COMMERCIAL VERTICAL FARM DESIGN

PHASE 1

California Polytechnic State University

Spring 2016

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Abstract

Until recent years, vertical farming, or urban agriculture, has usually been referred to as more of a science fiction concept than a realistic means of production. However, it is becoming increasingly apparent that these urban crop-growing structures are not only technologically feasible, but efficient. These structures are entering the realm of economic feasibility as well, appearing in niche markets near large cities. Most of the facilities, however, are generally restricted in size to single-story buildings. The objective of this project was to design a multi story structure that would be well-equipped to house a large vertical farming operation. Some of the key aspects of the design are the use of natural light, structural stability, and minimizing the costs of construction. This report details the decision-making process behind the development of the structural design. Included are detailed explanations of the methods used to find an efficient configuration for the building that allows the maximum possible amount of natural light to be used by the facility. Significant deviations from traditional structural design were made when doing so would significantly increase the level of light allowed to be used inside the building. Details are included regarding the structural, economic, and technological challenges faced when coming up with this design. The results seem to indicate that an efficient structure is a plausible reality and could be implemented, and that this field contains a vast amount of research opportunities to come.

SECTION I

CONCEPT DESIGN

Literature Review

Farming is a practice that allows us as a society to produce enough food, and other products, to supply the demand of a vast number of people. With expanding population growth our farming practices have become extremely efficient and have expanded its own land use. With urban populations expanding there has been trouble to find and cultivate enough land to meet the ever growing demand that is occurring. On top of that transportation for a number of these products can get expensive as food is often grown far away from these urban areas. Vertical farming or indoor farming is a diverse field of technology that is used to produce food, drugs, and other products in, or near, urban environments and is one way we can tackle this growing problem (Mougeot, 2000).

Vertical farming comes in many different forms and many scholars have different ideas on how it should best be designed and implemented into these urban environments. Dr. Despommier of Columbia University believes that the ways indoor farming should be done is through tall multi-story structures. He claims that one vertical farm with an architectural footprint of one square city block and rising up to 30 stories could provide enough nutrition to comfortably accommodate the needs of 10,000 people with the currently available technologies (Despommier, 2013). However he believes that farming plants is the limit to his concept of vertical farming and that “Cattle, horses, sheep, goats, and other large farm animals seem to fall well outside the paradigm of vertical farming” (Despommier, 2013). Despommier overall believes that vertical farming is a part of handling these problems of land scarcity, food demand, and transportation. He wants the entire urban environment to be able to sustain itself on food, waste, and pollution management and thinks that vertical farming can have a huge impact on all of these areas (Despommier, 2014).

Although the people at Green Sense Farms are not as concerned with pollution or waste management effects of vertical farms could provide they are in agreement in that vertical farming is a stepping stone in indoor plant production. At Green Sense Farms they have taken indoor farming to the operational level and have constructed a 30,000 square foot facility and designed with their own technology. What makes this facility special is the use of a completely enclosed and controlled environment. In this facility they use a powerful air recirculation system that filters the air, treats it for microbes' with UV radiation, and adds more CO_2 to allow for increased plant growth (Bhanoo, 2014). This facility also treats its water the same way continuously checking its nutrients, pH and electro conductivity for optimal plant absorption (Bhanoo, 2014). This facility is state-of-the-art but that is because funded and backed by MIT and is in a unique position to make this technology open source, to prototype rapidly (Bhanoo, 2014). Robert Colangelo, the president and founder of Green Sense Farms has stated that "This facility is like a Ferrari racecar designed specifically to optimize leafy greens, if we were to branch off into another direction we would have to redesign our system."(Bhanoo, 2014). This facility is on the leading edge of vertical/indoor farming technologies and is not a cost effective solution or model quite yet with their experimental technologies.

Although technologies in other scholar's facilities or designs are not put into practice to be economically viable quite yet there are some structures that have begun its' first attempt to make a sustainable vertical farming operation. The Sky Greens facility in Singapore is the first commercial vertical farm that was built and designed to begin to offset costs of importing vegetables (Shirk, 2015). This system is a unique technology that slowly moves platforms dipping them into nutrient rich baths that run off of less than $3 of electricity a month per tower (Shirk, 2015). This technology and facility is the world's first look at vertical farming on a commercial scale to see if it is an economically viable concept. This is the goal to start creating facilities and try to find ways to make these concepts more and more efficient to become viable enough to make a real impact in our upcoming problems of population growth, land scarcity, food demand, and waste

Evaluation of Natural Light Penetration

Maximally efficient use of natural light is critical in the case of a multistory facility. The location of the building can greatly affect the patterns of light exposure on the interior. Depending on the site latitude on the planet, a design in one location may not necessarily work in another distant location. In the case of a temperate zone like San Luis Obispo, the facility is located far north of the Tropic of Cancer, which results in significant horizontal light exposure from the South, East, and West. As a result, the facility can grow crops in natural light on three sides of the building.

