would require a cost-benefit analysis to answer the question, is it better to expand the boiler or to use rooftop units? This is a question that ADDITION AMBITION, a group of civil engineers, is not qualified to answer. We assumed that expanding the boiler would be more expensive. We also felt that the worst case scenario from a structural design perspective would be to have the roof top units for they create large dead loads on the roof. This is our reasoning.
for using roof top units, but a mechanical engineer could use actual calculations to solve this dilemma.

Another remaining design consideration requires electrical engineers. As civil engineers we were unable to design the electrical and lighting plans in great detail. We placed the lights with our architect hat on, but we were uncertain of the exact size that should be used for the addition. We used a standard lighting layout and size which we believe is satisfactory, but electrical engineers could say otherwise. The electrical engineers could also design all the wiring for the computers, lights, and wall plugs. Another possibility is to design how the light switches works if there is more than one light switch operating a light.

Another design possibility is for civil or mechanical engineers. The plumbing plan was never designed by ADDITION AMBITION. A civil or mechanical engineer would have the ability to design all of the plumbing for Dutton Christian Elementary School. They would need to determine the necessary slope of the pipes, where the existing plumbing runs in the school, and where to connect the new pipes. Also they would need to determine if the current system could handle the additions added water flow.

A hydraulically focused civil engineer could design the required sewer/drainage plan for the site. ADDITION AMBITION focused on the structural aspects of the design and ignored the hydraulic plans. The addition wipes out 9550 ft$^2$ of land that used to be used for drainage. It may be necessary to add on to the existing storm sewer in the area to allow for the necessary drainage required for the site. It should be determined if the
site will drain properly when the new addition is built or if some type of drainage system will need to be developed.

VI. Acknowledgments

There are many people who have contributed to our project and made the addition design possible. We would like to thank Professor De Rooy of Calvin College for being our advising professor and our mentor. There were many late nights where Professor De Rooy answered tough questions we had or solved a problem in our design with his extensive knowledge of structural engineering. We would like to thank Bob Terpstra, the principle of Dutton Christian Elementary School, and Jay Poll, the custodian/project manager. They have been our clients and contacts at Dutton and we had many long meetings with them to discuss what they wanted the addition to look like. Many ideas were bounced off of them and they have been open to ideas that we presented to them as well. We would like to thank Calvin Jen who is an architect and a Professor at Calvin College. He answered many of our architectural questions. We would also like to thank Roger Lamer, our industrial consultant. He brought up some issues about designing the addition that we had never even thought about until our meeting with him. We would like to thank Dennis Bekken and Design Works AE for furnishing us with electronic copies of the current plans of Dutton Christian Elementary School. We would also like to thank Jonathan Male and Exxel Engineering for furnishing us with the current site plan for Dutton Christian Elementary School. The last thank you we have is to God our Father for giving us the minds, talents, and stamina to be able to complete a task like this.
Appendix

Appendix A – 1. Dead Load Calculations
   2. Live Load Calculations
   3. Snow Load Calculations
   4. Wind Load Calculations
   5. Total Factored Load Calculations

Appendix B – Joist Design

Appendix C – Beam Design

Appendix D – Bearing Plate Design

Appendix E – Lintel and Support Tubing Design

Appendix F – Foundation Design

Appendix G – Masonry Design

Appendix H – Second to Last Floor Plan
Appendix A – 1. Dead Load Calculations

Dead Loads

Design Works Designed the former addition with these
Roof Design Loads
Snow = 30 psf
Membrane & Insul. = 3 psf
Steel Deck = 2 psf
Steel Joist = 2 psf
Mech. & Misc. = 5 psf
Ceiling & Lights = 3 psf

Total Loads = 45 psf

Snow is not a Dead Load
Appendix A – 2. Live Load Calculations

Live Loads

Schools:
- Classroom = 40 psf
- First floor Corridors = 100 psf

Minimum Roof Live Load

\[ L_r = 20 * R_1 * R_2 \]  
where \( 12 \leq L_r \leq 20 \)

\[ R_1 = \begin{cases} 1.2 & \text{for } A_t \leq 200 \text{ ft}^2 \\ 0.6 & \text{for } 200 < A_t < 600 \text{ ft}^2 \\ 1 & \text{for } A_t \geq 600 \text{ ft}^2 \end{cases} \]

\[ R_2 = \begin{cases} 1.2 & \text{for } F \leq 4 \\ 0.6 & \text{for } 4 < F < 12 \\ 1 & \text{for } F \geq 12 \end{cases} \]

\[ A_t := 1250 \text{ ft}^2 \]

\[ F < 4 \]

So \( R_1 := 0.6 \) and \( R_2 := 1 \)

\[ L_r := 20 * R_1 * R_2 \]

\[ L_r = 12 \text{ psf} \]
Appendix A – 3. Snow Load Calculations

Snow Loads

Figure 7-1 implies that the ground snow load \( p_g := 35 \) psf.

Table 7-4 shows that the addition has an importance factor of \( I := 1.1 \).

Dutton's roof has a slope less than 5 degrees because it is a "flat" roof.

We know the equation \( p_f = 0.7 * C_e * C_t * I * p_g \).

If \( p_g \) is greater than 20psf which it is in our case, then

\[ p_f, \text{ minimum} = 20 * I = 20 * 1.1 = 22 \text{ psf} \]

The addition has an Exposure Category C and a fully exposed roof.

Using table 7-2 is can be determined that

\( C_e := 1.0 \)

Table 7-3 is used to find \( C_t \). The addition is in the "other structure" category which implies that

\( C_t := 1.0 \)

\[ p_f := 0.7 * C_e * C_t * I * p_g \]

\[ p_f = 26.95 \text{ psf} \]

We do not need to analyze for unbalance loads for the addition's roof is nearly flat.

Calculations for the existing roofs additional snow load are on the following page.
Existing Roof - Increase Snow Load because of Drifting

New Addition Height = 17' 4"    Existing Height = 11' 5"

\[ \text{Fig. 7-9 shows that } h_d := 3.5 \text{ ft} \]

\[ \text{gamma} := 0.13 p + 14 \quad \text{gamma} = 18.55 \text{ pcf} \]

\[ h_b := \frac{P_f}{\text{gamma}} \quad h_b = 1.453 \text{ ft} \quad h_b > 0.2 \text{ so drift loads apply} \]

\[ h_c := \left( 5 + \frac{11}{12} \right) - h_b \quad h_c = 4.464 \text{ ft} \]

Leeward roof has \( h_d = 3.59 \text{ ft} \)

Windward Roof uses

\[ l_u := 95 + \left( \frac{4}{12} \right) \quad \text{Fig. 7.9} \]

\[ l_u = 191 + \left( \frac{10 + \frac{3}{8}}{12} \right) \quad l_u = 191.865 \text{ ft} \]

\[ \text{Fig. 7.9} \quad h_d := 4.92 \text{ ft} \quad \text{and we use 3/4 of this} \]

So \( h_d := 3.70 \text{ ft} \)

\[ h_d > h_b \text{ Implies that } w := 4 \frac{(h_d)^2}{h_c} \quad w = 12.267 \text{ ft} \]

Max intensity of drift surcharge is \( h_d \cdot \text{gamma} = 68.635 \frac{\text{lb}}{\text{ft}^2} \)

\[ \text{Area}_{\text{drift}} := \frac{1}{2} h_d \cdot w \quad \text{Area}_{\text{drift}} = 22.695 \text{ ft}^2 \]

\[ \text{Surcharge}_{\text{total}} := \text{gamma} \cdot \text{Area}_{\text{drift}} \]

\[ \text{Surcharge}_{\text{total}} = 420.989 \text{ lb/ft of width along edge of existing and new addition} \]
Appendix A – 4. Wind Load Calculations

Wind Loads

Cannot use simplified procedure because we have expansion joints
Minimum wind force is 10 psf

Basic Wind Speed \( \text{Vel} := 90 \) mph

Addition is defined as a low rise building because its height is 17 feet which is less than 30 feet. This implies that the addition has a directionality factor

\[ K_d := 0.85 \]

Table 1.1 says that our addition is a Building Classification III
Table 6.1 says that our addition has an Importance factor of \( I := 1.15 \)

Section 6.5.6.1 says that our addition has an Exposure Category C

Our Addition has no topographic effects so \( K_{zt} := 1 \)

Table 6-5 shows that \( K_z := 0.85 \)

We assume that our building is rigid.
This means that our gust factor is

\[ \text{Gust} := 0.85 \]

It is also assumed that our building is Enclosed

Table 6-7 shows that enclosed buildings have an internal pressure coefficient

\( G_{Cpi} := 0.18 \) or \(-0.18\)

The External Pressure Coefficients \( (G_{Cpf}) \) are found using Figure 6-4.
Our addition has a slope less than 5 degrees.