Since the sun's rays are in motion and always at an angle at all times of the year, the level of light exposure on any given level will follow a general pattern of more light on the outside and gradually less in the interior areas. On a building with a side facing directly south, the area that receives full sunlight is at a 79 degree angle from the vertical, projected down from the floor above. The sun will hit that peak angle during the summer solstice, with the rest of the days having more horizontal penetration of sunlight into the interior of the structure.

Using the rendering tool in Revit 2015, the level of sunlight penetration into a model story in a building is measured by an approximated source of sunlight. The renderings were created by taking a rendered snapshot of the model for every hour of daylight on several days. The particular days studied were the summer and winter solstices, and the fall and spring equinoxes. For each day studied, a series of images was created from the horizontal penetration levels throughout the day, illustrating the areas exposed to natural light throughout the day. (see page)

This was taken a step further when the areas of natural light exposure were calculated mathematically. Using data from several Internet databases, the position of the sun was taken at 15 minute intervals throughout the day for several dates of interest. From there, formulas created in a Mircosoft Excel spreadsheet were applied to generate the level of light penetration quickly. The levels of light exposure were calculated by hand from observation of the spreadsheet data.

For any given interval of time, there is a corresponding distance into the building that will be exposed to light for at least that amount of time. For example, a region with a 4 hour light rating would be exposed to a minimum of 4 hours of light throughout the day. However, some parts of the region would be exposed to direct sunlight for a significantly higher amount of time. Therefore, plants could be cycled through a region continuously such that the average level of light exposure would remain at 4 hours. The movement of plants would extend the amount of growable area inside the building.

This ideal depth to give an average exposure of 4 hours could be calculated with the data from the spreadsheet. Plotting the time on the x axis with the depth on the y axis and then using a curve fitting tool, a model equation was generated to approximate the data points. A simple integral with respect to x yields the total light exposure. The maximum depth at 4 hours is the average depth over the interval from 0 to 4 hours. This number varies depending the orientation of the building, the site location, and the time of year, which is why each day of the year yields a unique data set.

From the data, the question of building orientation and shape became apparent: south-facing or east/west-facing walls? South-facing walls dominate in the winter while walls facing in the other directions excel in summer months. Considering several days of the year as dates marking the beginning or end of a growing season, it was determined that east-west facing walls could be utilized for more growing seasons effectively, including the summer months where sunlight is more intense. There is still a portion of south-facing wall that can be used for growth as well.

The growth areas vary by season, with the ideal depth of the crop-raising system varying drastically between summer and winter. Each calendar year could be broken up into eight growing seasons of about 45 days each, with solstices and equinoxes defining most of the seasonal boundaries. Since the building is oriented with the primary growth zones facing east and west, the operation of the facility depends mostly on those areas. In the winter months, those faces of the building receive almost no consistent light, with effective depth as little as one foot on the winter solstice. It would probably be more effective to simply have two off seasons that span from November 6th to February 4th than to attempt growing that small amount of produce. The south face of the building could still house a crop supply that is ample enough to continue operation in the winter months. During the seasons where southern light penetration is more effective than east and west combined, the corner bays on the southern end of the building could be reoriented to grow for south-facing crops instead of east-west even during regular growing seasons. Alternatively, having a single winter off-season centered on the winter solstice could yield seven growing seasons a year instead of six, with November and January growing seasons outputting relatively small but still worthwhile crop yields. Ultimately, this is a consideration that will ultimately be resolved when economics are brought into play at a later point.

Hydroponic System Design

The primary purpose of selecting a hydroponic system instead of in-house soil beds is to significantly reduce the overall weight of the structure. Achieving this sort of reduction in the need for heavier structural members over a large area would drastically reduce the amount of material used, and by extension, the overall cost of construction.

The basic design of the system depends on using PVC pipes spanning up to 30 feet to house the plants. Each pipe would have holes drilled at the top, typically spaced about 8 inches apart in order to allow ample growing space for the plants. The pipe would also serve as part of a distribution system that delivers nutrient-enhanced water to the plants. The spinach plants themselves would each be placed in a small, nutrient-rich container of soil that can be easily snapped into one of the holes on the PCV pipe. The container would also be porous in order to allow further absorption of nutrients flowing through the PVC pipe during operation. A typical pipe spanning 30 feet would support approximately 40 plants, and would be fitted into its place on a moving support system that allows the seed containers to be easily changed out after each growing season.

The ends of the pipes would be fitted into two different movable conveyor systems on each side of the pipe. Each pair of conveyors could house up to 50 pipes, depending on the ultimate light penetration into the building. These conveyors would be compact and framed onto light supports placed directly above the structural steel girders. Ideally, these conveyors would also contain small hoses that supply each pipe directly with water and nutrients, thus minimizing the need for a complex network of pipes connecting each pipe to a water source separately. Instead, the conveyor hoses would take water directly from the storage tanks housed in the core of the building.