\[
\begin{align*}
A_1 := 0.4 & \quad A_2 := -0.65 & \quad A_3 := -0.37 & \quad A_4 := -0.25 \\
A_{1E} := 0.61 & \quad A_{2E} := -1.07 & \quad A_{3E} := -0.53 & \quad A_{4E} := -0.43 \\
B_1 := -0.45 & \quad B_2 := -0.66 & \quad B_3 := -0.37 & \quad B_4 := -0.45 & \quad B_5 := 0.4 & \quad B_6 := -0.25 \\
B_{1E} := -0.48 & \quad B_{2E} := -1.05 & \quad B_{3E} := -0.53 & \quad B_{4E} := -0.48 & \quad B_{5E} := 0.61 & \quad B_{6E} := -0.43 \\
\end{align*}
\]

Velocity Pressure can be determined from Equation 6.13 which is

\[ q_z := 0.00256 K_z K_{zt} K_d \text{ Vel}^2 \cdot I \]
For a flat roof  \( q_h = q_z \)

Main Wind Force Resisting System

Section 6.5.12.2.2 computes the main wind force for low rise buildings. Equation 6.16 is used. 

\[ p = q_h \times [(GC_{pf}) - (GC_{pi})] \]

Note - The value of GC_{pi} that gives the greatest pressure is used

<table>
<thead>
<tr>
<th>Case A Main</th>
<th>Case A Edge</th>
</tr>
</thead>
<tbody>
<tr>
<td>( p_{1am} := q_h \times (A_1 + 0.18) )</td>
<td>( p_{1ae} := q_h \times (A_{1E} + 0.18) )</td>
</tr>
<tr>
<td>( p_{1am} = 9.993 )</td>
<td>( p_{1ae} = 13.611 )</td>
</tr>
<tr>
<td>( p_{2am} := q_h \times (A_2 - 0.18) )</td>
<td>( p_{2ae} := q_h \times (A_{2E} - 0.18) )</td>
</tr>
<tr>
<td>( p_{2am} = -14.989 )</td>
<td>( p_{2ae} = -21.536 )</td>
</tr>
<tr>
<td>( p_{3am} := q_h \times (A_3 - 0.18) )</td>
<td>( p_{3ae} := q_h \times (A_{3E} - 0.18) )</td>
</tr>
<tr>
<td>( p_{3am} = -9.476 )</td>
<td>( p_{3ae} = -12.233 )</td>
</tr>
<tr>
<td>( p_{4am} := q_h \times (A_4 - 0.18) )</td>
<td>( p_{4ae} := q_h \times (A_{4E} - 0.18) )</td>
</tr>
<tr>
<td>( p_{4am} = -8.098 )</td>
<td>( p_{4ae} = -10.51 )</td>
</tr>
</tbody>
</table>
Case B Main

\[ p_{1bm} := q_h(B_1 - 0.18) \quad p_{1bm} = -10.854 \]

\[ p_{2bm} := q_h(B_2 - 0.18) \quad p_{2bm} = -14.989 \]

\[ p_{3bm} := q_h(B_3 - 0.18) \quad p_{3bm} = -9.476 \]

\[ p_{4bm} := q_h(B_4 - 0.18) \quad p_{4bm} = -10.854 \]

\[ p_{5bm} := q_h(B_5 + 0.18) \quad p_{5bm} = 9.993 \]

\[ p_{6bm} := q_h(B_6 - 0.18) \quad p_{6bm} = -8.098 \]

Case B Edge

\[ p_{1be} := q_h(B_{1E} - 0.18) \quad p_{1be} = -11.371 \]

\[ p_{2be} := q_h(B_{2E} - 0.18) \quad p_{2be} = -21.536 \]

\[ p_{3be} := q_h(B_{3E} - 0.18) \quad p_{3be} = -12.233 \]

\[ p_{4be} := q_h(B_{4E} - 0.18) \quad p_{4be} = -11.371 \]

\[ p_{5be} := q_h(B_{5E} + 0.18) \quad p_{5be} = 13.611 \]

\[ p_{6be} := q_h(B_{6E} - 0.18) \quad p_{6be} = -10.51 \]
Appendix A – 5. Total Factored Load Calculations

Total Factored Loads (Roof)

Include minimum concentrated loads!

Summary of Loads

Wind load for roof is negative and not included
Rain load is neglected due to snow load

\[ D = \text{dead load}; \quad D := 15 \ (\text{psf}) \]
\[ E = \text{earthquake load}; \quad E := 0 \]
\[ F = \text{load due to fluids with well-defined pressures and maximum heights}; \quad F := 0 \]
\[ F_a = \text{flood load}; \quad F_a := 0 \]
\[ H = \text{load due to lateral earth pressure, ground water pressure, or pressure of bulk materials}; \quad H := 0 \]
\[ L = \text{live load}; \quad L := 0 \]
\[ L_r = \text{roof live load}; \quad L_r := 12 \ (\text{psf}) \]
\[ R = \text{rain load}; \quad R := 0 \]
\[ S = \text{snow load}; \quad S := 27 \ (\text{psf}) \]
\[ T = \text{self-straining force}; \quad T := 0 \]
\[ W = \text{wind load}; \quad W := 0 \]

Factors

1. \[ 1.4(D + F) \]
2. \[ 1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R) \]
3. \[ 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W) \]
4. \[ 1.2D + 1.6W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R) \]
5. \[ 1.2D + 1.0E + 0.5L + 0.2S \]
6. \[ 0.9D + 1.6W + 1.6H \]
7. \[ 0.9D + 1.0E + 1.6H \]

Load One := 1.4(D + F) \quad Load One = 21

Load Two := 1.2(D + F + T) + 1.6(L + H) + 0.5(S) \quad \text{Load Two} = 31.5

Load Three := 1.2D + 1.6(S) + 0.5L \quad \text{Load Three} = 61.2

Load Four := 1.2D + 1.6(W) + 0.5(L) + 0.5(S) \quad \text{Load Four} = 31.5

Load Five := 1.2D + 1.0(E) + 0.5(L) + 0.2(S) \quad \text{Load Five} = 23.4

Load Six := 0.9D + 1.6W + 1.6H \quad \text{Load Six} = 13.5

Load Seven := 0.9D + 1.0E + 1.6H \quad \text{Load Seven} = 13.5

Design Load is Load Three at 61.2 psf

We are using a Total Roof Load of 65 psf
Appendix B – Joist Design

Design of Joists

Basic Loads:

Design load (factored) is 58 psf
A 7.5 ton rooftop unit is max ~ 1066 pounds.

Design Load for Joists:

Assuming spacing of less than 6’ then the Joist Load per foot is:

\[ W_{LRFD} = 6.58 \text{ lb/LF} \quad \text{(LRFD LOAD)} \]

To convert from LRFD to Steel Joist Institute load:

\[ W_{SJI} := \frac{W_{LRFD}}{(1.65 \times 0.9)} \quad W_{SJI} = 234.343 \text{ lb/LF} \]

Reverse engineering previous addition, Design Works worked w/ ~270 lb/LF
Using previous design load (previous addition) of 270 lb/Lf for increased safety factor:

Open Room in middle: Span = 42 ft.
Requires 22K11

Music Room and Teachers Lounge: Span = 30 ft.
Requires 20K5

Classrooms: Span = 34 ft
Requires 20K9

Hallways: Span = 15 ft
Requires 8K1
Appendix C – Beam Design

**Design of Beam**

Design load (factored) is 58 psf

Assuming spacing of less than 6’ then the Beam Load per foot is:

\[ W_{LRFD} := 6.58 \text{ lb/LF} \]

\[ W_{LRFD} = 348 \text{ lb/LF} \]

Concentrated Loads on Beam from Joists:

- Joist Length = 14’
- Joist Load = 348 lb/LF

\[ \text{Concload} := \left( \frac{1}{2} \right) 14 \times 348 \]

\[ \text{Concload} = 2.436 \times 10^3 \text{ lbs} \]

Moment from distributed load, 15 ft. Simply Support Beam:

\[ w := 348 \quad l := 15 \]

\[ M_{\text{max}} := \frac{w \cdot l^2}{8} \quad (\text{Table 5-17 LRFD}) \]

\[ M_{\text{max}} = 9.787 \times 10^3 \text{ lb*ft} \quad \text{equal to 9.8 kip-ft} \]

Moment from Concentrated Loads:
BEAM DESIGN #1

LOAD FROM JOISTS: 

\[ \left( 65 \text{ psf} \right) \left( 3.5 \text{ ft} \right) \left( \frac{14 \text{ ft}}{2} \right) = 2.5 \text{ kip} \]

UNIFORM LOAD: 

\[ \left( 65 \text{ psf} \right) \left( 5.5 \text{ ft} \right) = 358 \text{ kips} \]

\[ \begin{align*}
M_A &= 0 
\Rightarrow (358 \text{ kips} \cdot 15 \text{ ft} \cdot 7.5 \text{ ft}) + (2500 \text{ kips} \cdot 11 \text{ ft}) + (2500 \text{ kips} \cdot 11 \text{ ft}) (12 \text{ ft}) \\
R_E &= 5685 \text{ kips} \\
E_F &= 0 
\Rightarrow R_A = (358 \text{ kips} \cdot 15 \text{ ft}) + (2500 \text{ kips} \cdot 11 \text{ ft}) - 5685 \\
R_A &= 4685 \text{ kips}
\end{align*} \]

\[ M_{max} = 15222 \text{ ft.kips} + 6944 \text{ ft.kips} = 21.7 \text{ kips} \]

file:///C:/DOCUME~1/jmadd/LOCALS~1/Temp/~/LWP0000.htm (1 of 2)3/12/2005 8:52:27 AM
FROM LRFD TABLES w/ UNBRACED LENGTH = 15 ft

\( W10 \times 19 \)
Appendix D – Bearing Plate Design

Design of Bearing Plates

From Beam Design Calcs the design load on the CMU wall is 5.7 Kip

The Design strength of a Load Bearing CMU block is min. 1.7 ksi  
(pg 132 Masonry Structures - Behavior and Design, Drysdale, Hamid, Baker)

From AISC 14-19 the base plate area required:

\[ f_c := 1.7 \quad \phi := 0.60 \quad R_u := 5.7 \]

\[ A_{1\text{req}} := \frac{R_u}{\phi (0.85 \cdot f_c)} \]

\[ A_{1\text{req}} = 6.574 \text{ in}^2 \]

Using 8”x8” bearing plate (64 in$^2 >> 6.6$ in$^2$) for beams because of CMU size and attaching plate to CMU with grouted anchor bolts.

Required Thickness:

\[ k := 0.505 \quad (LRFD 1-22, W8x10) \]

\[ B := 8 \quad F_y := 36 \quad A_1 := 8.8 \]

\[ n := \left(\frac{B}{2}\right) - k \]

\[ t_{\text{min}} := \sqrt{\frac{2.22 \cdot R_u \cdot n^2}{A_1 \cdot F_y}} \]

\[ t_{\text{min}} = 0.259 \text{ in} \]

Use a 1/2” thick base plate

Concentrated Loads on CMU from Joists:

Joist Length = 42’

Joist Load = 270 lb/LF

\[ \text{ConcLoad} := \left(\frac{1}{2}\right) \cdot 42 \cdot 270 + 600 \quad (600 \text{ to account for HVAC}) \]
Conc\textsubscript{load} = 6.27 \times 10^3 \text{ lbs}

Joist bearing plate standard size = 4"x8"

Checking Area:

\[
\begin{align*}
\phi &:= 1.7 \quad f_u := 0.60 \\
A_{1req} &:= \frac{R_u}{\phi(0.85 \cdot f_c)} \\
A_{1req} &:= 7.232 \text{ in}^2
\end{align*}
\]

for joist base plate:

\[
\begin{align*}
k &:= \frac{11 \cdot 2}{16} \quad B := 8 \quad F_y := 36 \quad A_{x} := 8.4 \\
B &:= \left(\frac{B}{2}\right) - k \quad n := \sqrt{\frac{2.22 \cdot R_u \cdot n^2}{A_{1} F_y}} \\
t_{\text{min}} &:= 0.289 \text{ in}
\end{align*}
\]

Use a 1/2" thick base plate

Plate size for lintels:

From AISC 14-19 the base plate area required:

\[
\begin{align*}
\phi &:= 1.7 \quad f_u := 0.60 \\
R_u &= 3.55 \left(\frac{148}{12}\right) \left(\frac{1}{2}\right) \\
R_u &= 21.892 \\
A_{1req} &:= \frac{R_u}{\phi(0.85 \cdot f_c)} \\
A_{1req} &:= 25.25 \text{ in}^2
\end{align*}
\]
Using 8"x8" bearing plate (64in² > 25.25in²) for beams because of CMU size and attaching plate to CMU with grouted anchor bolts.

Required Thickness:

\[
\begin{align*}
k &:= 0.505 \quad \text{(LRFD 1-22, W8x10)} \\
B &:= 8 \quad F_r := 36 \quad A_1 := 8\cdot8 \\
\bar{n} &:= \left(\frac{B}{2}\right) - k \\
t &:= \sqrt{\frac{2.22 \cdot R_u \cdot \bar{n}^2}{A_1 \cdot F_y}} \\
t_{\text{min}} &:= 0.508 \quad \text{in} \\
\end{align*}
\]

Use a 5/8" thick base plate.
Appendix E – Lintel and Support Tubing Design

Beam Size forLintels

Design Loads used

Total Factored roof load = 65 psf

Span for:

- Teaching lounge windows = 25’ 4”
- Classroom windows = 10’ 0”
- Exterior doors = 11’ 0”
- Interior doors = 3’ 4”

Total Load Calculated for:

- North Exterior wall = 3784 lb/ft
- South “ “ “ = 3790 lb/ft
- East “ “ “ = 2949 lb/ft
- Main floor lounge southward wall = 4155 lb/ft
- Classroom North interior wall = 4327 lb/ft
- Storage Room “ “ “ = 3204 lb/ft

Deflection for glass or masonry is \( \frac{L}{600} \)

Deflection for steel beam as by AASHO is \( \frac{L}{800} \)

Mike DeJon
Emp 340
Team #7

ADDITION AMBITION

6’ 8”
Beam Size forLintels

- Teachers Lounge Windows

Beams must have $F_y = 50$ KSI
$F_u = 65$ KSI

Allowable load = 3784 lb/ft

Widths: 25'4" so Lintels spans 25'6" + 2'6" = 28'8"

Design for Moments

Min. General Design

$w_{min} = 0.9M_u$

Assume $w_{min}$ = 28% of

\[
M_u = \frac{w_{min} \cdot 28\% \cdot L^2}{8} = \frac{28 \times 28 \% \times 3784 \times 720}{8} = 3393.77 \text{ ft-kips}
\]

$w_0 = 1200$ lb/ft

\[
M = w_0 \cdot l^2 = 1200 \times 288 = 345600 \text{ ft-kips}
\]

LRFD Table 5-17 to find deflections

\[
\frac{w_0}{60} = \frac{1200}{60} = 0.074 = 0.53"\text{ max.,} \\
\frac{w_0}{60} = \frac{1200}{60} = 0.074 = 0.53"\text{ max.,}
\]

Divide windows into three sections

\[
\delta_{min} = \frac{5(2050)(106.27)}{384(17300)(80.7)} = 0.126''\text{ in.}
\]

\[
I = 561.9\text{ in.}^4 \quad w = 32794.12\text{ in.} \quad \delta = 3805.15\text{ in.}
\]

\[
\delta_{min} = 0.297\text{ in.} \\
I = 38.8\text{ in.}^4 \quad w = 2113.18\text{ in.} \quad \delta = 2133.13\text{ in.}
\]

\[
\delta_{min} = 0.1252 \times 0.53 = 0.07\text{ in.}
\]

OK
Design for teachers' lounge windows


Stress formula:

\[ \sigma = \frac{M}{I} \times \frac{r}{r_{cr}} \]

Critical radius:

\[ r_{cr} = \frac{I}{M} \]

From Table 3-50 in LRFD A-16.1-1965

- Steel:
  - Yield strength: 36 ksi
  - Ultimate strength: 50 ksi

Example:

- Load = 10 kips
- Beam length = 10 ft
- Beam width = 4 ft

Check:

\[ N = \frac{W}{A} \]

- Load = 10 kips
- Area = 4 ft

Yes

\[ x = \frac{1}{2} (106.67) \]

Net section:

\[ \frac{b}{d} = \frac{2}{4} \times \frac{1}{4} \]

Check:

\[ \frac{b}{d} = \frac{1}{4} \times \frac{1}{4} \]

in teachers' lounge window
Classrooms windows on North and South side of East side
Beams 37000 ksi 0.016" = 0.2"

LRFD Table 5-17 End moment

\[ \Delta_{max} = \frac{5LwL}{96EI} \]

\[ w = 3730 \text{ lb} \text{/ft} \]

\[ I = \frac{1}{12} bh^3 \]

\[ E = 30,000 \text{ ksi} \]

\[ L = 60' \]

\[ I = 5' \]

\[ w = 272 \text{ lb} \text{/ft} \]

\[ I = 30.8 \text{ in}^4 \]

\[ \Delta_{max} = 0.0336 \text{ in} < 0.2" \text{ so fits OK} \]

Tube Design

\[ \frac{F}{A} = 10 \]

\[ F = 2 \text{ ksi} \]

\[ A = 2 \text{ in} \]

\[ \frac{F}{A} = 26.9 \text{ ksi} \]

\[ \frac{26.9}{20} = 22.5 \text{ ksi} \]

\[ W = 2 \times 10^6 \text{ lb} \text{/in}^2 \]

\[ \frac{W}{A} = 10.66 \text{ lb} / \text{in} \]

\[ \frac{W}{A} \text{ for Lintel} \]

\[ \frac{W}{A} \text{ for tube} \]
Lintels cont.