For optimal efficiency throughout the year, the conveyor, support, and irrigation systems could be designed as a modular system with the ability to be easily assembled and disassembled. With the average light depth moving significantly every season, the conveyer modules would have to be capable of rapid reconfiguration by a crew of workers in just a day or two. Conveyors could be manufactured in eight or sixteen inch sections and linked together mechanically to create a system of any desired depth. This way, the target depth of light penetration would always be matched to the length of the conveyors. During the winter, when the east-west walls are in an off-season, the system could be quickly reconfigured to house more plants on the south-facing wall for a season or two. The output volume would not be as high, but it could be economically worthwhile to keep that portion of the building in operation during the winter off-season.

Choice of Produce

Spinach is the choice item for this project due to its fast harvest time, small size, and ability to flourish in partial shade. Most common species of this plant only require 3-4 hours of sunlight in order to effectively grow just as well as in full sunlight. Therefore, placing plants on the east-facing and west-facing sides of the building is a viable design despite the fact that those faces only receive roughly half the sunlight as an open space would.

Other crops with similar properties could also be grown in place of the spinach plants. The only required changes would be in the layout of the crop itself in order to optimize light exposure for that specific crop. Potential alternate crops to be used in the same system would include other leafy greens (cabbage, lettuce, etc.), legumes (peas, beans, etc.), and herbs (basil, rosemary, etc.). The only changes accompanying a change in crops would be altering the chemical composition of the nutrient solution fed to the plants. This would have an effectively negligible effect on the total weight inside the structure as long as the total water storage requirements did not exceed the designed capacity.

Material Considerations

Steel allows some of the smallest sections per unit strength and therefore the most light penetration, so the primary structural material will consist of hot rolled sections. Considering the humid nature of the indoor environment, the idea of coating the structural steel members in a corrosion-resistant material seems beneficial. The added weight to the structure is minimal, but the resulting extension of the building's serviceable lifespan would be substantial. Significant portions of the building--such as the diaphragm trusses--can also be prefabricated in a shop and shipped out to the site, which yields both higher quality and lesser cost.

The decking system benefits the overall design of the facility if it allows light to penetrate through, extending downward for several floors. An open system such as premade steel or fiberglass grates would allow such penetration far more than a solid diaphragm like concrete or wood. Unlike residential or other building classes designed for constant use, this facility would only serve a small maintenance crew for planting, upkeep, and harvesting the crops. Therefore, an open system would not be a significant issue in terms of occupation. It is also lighter in weight when compared to solid diaphragm systems, so it would lessen the total weight of the structure.

Structural Design Considerations

The primary goals of the structural design are to achieve the maximum possible level of natural light exposure, to minimize use of materials, and to minimize the total cost of the facility.

The floor system is also intended to allow increased exposure to sunlight as it penetrates from the floors above. An open grate system was preferable to a traditional concrete and steel deck floor system, which blocks vertical light sources entirely. Among the options for an open system, both steel bar grating and Gator Deck fiberglass grating were considered.

Gravity Beams and Girders

The dead load was kept relatively light because of the lack of concrete decking throughout the building. The major contributors were the steel grate decking and the infrastructure holding up the rows of plants (which counts as dead load because it is to be anchored to the structure itself). The live load was assumed at 80 psf (corridor loading) for all areas due to the potential occupancy conditions. The usual occupancy would be only a handful of maintenance staff carrying light equipment, which only warrants a live load of up to 50 psf. The critical loading occurs either when a tour group passes through the building, or when additional workers are brought in during a harvest. The harvest condition also conflicts with the use of live load reduction as specified in ASCE 7-10. The live load reduction section assumes that large areas are seldom loaded to the absolute maximum capacity in every location, so reducing it is justified. Some exceptions are listed in which live load cannot be reduced in the case of very heavy loads, but corridor loading is not among them. However, a harvest day may actually bring live load conditions very close to that, which warrants restricted implementation of the live load reduction formula. Reduction was ultimately still used, but beams loaded to greater than 90 percent of full capacity were not used in the final design just in case the beams needed a bit of extra strength for a load that exceeded the condition prescribed by the live load reduction.

In the core bays, live load reduction was not used because it was expected that all of the large water tanks would be filled simultaneously. That scenario defeats the purpose of the live load reduction, which is why it was not justifiable to implement in the center of the building. The load condition is more akin to a storage area, which is an area that is not eligible for live load reduction in the code. The core bays were considered to have nonreducible live load, which made the beams considerably larger and deeper. This drops the ceiling clearance height in that area by a few inches, but does not obstruct the movement of people throughout that area or the natural light getting to the growth areas.

Columns

The vertical loads going to the exterior columns are relatively light, and therefore demand lighter column sections. A typical W8 section used in compression has a width of about 8 inches, but HSS sections can be kept as narrow as 6 inches and still support the demand loads. Therefore, columns lining the perimeter of the building were designed as HSS 6x6 sections in order to boost the total net area of wall that allows light into the building. In the core of the building, where loads from the water storage area contribute to significantly heavier column loads, HSS sections would have to be much larger. For these areas, wide flange sections were used since HSS sections lose efficiency at higher levels of axial loading, and because light exposure in a storage and maintenance area is not as important.