Master Bedroom, Foyers, Lounge, Bathroom, and Secretary’s Closet Door Lintels

Load = 1.2(22.5 \text{ lb} / \text{ft} + 48.3 \text{ lb} / \text{ft} + 83.3 \text{ psi} (17 \text{ in} - 7 \text{ in}) ) + 2340 \text{ lb} = 3374.75 \text{ lb/ft}

AM510 1/8" limits Deflection in steel beams to $\frac{L}{800}$

Interior door $L = 3.5$ ft

$\frac{L}{800} = \frac{3.5}{800} = 0.004375 = 0.004375$ in

$\Delta_{max} = \frac{5Wl^2}{384EI}$

$W = 3374.75 \text{ lb/ft} + 1.2 \text{ (total)} \times 0.5 \text{ in} = 58 \text{ in} \times 29,000 \text{ kips} \times I (\text{language})$

$y_{max} = \frac{5(3.3597 \text{ kips/ft} \times \frac{58}{384} \text{ in}^3)}{30.817 \text{ in}^3} = 0.0405 \text{ in} < 0.07 \text{ in} = \text{OK}$

Classroom Door Lintels

Load = 1.2(22.5 \text{ lb} / \text{ft} + 31.5 \text{ lb/ft} + 83.3 \text{ psi} (9 \text{ in}) ) + 250 \text{ lb} = 3547.76 \text{ lb/ft}

$\Delta_{max} = \frac{5Wl^2}{384EI}$

$W = 3547.76 \text{ lb/ft} + 1.2 \text{ (total)} = 3559.76 \text{ lb/ft}$

$y_{max} = \frac{5(3559.76 \text{ kips/ft} \times \frac{58}{384} \text{ in}^3)}{30.817 \text{ in}^3} = 0.0425 \text{ in} < 0.07 \text{ in} = \text{OK}$

All other interior doors have a smaller load so use W8x10
Lintels Cont.

Exterior Door on North wall

Load = 1.2(22.5' + 127.25 psf + 8.8") = 975 kips

\[ W_{nk} = \frac{975 \times \frac{12}{12}}{800} = 11.75 \text{ kips} \]

\[ y = 0.01375 \times 0.11 = 0.0016 	ext{ in} \]

\[ 11.75 \times 2.8 = 34.0 \text{ kips} \]

Shear in the middle of the door

\[ V_{max} = \frac{5}{84} \left[ 12 \frac{2}{8} \times \frac{12}{12} \right] (132 \text{ in})^4 = 0.996 \text{ kips} \]

Deflection in the middle of the door

\[ \delta_{max} = \frac{5}{384} \left[ 12 \frac{2}{8} \times \frac{12}{12} \right] (148 \text{ in})^4 = 0.674 \text{ in} \]

I = 197.2 in^4

W16 x 22

\[ w = 335.4 \text{ kips} + 1.2(30) = 335.4 + 36 = 371.4 \text{ kips} \]

I = 375 in^4

W16 x 31

\[ w = 335.4 \text{ kips} + 1.2(30) = 335.4 + 36 = 371.4 \text{ kips} \]

I = 375 in^4

W12 x 23

\[ w = 335.4 \text{ kips} + 1.2(30) = 335.4 + 36 = 371.4 \text{ kips} \]

I = 301 in^4

W10 x 26

I = 1405 in^4

W6 x 26 for exterior doors
Appendix F – Foundation Design

Foundation Design

Approximate densities of materials

\[
\begin{align*}
\text{CMU} & := 125 \text{ lb/ft}^3 \\
\text{Brick} & := 125 \text{ lb/ft}^3 \\
\text{Concrete} & := 15 \text{ lb/ft}^3 \\
\text{Insulation} & := 2.25 \text{ psf}
\end{align*}
\]

Allowable Soil Bearing Pressure was found by Design Works

\[ q_a := 225 \text{ psf} \]

For Concrete

\[ f_y := 6000 \text{ psi} \quad f_c := 300 \text{ psi} \]

North Exterior Wall Foundation Design (N.E.W.)

Loads

Joist

Spaced 5' 80' span => 80'/5' - 1 = 15...So 15 joists are used along this section.
The Joists in this location weigh 7.5 lb/ft and span 30 feet => 225 lb/truss
N.E.W. will take 1/2 of this load = 112.5 lb for every 5' section

\[ \text{Joist}_{wt} := 22.5 \text{ lb/ft} \]

Wall

\[
\begin{align*}
\text{CMU} & \cdot \frac{8}{12} + \text{Brick} & \cdot \frac{4}{12} + \text{Insulation} & = 127.25 \text{ psf} \\
\text{Wall}_{wt} & := \left( \text{CMU} \cdot \frac{8}{12} + \text{Brick} \cdot \frac{4}{12} + \text{Insulation} \right) \cdot \text{Height}_{wall}
\end{align*}
\]

Wall has height 17'4"

\[ \text{Height}_{wall} := 17 + \frac{1}{3} \]

\[
\begin{align*}
\text{Wall}_{wt} & = 2.206 \times 10^3 \text{ lb/ft} \\
\text{Concrete Block}
\end{align*}
\]

\[ \text{Concrete}_{wt} := \text{Concrete} \cdot \left[ 3 \left( 1 + \frac{2}{12} \right) \right] \]

\[ \text{Concrete}_{wt} = 525 \text{ lb/ft} \]
**Roof**

Roof transfers 65 psf. N.E.W takes 1/2 of the load from room width.

\[
\text{Roof}_{\text{wt}} := 65 \left( \frac{1}{2} \right) \cdot 30
\]

\[
\text{Roof}_{\text{wt}} = 975 \text{ lb/ft}
\]

**Dead Load**

\[
\text{DL} := \text{Joist}_{\text{wt}} + \text{Wall}_{\text{wt}} + \text{Concrete}_{\text{wt}}
\]

\[
\text{DL} = 2.753 \times 10^3 \text{ lb/ft}
\]

**Live Load**

\[
\text{LL} := \text{Roof}_{\text{wt}}
\]

\[
\text{LL} = 975 \text{ lb/ft}
\]

**Total Load**

\[
\text{TL} := 1.2 \times \text{DL} + \text{LL}
\]

\[
\text{TL} = 4.279 \times 10^3 \text{ lb/ft}
\]

1. Consider a 1 ft strip of footing and wall. Also assume 12" height of footing (which is a minimum).

\[
q_a = 2.25 \times 10^3 \text{ psf}
\]

\[
\text{Height}_{\text{footing}} = 12 \text{ in}
\]

Allowable net soil pressure = \(q_n\) - weight/ft^2 of footing

\[
q_n := q_a - \text{Concrete} \cdot 1
\]

\[
q_n = 2.1 \times 10^3 \text{ psf}
\]

\[
\text{Area}_{\text{reqd}} := \frac{\text{TL}}{q_n}
\]

\[
\text{Area}_{\text{reqd}} = 2.038 \text{ ft}^2 / \text{ft of length}
\]

\[
\text{Width} := \text{Area}_{\text{reqd}} \cdot 1
\]

\[
\text{Width} = 2.038 \text{ ft}
\]

Use \(\text{Width} := \left(2 + \frac{4}{12}\right)\)

\[
\text{Width} = 2.333 \text{ ft}
\]
Factored Net Pressure

\[ q_{nu} = \frac{(1.2 \cdot DL + 1.6 \cdot LL)}{\text{Width}} \]

\[ q_{nu} = 2.084 \times 10^3 \frac{\text{lb}}{\text{ft}^2} \]

2. Checking Shear

\[ d := 12 - 3 - 0.5 \quad d = 8.5 \quad \text{in} \]

The tributary area for shear does not exist for this footing because the footing is too narrow
\[ 2d + 12 > 2' 4'' \]
\[ 12'' \text{ is the minimum thickness for the footing so we cannot make it smaller. The width is also at a minimum value.} \]
Therefore we will use a footing that is 12" thick and 2' 4" wide.

3. Design of Longitudinal Reinforcement.

The addition does not need longitudinal reinforcement because the footing is narrower than the wall + 2d, where d is the effective depth. We only need reinforcement along the length of the wall which is designed in the next section.