Roof Canopy

At the highest level of the structure, natural light is effectively as abundant as it would be in an open area. This level would ideally house crop-growing systems in all areas, with water storage and maintenance areas kept at a minimum. The only structure needed above the floor level would be a lightweight canopy to enclose the area in order to regulate temperature and humidity in the facility. A traditional roof structure that consisted mostly of concrete over steel deck would block out much of the light that could potentially used to grow crops across the entire floor area of the uppermost floor. The structure would essentially be a traditional greenhouse housed on the top of the building. The walls and roof would be composed of the same polyethylene panels that act as cladding in the rest of the building. In addition to allowing more light than a standard roof composed of concrete decking, it would also have a fraction of the weight. The roof would also not be expected to have any live load applied to it since the panels are not designed to support the load of workers servicing them from above. Instead, the roof panels could be serviced by removing them from below and performing maintenance on them at ground level.

The structural system supporting the panels would be a series of long-spanning, light steel beams. The primary beams would be spaced at 6’-8” and would be WT sections. The girders, too heavily loaded to use a light weight WT section, would most likely be double angles. Both section types allow the light panels to be easily and comfortably inserted into the space between steel beams. They could be easily placed and removed from that position, but still offer an effective form of insulation from the outside environment. This results in a design that is far lighter than traditional roof systems, is made of cheaper materials, and is only loaded heavily by wind pressures. The walls throughout the building could employ a similar system that would be designed for wind loads.

Wind & Seismic

The basic seismic parameters were analyzed for coordinates on Cal Poly property in the agricultural area. Specifically, the building’s model site is located northwest of the core of campus. across the street from the Cal Poly poultry unit near Highway 1. There are no known fault lines in the immediate vicinity, but the area is still a high seismic zone as expected in coastal California. The building was designed to a Seismic Design Category E.

Wind parameters were determined from basic conditions outlined in ASCE 7-10 for the same site. The area has moderately hilly terrain, occasional growths of trees, and several buildings in close proximity. With those obstructions in place, the building was assigned to wind exposure category C. Category D is usually reserved for areas with effectively no wind barriers such as large flat plains, or areas on the ocean; category B mostly applies to tightly packed urban areas. The total wind speed on the leeward side was calculated to be 27 psf, a number used throughout the analysis of the diaphragm.

Wind controlled the overall lateral base shear in the east-west direction while seismic controlled in the north-south direction. Thus, the corresponding loading on the brace frames in each direction was based on those numbers. However, with the design of the diaphragm, the magnitude of the forces varied more subtly. In the case of seismic loading, a large fraction of the base shear was concentrated at the top floor, while wind load had a more equitable distribution to all the floors.

Diaphragm

Ordinarily, the floor system resting upon steel beams would be corrugated steel decking with a relatively thin layer of concrete on top. The concrete can adequately resist and transfer shear forces that come to it. This floor system functions easily as a diaphragm to resist lateral forces in any given story, but it does block out sunlight from above. An alternate floor system could effectively “recycle” light from near-overhead directions as it could extend downward through multiple floors. The open grate floor system does not provide a reliable system capable of functioning as a rigid diaphragm similar to concrete. A series of braces beneath the floor system would still provide ample light exposure to floors below, but would achieve a level of stiffness comparable to the traditional concrete diaphragm.

The diaphragm used in this design was the most non-conventional system in the entire building, considering that there was not a stable floor system in place to act as a truly rigid structure capable of transferring force to the places it was intended to go. Instead, a system of small, diagonal braces was put in place of a solid floor system to transfer loads laterally around each floor. The decision to use a design like this is consistent with one of the end goals of maximizing the exposure to natural exterior lighting, which is blocked by a solid floor system. An open truss would allow upwards of 90 percent of vertical light to penetrate through, with the only sources of opacity existing as either small angle sections or thin steel bars.

In the east-west direction, the method of analysis used relies on locating the single 30-foot span of truss with the highest stress level. The truss bearing that load is located in the far north bay, which is essentially a cantilever since it is not between two rows of collectors. On the short span of 30 feet, that single truss must bear the entire wind load striking the exterior since there is no way to distribute the load to multiple trusses. The windward pressure on that single truss far exceeds the total seismic or wind base shear distributed among six or more rows of truss throughout an entire floor. Therefore, analyzing that small truss and designing for that force across the entire diaphragm is conservative. Once the lateral wind loads are transmitted to the ends of the small span trusses, the girders in the east-west direction act as distributors, spreading the load out over several trusses. On that scale, all of the small trusses behave uniformly as a single diaphragm over the full span between the two rows of vertical brace frames.

In the north-south direction, seismic lateral force was the clear governing factor over wind, so base shear was distributed according to relative mass. Initially, the north-south spanning trusses were linked in two locations across the entire building at bays near the braced frames. This establishment of continuity across the whole structure justified calculating the lateral forces based on rigid diaphragm behavior. The segments of mass on the outside bays were effectively cantilevered outside of the braced frames surrounding the innermost row of bays. The resulting force from those outside areas would have to be concentrated on two short spans of truss in order to be transferred onto collectors on grid lines B and C.