4. Temperature/Shrinkage Reinforcement

By ACI sec 7.12.2
\[ A_s = 0.0018 \times \text{Base} \times \text{Height} \]

\[ A_s := 0.0018 \times (\text{Width} \cdot 12) \times \text{Height}_{\text{footing}} \]

\[ A_s = 0.605 \text{ in}^2 \]

The maximum spacing = 5\times \text{thickness or } 18'', the smaller of the two. In our case the max spacing is 18''

Provide 2 #5 bars 7in o.c. for temp./shrinkage reinforcement.
This will give us an Area Steel of

\[ A_{\text{actual}} := 2 \cdot 0.31 \quad A_{\text{actual}} = 0.62 \text{ in}^2 \]

Which is greater than the required 0.605 in^2
South Exterior Wall Foundation Design (S.E.W.)

**Loads**

**Joist**

Spaced 5’ 85’ span => 85’/5’ - 2 = 15...Because of two load bearing walls there are 15 joists are used along this section.
The Joists in this location weigh 9 lb/ft and span 35 feet => 315 lb/truss
S.E.W. will take 1/2 of this load = 157.5 lb for every 5’ section

\[ \text{Joist wt} = 31.5 \text{ lb/ft} \]

**Wall**

CMU \( \frac{8}{12} \) + Brick \( \frac{4}{12} \) + Insulation = 127.25 psf

Wall has height 17’4”

\[ \text{Height wall} = 17 + \frac{1}{3} \]

\[ \text{Wall wt} = 2.206 \times 10^3 \text{ lb/ft} \]

**Concrete Block**

\[ \text{Concrete wt} = 525 \text{ lb/ft} \]

**Roof**

Roof transfers 65 psf. S.E.W takes 1/2 of the load from room width.

\[ \text{Roof wt} = 975 \text{ lb/ft} \]

**Dead Load**

\[ \text{DL} = \text{Joist wt} + \text{Wall wt} + \text{Concrete wt} \]

\[ \text{DL} = 2.762 \times 10^3 \text{ lb/ft} \]
Live Load

\[ LL := \text{Roof}_{wt} \]

\[ LL = 975 \text{ lb/ft} \]

Total Load

\[ TL := 1.2DL + LL \]

\[ TL = 4.29 \times 10^3 \text{ lb/ft} \]

The Roof LL already had the 1.6 factor taken into account

1. Consider a 1 ft strip of footing and wall. Also assume 12” height of footing (which is a minimum).

\[ q_a = 2.25 \times 10^3 \text{ psf} \]

\[ \text{Height}_{min} = 12 \text{ in} \]

Allowable net soil pressure = \( q_n \) - weight/ft^2 of footing

\[ q_{a} := q_a - \text{Concrete} \cdot 1 \]

\[ q_n = 2.1 \times 10^3 \text{ psf} \]

\[ \text{Area}_{reqd} := \frac{TL}{q_n} \]

\[ \text{Area}_{reqd} = 2.043 \text{ ft}^2 / \text{ft of length} \]

\[ \text{Width} := \text{Area}_{reqd} \cdot 1 \]

\[ \text{Width} = 2.043 \text{ ft} \]

Use\[ \text{Width} := \left( 2 + \frac{4}{12} \right) \]

\[ \text{Width} = 2.333 \text{ ft} \]

Factored Net Pressure

\[ q_{nu} := \frac{(1.2DL + 1.6LL)}{\text{Width}} \]

\[ q_{nu} = 2.089 \times 10^3 \text{ lb/ft}^2 \]

2. Checking Shear

\[ d := 12 - 3 - 0.5 \]

\[ d = 8.5 \text{ in} \]

The tributary area for shear does not exist for this footing because the footing is too narrow

\[ 2d + 12 > 2' 4'' \]

12” is the minimum thickness for the footing so we cannot make it smaller. The width is also at a minimum value.

Therefore we will use a footing that is 12” thick and 2’ 4” wide.
3. Design of Longitudinal Reinforcement.

The addition does not need longitudinal reinforcement because the footing is narrower than the wall + 2d, where d is the effective depth. We only need reinforcement along the length of the wall which is designed in the next section.

4. Temperature/Shrinkage Reinforcement

By ACI sec 7.12.2
As = 0.0018 * Base * Height

\[ A_s := 0.0018 \cdot \text{Width} \cdot \text{Height}_{\text{footing}} \]

\[ A_s = 0.605 \text{ in}^2 \]

The maximum spacing = 5*thickness or 18", the smaller of the two. In our case the max spacing is 18"

Provide 2 #5 bars 7in o.c. for temp./shrinkage reinforcement. This will give us an Area Steel of

\[ A_{\text{actual}} := 2 \cdot 0.31 \quad A_{\text{actual}} = 0.62 \text{ in}^2 \]

Which is greater than the required 0.605 in^2
East Exterior Wall Foundation Design (E.E.W.)

Loads

Joist
No Joist Load is felt by this wall

Wall
\[
\text{CMU} \cdot \frac{8}{12} + \text{Brick} \cdot \frac{4}{12} + \text{Insulation} = 127.25 \text{ psf}
\]

Wall has height 17'4"
\[
\text{Height}_{\text{wall}} := 17 + \frac{1}{3}
\]

\[
\text{Wall}_{\text{wt}} = \left( \text{CMU} \cdot \frac{8}{12} + \text{Brick} \cdot \frac{4}{12} + \text{Insulation} \right) \cdot \text{Height}_{\text{wall}}
\]

\[
\text{Wall}_{\text{wt}} = 2.206 \times 10^3 \frac{\text{lb}}{\text{ft}}
\]

Concrete Block
\[
\text{Concrete}_{\text{wt}} := \text{Concrete} \cdot 3 \left( 1 + \frac{2}{12} \right)
\]

\[
\text{Concrete}_{\text{wt}} = 525 \frac{\text{lb}}{\text{ft}}
\]

Roof
Roof transfers 65 psf. E.E.W takes 1/2 of the load from room width.

\[
\text{Roof}_{\text{wt}} := 65 \cdot 1.25
\]

\[
\text{Roof}_{\text{wt}} = 162.5 \frac{\text{lb}}{\text{ft}}
\]

Dead Load
\[
\text{DL} := \text{Wall}_{\text{wt}} + \text{Concrete}_{\text{wt}}
\]

\[
\text{DL} = 2.731 \times 10^3 \frac{\text{lb}}{\text{ft}}
\]

Live Load
\[
\text{LL} := \text{Roof}_{\text{wt}}
\]

\[
\text{LL} = 162.5 \frac{\text{lb}}{\text{ft}}
\]
Total Load

\[ TL := 1.2DL + LL \]

\[ TL = 3.439 \times 10^3 \text{ lb/ft} \]

1. Consider a 1 ft strip of footing and wall. Also assume 12" height of footing (which is a minimum).

\[ qa = 2.25 \times 10^3 \text{ psf} \]

Height of footing \( h := 12 \text{ in} \)

Allowable net soil pressure = \( qn \) - weight/ft\(^2\) of footing

\[ qa := qa - \text{Concrete} \cdot 1 \]

\[ qn = 2.1 \times 10^3 \text{ psf} \]

\[ \frac{TL}{qn} \]

Area \( A \) = \( \frac{TL}{qn} \) \quad \text{Area reqd} = 1.638 \text{ ft}^2 / \text{ft of length} \]

Width \( W \) := Area reqd \cdot 1

\[ \text{Width} = 1.638 \text{ ft} \]

Use \( \text{Width} := \left( 2 + \frac{4}{12} \right) \)

\[ \text{Width} = 2.333 \text{ ft} \]

Factored Net Pressure

\[ qnu := \frac{(1.2DL + 1.6LL)}{\text{Width}} \]

\[ qnu = 1.516 \times 10^3 \text{ lb/ft}^2 \]

2. Checking Shear

\[ d := 12 - 3 - 0.5 \]

\[ d = 8.5 \text{ in} \]

The tributary area for shear does not exist for this footing because the footing is too narrow (2d+12>2' 4"").

12" is the minimum thickness for the footing so we cannot make it smaller. The width is also at a minimum value.

Therefore we will use a footing that is 12" thick and 2' 4" wide.

3. Design of Longitudinal Reinforcement.
The addition does not need longitudinal reinforcement because the footing is narrower than the wall + 2d, where d is the effective depth. We only need reinforcement along the length of the wall which is designed in the next section.

4. Temperature/Shrinkage Reinforcement

By ACI sec 7.12.2

\[ A_s = 0.0018 \times \text{Base} \times \text{Height} \]

\[ A_s = 0.0018 \times (\text{Width} - 12) \times \text{Height}_{\text{footing}} \]

\[ A_s = 0.605 \text{ in}^2 \]

The maximum spacing = 5*thickness or 18", the smaller of the two. In our case the max spacing is 18"

Provide 2 #5 bars 7" o.c. for temp./shrinkage reinforcement. This will give us an Area Steel of

\[ A_{\text{actual}} = 2 \times 0.31 \]

\[ A_{\text{actual}} = 0.62 \text{ in}^2 \]

Which is greater than the required 0.605 in^2
Music/Teacher Interior South Wall Foundation Design (M.I.S.W.)