The lateral trusses would be completed by utilizing the existing gravity system beams as the chords of each truss. The truss elements were initially placed so that every beam would be attached to some form of bracing, thus reducing the potential for lateral torsional buckling drastically. Reducing the unbraced length of each beam from 30 feet to 10 feet provided a source of bracing to strengthen the gravity system while outlining the design of the diaphragm. The original idea was to use channel sections as these short braces, but were discovered to have slenderness issues at smaller sizes. Single angle sections were eventually selected as the short braces because they have a higher radius of gyration in both directions than a channel section at a small size, which eliminates the excessive slenderness and potential for compressive buckling.

The diagonal cross braces, measuring up to 12 feet in length, were also conceived of as channel sections originally, but were not efficient at that length due to slenderness issues. The single diagonal brace designed for in the initial analysis was eventually replaced with two diagonal cross braces in each bay, which would be designed simply as metal bars. Although nearly useless in compression, the bars had a strong tensile capacity and could carry the loads while retaining low weight and cross sectional area. The change in design is justified by the existing single-brace calculations because only bars in tension would be considered viable carriers of load. The initial analyses showed all diagonal bars in compression, but simply switching the direction of loading would yield forces of the same magnitude in tension. Since the truss is symmetric and the number of bars in tension is identical regardless of force direction, the calculations are accurate in terms of the magnitude of tensile force resisted by the bars.

For the analysis of shear loads to the chords and collectors, wind and seismic were both in consideration. Wind may have governed net base shear on the entire building, but the distribution of seismic forces was more heavily concentrated at the upper levels of the building, which led to seismic force being the controlling factor in both directions for a single floor.

Lateral System

In order to maximize light penetration, the goal is to bring the lateral system inside as much as possible in order to avoid interference with the natural light penetration. Such systems considered included an interior core of concrete shearwalls, exterior and interior steel brace frames, and exterior moment frames. The design of the lateral force resisting system consists of balancing the light penetration issue and the torsion issue. Brace frames were ultimately selected as the system type, but their location was not conventional because placing them at the edges of the building would block out more light. If located on the outside face of the building, the combination of larger gravity members, braces, and potentially large gusset plates would result in a significant reduction of natural light coming from the outside, with sometimes as much as 15% of the total window area compromised . Moreover, the presence of diagonal braces would introduce inconsistent light patterns that would reflect in the growth patterns of the crops. Such an unpredictable growth pattern would make maintenance and harvesting exceedingly difficult on the farmers operating the facility. For these reasons, the brace frame systems were brought away from the edges of the building to interior bays; the the added complications of torsion and internal congestion were the consequences of switching to this design.

In the east-west direction, the rows of brace frames were simply moved from the northern and southern edges of the building to a location one grid line toward the center. The brace frames are perpendicular to the face of the building, thus causing minimal light interference. The braces spanning crop-growing areas are also oriented so the end near the wall is anchored at ceiling level, and the interior end at floor level. This was done to ensure that light entering the building at an angle was blocked out as little as possible. This does create an issue in which movement in the crop-growing areas is limited by the braces, which is resolved by corridors being routed through the central row of bays. The center bay would also have a brace across it, but there is still ample space for doorways allowing movement in that area. On the north side, the missing bay in the center would restrict movement to the far north bays, which is why a short balcony was inserted to allow that movement. The beam would be a W10x26 to match the typical growth bays, which is adequate because the balcony beam's load demands are less than that of the beams in crop-growing areas. The balcony would be used as a corridor with a tributary area much smaller than that of W10x26 girders of the same span length. Additionally, it would not expected to house large water tanks or crops, leaving only the corridor live load, which is identical to the one assumed in the other areas, thus justifying the low load demands.

In the north-south direction, a similar action was taken to bring the brace frames inside the building to the core area, but doing so in this axis gave rise to another issue: torsion. With the frames placed in the central grid lines, the distance between them was reduced to 20 feet, or one third of the total building width. This design choice works because the building was designed to be highly symmetric on this axis anyway. Only the accidental torsion of five percent specified in the ASCE 7-10 contributed to the rigid diaphragm twisting. The effects of torsion in a level were actually kept relatively low. In terms of the brace layout itself, two braces per bay were designed similarly to a zipper frame, but without intermediate columns. This cuts down on slenderness of the braces in compression and increases redundancy of the lateral system in this direction.

Foundations

The only foundations required at first would be isolated footings at the base of the columns since there are no structural walls present in the design. The default system was considered to be a simple reinforced concrete pad footing beneath each column. The typical bearing pressures for soil per the CBC 2013 (identical to 2012 IBC) were used, with stiff soil being the assumed soild type. This worked for the exterior columns on the building, with square footings of side length 11 feet. The interior column footings however, would have required a side length of more than 20 feet. A deep-foundation solution such as a concrete caisson could be more effective in supporting the load, so that route was taken in the calculations. In the case of the building experiencing overturning forces, the deep foundations are also functional for tension resistance. A simple pad footing simply cannot provide this type of resistance, so deep foundations were used below any column where overturning was likely to occur. If buildings like this were to be actually constructed, an analysis by a geotechnical engineer would be valuable when designing facilities in excess of 5 stories such as this one.

Economic Considerations

Given the complex nature of the motion of sunlight throughout the day, it would be intuitive that an unusual shape would best optimize the building for light penetration. However, considering the nature of operating such a facility, keeping the building rectangular would make management and usage far more simple and streamlined. The rectangular bays allow the conveyor systems to be easily reconfigured for different growing seasons to maximize production. For example, southern exposure is more favorable during the winter, and therefore the corner bays should be oriented southward instead of east and west.

Additionally, some material choices such as fiberglass decking and polyethelyne were chosen because of their economic advantages. Using a deck system made of a material that is already corrosion resistant in a humid environment is superior to painstakingly coating a vulnerable steel grate system in an anti-corrosion covering. The fiberglass does not have the spanning capabilities that steel does, but adding an extra beam per bay for support was economically favorable to the added labor costs of protecting the steel decking from corrosion. The center bays were an exception, however, because the fiberglass decking could not support the heavy load of water storage tanks. The polyethelyne panels are economically favorable over glass because they are lighter, cheaper, and easier to replace. Further phases of projects like this will be subject to a wider range of options regarding the control of construction and operation costs in the facility.

SECTION II

STRUCTURAL CALCULATIONS

STRUCTURAL DESIGN OVERVIEW

Building Codes Considered

ASCE 7-10
CBC 2013
AISC Steel Design Manual, 14th Edition
ACI 318-04

Sds=0.791g
Sd1=0.450g
Soil Classification: D
Seismic Design Category: E

Basic Wind Speed: 110 MPH
Exposure Category: C

Materials

Wide Flange Steel Sections: ASTM A992 Steel (50 ksi)
Hollow Structural Sections: ASTM A500 Steel (46 ksi)
Single and Double Angle Sections: ASTM A36 (36 ksi)
WT Steel Sections: ASTM A36 (36 ksi)
Solid Steel Bars: ASTM A36 (36 ksi)
Normal Weight Concrete: 4 ksi
Reinforcing Steel: ASTM Gr. 60 (60 ksi)
Fiberglass Decking: Gator Deck T5020 Grade
Steel Decking: 15-W-2: 2-½” x 3/16”(15-2-103) Welded Steel Bar Grating

S2

Design Maps Summary Report

User–Specified Input

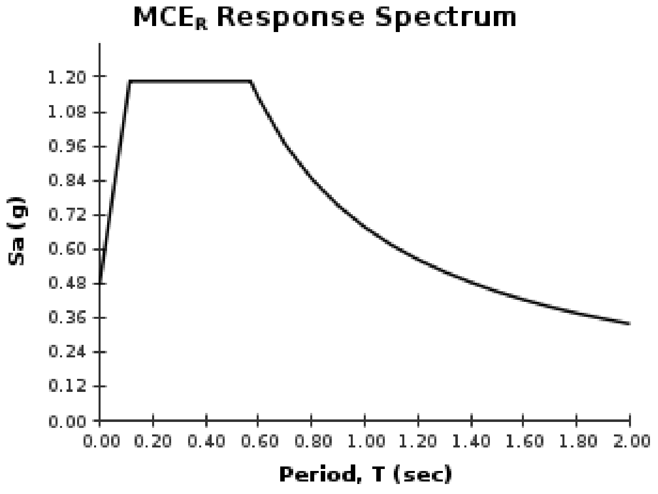
Report Title	Cal Poly Vertical Farm Project Mon May 2, 2016 07:03:12 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	35.3128°N, 120.68°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

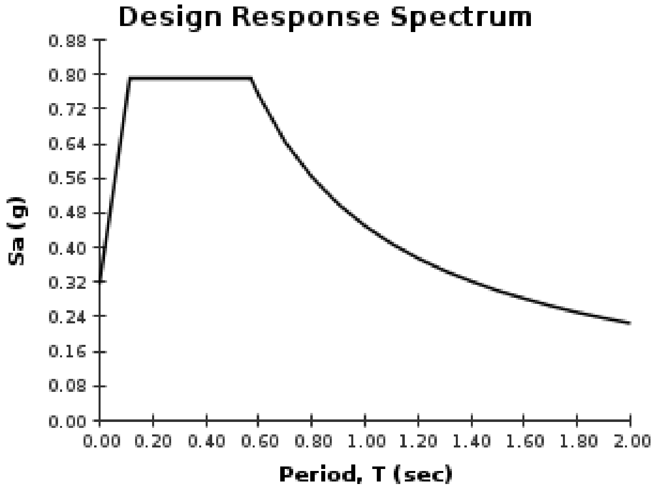
USGS–Provided Output

S_S = 1.133 g **S_{MS} =** 1.186 g **S_{DS} =** 0.791 g

S_1 = 0.430 g **S_{M1} =** 0.676 g **S_{D1} =** 0.450 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.