Loads

Joist

North Side

Spaced 5’  80’ span => 80/5' - 1 = 15...So 15 joists are used along this section.
The Joists in this location weigh 7.5 lb/ft and span 30 feet => 225 lb/truss
M.I.S.W. will take 1/2 of this load = 112.5 lb for every 5’ section

\[ \text{Joist}_{wt1} := 22.5 \frac{\text{lb}}{\text{ft}} \]

South Side

Spaced 5’  80’ span => 80/5' - 1 = 15...So 15 joists are used along this section.
The Joists in this location weigh 11.5 lb/ft and span 42 feet => 483 lb/truss
M.I.S.W. will take 1/2 of this load = 241.5 lb for every 5’ section

\[ \text{Joist}_{wt2} := 48.3 \frac{\text{lb}}{\text{ft}} \]

Wall

CMU \( \frac{8}{12} \) = 83.333 psf

Wall has height 17’4"  \( \text{Height}_{wall} := 17 + \frac{1}{3} \)

\[ \text{Wall}_{wt} := \left( \text{CMU} \frac{8}{12} \right) \cdot \text{Height}_{wall} \]

\[ \text{Wall}_{wt} = 1.444 \times 10^3 \frac{\text{lb}}{\text{ft}} \]

Concrete Block

Concrete := Concrete \( \frac{8}{12} \)

\[ \text{Concrete}_{wt} = 300 \frac{\text{lb}}{\text{ft}} \]
Roof

Roof transfers 65 psf. M.I.S.W takes 1/2 of the load from room width.

\[
\text{Roof}_{\text{wt}} = 65 \left( \frac{1}{2} \cdot 30 + \frac{1}{2} \cdot 42 \right)
\]

\[
\text{Roof}_{\text{wt}} = 2.34 \times 10^3 \text{ lb/ft}
\]

Dead Load

\[
\text{DL} := \text{Joist}_{\text{wt1}} + \text{Joist}_{\text{wt2}} + \text{Wall}_{\text{wt}} + \text{Concrete}_{\text{wt}}
\]

\[
\text{DL} = 1.815 \times 10^3 \text{ lb/ft}
\]

Live Load

\[
\text{LL} := \text{Roof}_{\text{wt}}
\]

\[
\text{LL} = 2.34 \times 10^3 \text{ lb/ft}
\]

Total Load

\[
\text{TL} := 1.2\text{DL} + \text{LL}
\]

\[
\text{TL} = 4.518 \times 10^3 \text{ lb/ft}
\]

The Roof LL already had the 1.6 factor taken into account

1. Consider a 1 ft strip of footing and wall. Also assume 12" height of footing (which is a minimum).

\[
\text{qa} = 2.25 \times 10^3 \text{ psf}
\]

\[
\text{Height}_{\text{footing}} := 12 \text{ in}
\]

Allowable net soil pressure = \( qn \) - weight/ft^2 of footing

\[
\text{qa} := \text{qa} - \text{Concrete} \cdot 1
\]

\[
\text{qn} = 2.1 \times 10^3 \text{ psf}
\]

\[
\text{Area}_{\text{reqd}} := \frac{\text{TL}}{\text{qn}}
\]

\[
\text{Area}_{\text{reqd}} = 2.152 \text{ ft}^2/\text{ft of length}
\]

\[
\text{Width} := \text{Area}_{\text{reqd}} \cdot 1
\]

\[
\text{Width} = 2.152 \text{ ft}
\]

Use

\[
\text{Width} := \left( 2 + \frac{4}{12} \right)
\]

\[
\text{Width} = 2.333 \text{ ft}
\]
Factored Net Pressure

\[ q_{nu} = \frac{(1.2 \cdot DL + 1.6 \cdot LL)}{\text{Width}} \]

\[ q_{nu} = 2.538 \times 10^3 \, \frac{\text{lb}}{\text{ft}^2} \]

2. Checking Shear

\[ d := 12 - 3 - 0.5 \quad d = 8.5 \, \text{in} \]

The tributary area for shear does not exist for this footing because the footing is too narrow

\[ 2d + 12 > 24' \]

12" is the minimum thickness for the footing so we cannot make it smaller. The width is also

at a minimum value.

Therefore we will use a footing that is 12" thick and 2’ 4” wide.

3. Design of Longitudinal Reinforcement.

The addition does not need longitudinal reinforcement because the footing is narrower

than the wall + 2d, where d is the effective depth. We only need reinforcement along the

length of the wall which is designed in the next section.

4. Temperature/Shrinkage Reinforcement

By ACI sec 7.12.2

\[ A_s = 0.0018 \times \text{Base} \times \text{Height} \]

\[ A_s := 0.0018 \times \text{Width} \times 12 \times \text{Height}_{\text{footing}} \]

\[ A_s = 0.605 \, \text{in}^2 \]

The maximum spacing = 5*thickness or 18", the smaller of the two.

In our case the max spacing is 18"

Provide 2 #5 bars 7in o.c. for temp./shrinkage reinforcement.

This will give us an Area Steel of

\[ A_{\text{actual}} := 2 \times 0.31 \quad A_{\text{actual}} = 0.62 \, \text{in}^2 \]

Which is greater than the required 0.605 in^2
Middle Room South Wall Foundation Design (M.R.S.W.)

**Loads**

**Joist**

**North Side**

Spaced 5' 48'9" span => 50'/5' = 10...So 10 joists are used along this section.  
The Joists in this location weigh 11.5 lb/ft and span 42 feet => 483 lb/truss  
M.R.S.W. will take 1/2 of this load = 241.5 lb for every 5' section

\[
\text{Joist}_{1} = 48.3 \text{ lb/ft}
\]

**South Side**

Spaced 5' 48'9" span => 50'/5' = 10...So 10 joists are used along this section.  
The Joists in this location weigh 9 lb/ft and span 35 feet => 315 lb/truss  
M.R.S.W. will take 1/2 of this load = 157.5 lb for every 5' section

\[
\text{Joist}_{2} = 31.5 \text{ lb/ft}
\]

**Wall**

CMU \(\frac{8}{12}\) = 83.333 psf

Wall has height 17'4"  
Height_{wall} = 17 + \frac{1}{3}

Wall_{wt} = 1.444 \times 10^{3} \text{ lb/ft}

**Concrete Block**

Concrete_{wt} = 300 \text{ lb/ft}
Roof

Roof transfers 65 psf. M.R.S.W takes 1/2 of the load from room width.

\[
\text{Roof}_{\text{wt}} = 65 \left( \frac{1}{2} \cdot 35 + \frac{1}{2} \cdot 42 \right)
\]

\[
\text{Roof}_{\text{wt}} = 2.502 \times 10^3 \text{ lb/ft}
\]

Dead Load

\[
\text{DL} := \text{Joist}_{\text{wt1}} + \text{Joist}_{\text{wt2}} + \text{Wall}_{\text{wt}} + \text{Concrete}_{\text{wt}}
\]

\[
\text{DL} = 1.824 \times 10^3 \text{ lb/ft}
\]

Live Load

\[
\text{LL} := \text{Roof}_{\text{wt}}
\]

\[
\text{LL} = 2.502 \times 10^3 \text{ lb/ft}
\]

Total Load

\[
\text{TL} := 1.2 \times \text{DL} + \text{LL}
\]

The Roof LL already had the 1.6 factor taken into account

\[
\text{TL} = 4.692 \times 10^3 \text{ lb/ft}
\]

1. Consider a 1 ft strip of footing and wall. Also assume 12" height of footing (which is a minimum).

\[
q_a = 2.25 \times 10^3 \text{ psf}
\]

\[
\text{Height}_{\text{footing}} := 12 \text{ in}
\]

Allowable net soil pressure = \(q_a - \text{Concrete} \cdot 1\)

\[
q_n = 2.1 \times 10^3 \text{ psf}
\]

\[
\text{Area}_{\text{reqd}} := \frac{\text{TL}}{q_n}
\]

\[
\text{Area}_{\text{reqd}} = 2.234 \text{ ft}^2 / \text{ft of length}
\]

\[
\text{Width} := \text{Area}_{\text{reqd}} \cdot 1
\]

\[
\text{Width} = 2.234 \text{ ft}
\]

Use \[
\text{Width} := \left( 2 + \frac{4}{12} \right) \]

\[
\text{Width} = 2.333 \text{ ft}
\]
Factored Net Pressure \( q_{nu} = \frac{(1.2 \, DL + 1.6 \, LL)}{\text{Width}} \)

\[ q_{nu} = 2.654 \times 10^3 \, \frac{\text{lb}}{\text{ft}^2} \]

2. Checking Shear

\( d := 12 - 3 - 0.5 \quad d = 8.5 \, \text{in} \)

The tributary area for shear does not exist for this footing because the footing is too narrow
\( 2d+12 > 2'4" \)
\( 12" \) is the minimum thickness for the footing so we cannot make it smaller. The width is also
at a minimum value.

Therefore we will use a footing that is 12" thick and 2’ 4” wide.

3. Design of Longitudinal Reinforcement.

The addition does not need longitudinal reinforcement because the footing is narrower
than the wall + 2d, where d is the effective depth. We only need reinforcement along the
length of the wall which is designed in the next section.

4. Temperature/Shrinkage Reinforcement

By ACI sec 7.12.2
\( A_s = 0.0018 \times \text{Base} \times \text{Height} \)

\( A_s := 0.0018 (\text{Width} - 12) \times \text{Height}_{\text{footing}} \)

\( A_s = 0.605 \, \text{in}^2 \)

The maximum spacing = 5*thickness or 18", the smaller of the two. In our case the max spacing is 18"

Provide 2 #5 bars 7in o.c. for temp./shrinkage reinforcement. This will give us an Area Steel of

\( A_{actual} := 2 \times 0.31 \quad A_{actual} = 0.62 \, \text{in}^2 \)

Which is greater than the required 0.605 in^2
Storage Room North Wall Foundation Design (S.R.N.W.)

Loads

Joist

**North Side**

Spaced 5' 35'7” span => 40'/5' - 1 = 7...So 7 joists are used along this section. The Joists in this location weigh 5.1 lb/ft and span 15 feet => 76.5 lb/truss S.R.N.W. will take 1/2 of this load = 38.25 lb for every 5' section

\[ \text{Joist}_{\text{wt1}} = 7.6 \frac{\text{lb}}{\text{ft}} \]

**South Side**

Spaced 5' 35'7” span => 40'/5' - 1 = 7...So 7 joists are used along this section. The Joists in this location weigh 7.5 lb/ft and span 29 feet => 217.5 lb/truss S.R.N.W. will take 1/2 of this load = 108.75 lb for every 5' section

\[ \text{Joist}_{\text{wt2}} = 21.7 \frac{\text{lb}}{\text{ft}} \]

Wall

CMU $\frac{8}{12} = 83.333$ psf

Wall has height 17'4” Height_wall := $17 + \frac{1}{3}$

\[ \text{Wall}_{\text{wt}} = 1.444 \times 10^3 \frac{\text{lb}}{\text{ft}} \]

Concrete Block

Concrete := Concrete $\cdot \frac{8}{12}$

Concrete_{wt} = 300 \frac{\text{lb}}{\text{ft}}
Roof transfers 65 psf. S.R.N.W takes 1/2 of the load from room width.

\[ \text{Roof}_{\text{wt}} = 65 \left( \frac{1}{2} \cdot 29 + \frac{1}{2} \cdot 15 \right) \]

\[ \text{Roof}_{\text{wt}} = 1.43 \times 10^3 \ \text{lb/ft} \]

Dead Load

\[ \text{DL} := \text{Joist}_{\text{wt}1} + \text{Joist}_{\text{wt}2} + \text{Wall}_{\text{wt}} + \text{Concrete}_{\text{wt}} \]

\[ \text{DL} = 1.774 \times 10^3 \ \text{lb/ft} \]

Live Load

\[ \text{LL} := \text{Roof}_{\text{wt}} \]

\[ \text{LL} = 1.43 \times 10^3 \ \text{lb/ft} \]

Total Load

\[ \text{TL} := 1.2\text{DL} + \text{LL} \]

\[ \text{TL} = 3.559 \times 10^3 \ \text{lb/ft} \]

The Roof LL already had the 1.6 factor taken into account

1. Consider a 1 ft strip of footing and wall. Also assume 12" height of footing (which is a minimum).

\[ q_a = 2.25 \times 10^3 \ \text{psf} \]

\[ \text{Height}_{\text{reqd}} := 12 \ \text{in} \]

Allowable net soil pressure = \( q_n \) - weight/ft^2 of footing

\[ q_n := q_a - \text{Concrete} \cdot 1 \]

\[ q_n = 2.1 \times 10^3 \ \text{psf} \]

\[ \text{Area}_{\text{reqd}} := \frac{\text{TL}}{q_n} \]

\[ \text{Area}_{\text{reqd}} = 1.695 \ \text{ft}^2 / \text{ft of length} \]

\[ \text{Width} := \text{Area}_{\text{reqd}} \cdot 1 \]

\[ \text{Width} = 1.695 \ \text{ft} \]

Use \[ \text{Width} := 2 + \frac{4}{12} \]

\[ \text{Width} = 2.333 \ \text{ft} \]
Factored Net Pressure

\[ q_{nu} = \frac{(1.2 \cdot DL + 1.6 \cdot LL)}{\text{Width}} \times 10^3 \text{ lb/ft}^2 \]

2. Checking Shear

\[ d := 12 - 3 - 0.5 \quad \text{d} = 8.5 \text{ in} \]

The tributary area for shear does not exist for this footing because the footing is too narrow \(2d+12 > 2'4"\).

12" is the minimum thickness for the footing so we cannot make it smaller. The width is also at a minimum value.

Therefore we will use a footing that is 12" thick and 2’ 4" wide.

3. Design of Longitudinal Reinforcement.

The addition does not need longitudinal reinforcement because the footing is narrower than the wall + 2d, where d is the effective depth. We only need reinforcement along the length of the wall which is designed in the next section.

4. Temperature/Shrinkage Reinforcement

By ACI sec 7.12.2

\[ A_s := 0.0018 \times \text{Base} \times \text{Height} \]

\[ A_s := 0.0018 \times (\text{Width} \times 12) \times \text{Height}_{\text{footing}} \]

\[ A_s = 0.605 \text{ in}^2 \]

The maximum spacing = 5*thickness or 18", the smaller of the two.

In our case the max spacing is 18"

Provide 2 #5 bars 7in o.c. for temp./shrinkage reinforcement.

This will give us an Area Steel of

\[ A_{\text{actual}} = 2 \times 0.31 \quad A_{\text{actual}} = 0.62 \text{ in}^2 \]

Which is greater than the required 0.605 in^2
Storage Room South Wall Foundation Design (S.R.S.W.)

**Loads**

**Joist**

**North Side**

Spaced 5’ 357” span => 40/5’ -1 = 7...So 7 joists are used along this section.
The Joists in this location weigh 7.5 lb/ft and span 29 feet => 217.5 lb/truss
S.R.S.W. will take 1/2 of this load = 108.75 lb for every 5’ section

\[
\text{Joist} = 21.75 \text{ lb/ft}
\]

**South Side**

Spaced 5’ 357” span => 40/5’ -1 = 7...So 7 joists are used along this section.
The Joists in this location weigh 9 lb/ft and span 35 feet => 315 lb/truss
S.R.S.W. will take 1/2 of this load = 157.5 lb for every 5’ section

\[
\text{Joist} = 31.5 \text{ lb/ft}
\]

**Wall**

CMU \[\frac{8}{12}\] = 83.333 psf

Wall has height 17’4”

\[
\text{Height}_{\text{wall}} = 17 + \frac{1}{3}
\]

\[
\text{Wall}_{\text{wt}} = 1.444 \times 10^3 \text{ lb/ft}
\]

**Concrete Block**

\[
\text{Concrete}_{\text{wt}} = \text{Concrete} \times 3 \times \left(\frac{8}{12}\right)
\]

\[
\text{Concrete}_{\text{wt}} = 300 \text{ lb/ft}
\]
Roof

Roof transfers 65 psf. S.R.S.W takes 1/2 of the load from room width.

\[
\text{Roof wt} = 65 \left( \frac{1}{2} \cdot 29 + \frac{1}{2} \cdot 35 \right) = 2.08 \times 10^3 \text{ lb/ft}
\]

Dead Load

\[
\text{DL} := \text{Joist wt}_1 + \text{Joist wt}_2 + \text{Wall wt} + \text{Concrete wt}
\]

\[
\text{DL} = 1.798 \times 10^3 \text{ lb/ft}
\]

Live Load

\[
\text{LL} := \text{Roof wt}
\]

\[
\text{LL} = 2.08 \times 10^3 \text{ lb/ft}
\]

Total Load

\[
\text{TL} := 1.2\text{DL} + \text{LL}
\]

The Roof LL already had the 1.6 factor taken into account

\[
\text{TL} = 4.237 \times 10^3 \text{ lb/ft}
\]

1. Consider a 1 ft strip of footing and wall.

Also assume 12” height of footing (which is a minimum).

\[
q_a = 2.25 \times 10^3 \text{ psf}
\]

Height (min.) := 12 in

Allowable net soil pressure = \(q_n - \frac{\text{weight}}{\text{ft}^2}\) of footing

\[
q_n := q_a - \text{Concrete} \cdot 1
\]

\[
q_n = 2.1 \times 10^3 \text{ psf}
\]

\[
\text{Area reqd} := \frac{\text{TL}}{q_n}
\]

Area reqd = 2.018 \text{ ft}^2 / \text{ft of length}

\[
\text{Width} := \text{Area reqd} \cdot 1
\]

Width = 2.018 ft

Use \[
\text{Width} := 2 + \frac{4}{12}
\]

Width = 2.333 ft
Factored Net Pressure

\[ q_{nu} = \frac{(1.2DL + 1.6LL)}{\text{Width}} \]

\[ q_{nu} = 2.351 \times 10^3 \frac{\text{lb}}{\text{ft}^2} \]

2. Checking Shear

\[ d := 12 - 3 - 0.5 \quad d = 8.5 \quad \text{in} \]

The tributary area for shear does not exist for this footing because the footing is too narrow
\[ 2d + 12 > 2'4" \]
\[ 12" \text{ is the minimum thickness for the footing so we cannot make it smaller. The width is also} \]
\[ \text{at a minimum value.} \]

Therefore we will use a footing that is 12" thick and 2' 4" wide.