MCE$_R$ Response Spectrum

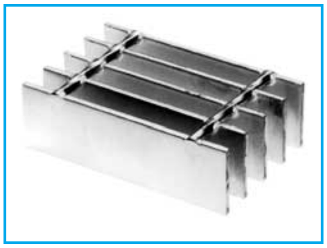

Design Response Spectrum

For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

Steel Bar Grating

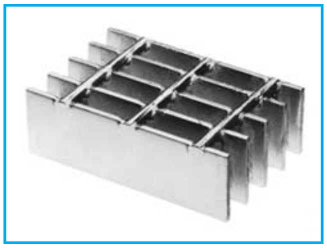
WELDED

15/16" Center to Center of Bearing Bars



15-W-4

Cross Rods 4" C/C

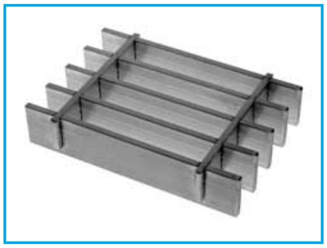


15-W-2

Cross Rods 2" C/C

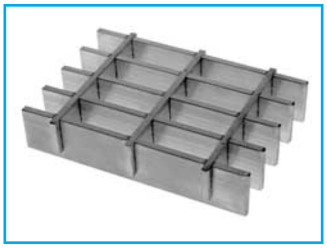
PRESS-LOCKED

15/16" Center to Center of Bearing Bars



15-P-4

Cross Bars 4" C/C



15-P-2

Cross Bars 2" C/C

LOAD & DEFLECTION TABLE

Bar Size	Symbol		Approx. Weight psf	Sec. Mod Per Ft. Of Width		SPAN (Length of Bearing Bar) 2'-0"	2'-6"	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	8'-0"	9'-0"
3/4" x 1/8"	15-4-32	W	4.7	0.150	U	450	288	200	147	113	89							
		P	5.1		D	0.099	0.155	0.223	0.304	0.397	0.503							
	15-2-32	W	5.3		C	450	360	300	257	225	200							
		P	6.1		D	0.079	0.124	0.179	0.243	0.318	0.402							
3/4" x 3/16"	15-4-33	W	6.9	0.225	U	675	432	300	220	169	133	108						
		P	7.7		D	0.099	0.155	0.223	0.304	0.397	0.503	0.621						
	15-2-33	W	7.5		C	675	540	450	386	338	300	270						
		P	9.1		D	0.079	0.124	0.179	0.243	0.318	0.402	0.497						
1" x 1/8"	15-4-42	W	6.1	0.267	U	800	512	356	261	200	158	128	106	89				
		P	6.5		D	0.074	0.116	0.168	0.228	0.298	0.377	0.466	0.563	0.670				
	15-2-42	W	6.7		C	800	640	533	457	400	356	320	291	267				
		P	7.5		D	0.060	0.093	0.134	0.182	0.238	0.302	0.372	0.451	0.536				
1" x 3/16"	15-4-43	W	8.9	0.400	U	1200	768	533	392	300	237	192	159	133				
		P	9.8		D	0.074	0.116	0.168	0.228	0.298	0.377	0.466	0.563	0.670				
	15-2-43	W	9.6		C	1200	960	800	686	600	533	480	436	400				
		P	11.2		D	0.060	0.093	0.134	0.182	0.238	0.302	0.372	0.451	0.536				
1-1/4" x 1/8"	15-4-52	W	7.5	0.417	U	1250	800	556	408	313	247	200	165	139	118	102		
		P	8.2		D	0.060	0.093	0.134	0.182	0.238	0.302	0.372	0.451	0.536	0.629	0.730		
	15-2-52	W	8.1		C	1250	1000	833	714	625	556	500	455	417	385	357		
		P	9.5		D	0.048	0.074	0.107	0.146	0.191	0.241	0.298	0.360	0.429	0.504	0.584		
1-1/4" x 3/16"	15-4-53	W	11.0	0.625	U	1875	1200	833	612	469	370	300	248	208	178	153		
		P	12.3		D	0.060	0.093	0.134	0.182	0.238	0.302	0.372	0.451	0.536	0.629	0.730		
	15-2-53	W	11.6		C	1875	1500	1250	1071	938	833	750	682	625	577	536		
		P	14.2		D	0.048	0.074	0.107	0.146	0.191	0.241	0.298	0.360	0.429	0.504	0.584		
1-1/2" x 1/8"	15-4-62	W	8.9	0.600	U	1800	1152	800	588	450	356	288	238	200	170	147	113	89
		P	9.6		D	0.050	0.078	0.112	0.152	0.199	0.251	0.310	0.376	0.447	0.524	0.608	0.794	1.006
	15-2-62	W	9.4		C	1800	1440	1200	1029	900	800	720	655	600	554	514	450	400
		P	10.9		D	0.040	0.062	0.089	0.122	0.159	0.201	0.248	0.300	0.358	0.420	0.487	0.636	0.804
1-1/2" x 3/16"	15-4-63	W	13.1	0.900	U	2700	1728	1200	882	675	533	432	357	300	256	220	169	133
		P	14.4		D	0.050	0.078	0.112	0.152	0.199	0.251	0.310	0.376	0.447	0.524	0.608	0.794	1.006
	15-2-63	W	13.7		C	2700	2160	1800	1543	1350	1200	1080	982	900	831	771	675	600
		P	16.3		D	0.040	0.062	0.089	0.122	0.159	0.201	0.248	0.300	0.358	0.420	0.487	0.636	0.804
1-3/4" x 3/16"	15-4-73	W	15.2	1.225	U	3675	2352	1633	1200	919	726	588	486	408	348	300	230	181
		P	16.5		D	0.043	0.067	0.096	0.130	0.170	0.215	0.266	0.322	0.383	0.450	0.521	0.681	0.862
	15-2-73	W	15.8		C	3675	2940	2450	2100	1838	1633	1470	1336	1225	1131	1050	919	817
		P	18.4		D	0.034	0.053	0.077	0.104	0.136	0.172	0.213	0.257	0.306	0.360	0.417	0.545	0.689
2" x 3/16"	15-4-83	W	17.3	1.600	U	4800	3072	2133	1567	1200	948	768	635	533	454	392	300	237
		P	18.6		D	0.037	0.058	0.084	0.114	0.149	0.189	0.233	0.282	0.335	0.393	0.456	0.596	0.754
	15-2-83	W	17.9		C	4800	3840	3200	2743	2400	2133	1920	1745	1600	1477	1371	1200	1067
		P	20.5		D	0.030	0.047	0.067	0.091	0.119	0.151	0.186	0.225	0.268	0.315	0.365	0.477	0.603
2-1/4" x 3/16"	15-4-93	W	19.4	2.025	U	6075	3888	2700	1984	1519	1200	972	803	675	575	496	380	300
		P	20.7		D	0.033	0.052	0.074	0.101	0.132	0.168	0.207	0.250	0.298	0.350	0.406	0.530	0.670
	15-2-93	W	20.0		C	6075	4860	4050	3471	3038	2700	2430	2209	2025	1869	1736	1519	1350
		P	22.6		D	0.026	0.041	0.060	0.081	0.106	0.134	0.166	0.200	0.238	0.280	0.324	0.424	0.536
2-1/2" x 3/16"	15-4-103	W	21.4	2.500	U	7500	4800	3333	2449	1875	1481	1200	992	833	710	612	469	370
		P	22.7		D	0.030	0.047	0.067	0.091	0.119	0.151	0.186	0.225	0.268	0.315	0.365	0.477	0.603
	15-2-103	W	22.0		C	7500	6000	5000	4286	3750	3333	3000	2727	2500	2308	2143	1875	1667
		P	24.7		D	0.024	0.037	0.054	0.073	0.095	0.121	0.149	0.180	0.215	0.252	0.292	0.381	0.483