3. Design of Longitudinal Reinforcement.

The addition does not need longitudinal reinforcement because the footing is narrower than the wall + 2d, where d is the effective depth. We only need reinforcement along the length of the wall which is designed in the next section.

4. Temperature/Shrinkage Reinforcement

By ACI sec 7.12.2

\[ A_s := 0.0018 \times \text{Base} \times \text{Height} \]

\[ A_s := 0.0018 \times (\text{Width} \times 12) \times \text{Height}_{\text{footing}} \]

\[ A_s = 0.605 \quad \text{in}^2 \]

The maximum spacing = 5*thickness or 18", the smaller of the two. In our case the max spacing is 18"

Provide 2 #5 bars 7in o.c. for temp./shrinkage reinforcement. This will give us an Area Steel of

\[ A_{\text{actual}} := 2 \times 0.31 \quad A_{\text{actual}} = 0.62 \quad \text{in}^2 \]

Which is greater than the required 0.605 in^2
Appendix G – Masonry Design

ROOF LOAD CALCULATION

1/2 of the biggest area covered by the joist

\[ \text{Area} := \frac{1}{2} (84.67 \text{ ft} \times 35.2 \text{ ft}) \]

\[ \text{Area} = 1.49 \times 10^3 \text{ ft}^2 \]

Roof Load

\[ \text{RoofLoad} := 15 \frac{\text{lb}}{\text{ft}^2} \times \text{Area} \]

\[ \text{RoofLoad} = 2.235 \times 10^4 \text{ lb} \]

The roof load is then calculated in lb/ft unit

\[ \text{TotalRoofLoad} := \frac{\text{RoofLoad}}{84.67 \text{ ft}} \]

\[ \text{TotalRoofLoad} = \frac{264 \text{ lb}}{\text{ft}} \]
Design of a Reinforced Masonry Wall with Out-of-Plane Loads

Using the 2002 MSJC ASD Design Code

Material and Construction Data

8 in. CMU, Full grout, running bond
Wall Weight = 77.9 psf (From Tables)
Type N mortar Portland cement lime / Mortar cement Mortar, Coarse Grout
CMU Concrete Density = 115 pcf
$f_m' = 1353$ psi (Specified)
$E_m = 900f_m' = 1217846$ psi

Wall Design Details

Thickness = 7.625 in.
Height = 208 in. (Simply Supported Wall, Effective height = H)
$x = 3.813$ in.
#6 Bars, Grade 60
Reinforcement Spacing = 48 in. On-Center

Wall Support: Simply Supported Wall

Specified Load Components

<table>
<thead>
<tr>
<th>Load</th>
<th>P (lb)</th>
<th>e (in)</th>
<th>W1 (psf)</th>
<th>W2 (psf)</th>
<th>L (in)</th>
<th>h1 (in)</th>
<th>h2 (in)</th>
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</table>

Controlling Load Cases

Section Forces with Controlling Flexure and Axial Load -- $D + L + L_r + S + R + W$
| x/H = 0.590 from bottom of wall                                      |
| V = -20.8615 lb/ft                                                   |
| M_L = 6540.9761 lb-in./ft                                           |
| P = 817.609 lb/ft at e = 0.381013 in                                 |
| M_T = M_L + P_e = 6852.4961 lb-in./ft                               |
| Moment Capacity = 14659.631 lb-in./ft (1221.6359 lb-ft/ft) at this axial load |
| Shear Capacity = 2243.7223 lb/ft                                     |

The wall is adequate for these critical section forces.

Section Forces with Controlling Shearing Force—D + L + L_r + S + R + W

| x/H = 0.000 from bottom of wall                                      |
| V = 132.538 lb/ft                                                   |
| M_L = 0 lb-in./ft                                                   |
| P = 1614.27 lb/ft at e = 0 in                                      |
| M_T = M_L + P_e = 0 lb-in./ft                                      |
| Moment Capacity = 15802.488 lb-in./ft at this axial load           |
| Shear Capacity = 2243.7223 lb/ft                                    |

The wall is adequate for these critical section forces.

These were found to be load cases that controlled the design.
The flexural, shear and axial forces shown are those occurring at the critical section for the case controlled by flexure and at the critical section for the case controlled by shear.

The following design calculations are for the section with controlling bending moment.

Section Design Forces Used

| V = -20.8615 lb/ft (Computed from Loads)                           |
| M_L = 6540.9761 lb-in./ft (Computed from Loads)                   |
| P = 817.609 lb/ft at e = 0.381013 in (Computed from Loads)        |

Computed Design Values

Note: 1/3 stress increase was used
Effective Width = 48 in.
Web Width = 48 in. on effective width
allowable shearing force = 2243.72 lb/ft
the wall is adequate in shear

required $A_g = 0.1404 \text{ in}^2$ each reinforced cell (0.0351 in$^2$/ft) OK
d = 3.813 in.
n = 23.82
$k_{balanced} = 0.3092$
$b_{balanced} = 0.8869$
k = 0.2217
j = 0.9261
$P_{max}$ (compression) = 89092.4 lbs (22273.1 lbs/ft) OK
$P_{max}$ (tension) = 14060 lbs (3520 lbs/ft) OK

the wall has adequate capacity.
Reinforced Wall Interaction Diagram
Using the 2002 MSJC ASD Design Code

CMU Size = 8 in. (Fully grouted)
F_y = 60 ksi  f_m' = 1353 psi
A_s = 0.44 sq in.  c = 3.813 in.  H = 208 in. (17.33 ft)

Axial Compression, P (lbs/ft)

When reinforcement spacing exceeds 6T, stresses in the unreinforced masonry spanning between bars should be designed to meet MSJC Section 2.2.3.2.

- Load combinations with stress increase

![Diagram image]
**MASONRY -- Concrete Masonry Design System**

**National Concrete Masonry Association**

**Prjct:** Dutton Christian School  
**Topic:** Calculated Loads  
**Name:** Rama Suparta  
**Date:**  
**Chkd:**

---

**Design of a Reinforced Masonry Wall**  
**Using the 2002 MSJC ASD Design Code**

**Wall Support:** Simply Supported Wall

### Specified Load Components

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<tr>
<th>Load</th>
<th>$W_1$ (lb)</th>
<th>$W_2$ (psf)</th>
<th>L (in)</th>
<th>h1 (in)</th>
<th>h2 (in)</th>
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### Load Reaction Components

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<th>Rv (lb)</th>
<th>MT (lb-in)</th>
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### Section Forces at Bottom

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<th>V (lb)</th>
<th>P (lb)</th>
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## Project: Dutton Christian School
### National Concrete Masonry Association

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Appendix H – Second to Last Floor Plan
DUTTON CHRISTIAN ELEMENTARY SCHOOL

6980 HANNA LAKE SE
CALEDONIA, MICHIGAN

EAST BUILDING ADDITION

CIVIL

C1 SITE PLAN

ARCHITECTURAL

A1 ARCHITECTURAL DEMOLITION PLAN
A2 EXTERIOR ELEVATIONS
A3 FLOOR PLAN
A4 ROOF PLAN
A5 WALL / GENERAL SECTIONS
A6 FINISH FLOOR PLAN
A7 WALL FINISH PLAN
A8 REFLECTED CEILING PLAN

STRUCTURAL

S1 FOUNDATION PLAN
S2 BEARING WALL SECTIONS
S3 FOUNDATION DETAILS
S4 LINTEL PLAN
S5 ROOF FRAMING PLAN
S6 STRUCTURAL DETAILS

CALVIN COLLEGE ENGINEERING
TEAM 9: ADDITION AMBITION

Kevin Koning, Rama Suparta, Mike DeJong, Mike Kooy