U = safe uniform load, psf
C = safe concentrated load, pfw
D = deflection, inches
E = modulus of elasticity, 29,000,000 psi
F = fiber stress, 18,000 psi

Material: ASTM A-569 standard

Deflection: Spans and loads to the right of the bold line exceed 1/4" deflection for uniform load of 100 psf which provides safe pedestrian comfort. These can be exceeded for other types of loads with engineer's approval.

Serrated Bars: For serrated grating, the depth of grating required for a specified load is 1/4" deeper than that shown in the table.

General: Loads and deflections are theoretical and based on static loading.

W/P-15 PANEL WIDTH (inches)

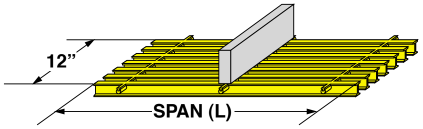
Note: P-Press Locked cross bars typically extend 1/8" each side. W-Welded cross rods may extend 1/8" each side. Panel widths do not include these extensions.

No. of Bars	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
1/8" Bar	1 1/16	2	2 15/16	3 7/8	4 13/16	5 3/4	6 11/16	7 5/8	8 9/16	9 1/2	10 7/16	11 3/8	12 5/16	13 1/4	14 3/16	15 1/8	16 1/16	17	17 15/16
3/16" Bar	1 1/8	2 1/16	3	3 15/16	4 7/8	5 13/16	6 3/4	7 11/16	8 5/8	9 9/16	10 1/2	11 7/16	12 3/8	13 5/16	14 1/4	15 3/16	16 1/8	17 1/16	18
No. of Bars	**21**	**22**	**23**	**24**	**25**	**26**	**27**	**28**	**29**	**30**	**31**	**32**	**33**	**34**	**35**	**36**	**37**	**38**	**39**
1/8" Bar	18 7/8	19 13/16	20 3/4	21 11/16	22 5/8	23 9/16	24 1/2	25 7/16	26 3/8	27 5/16	28 1/4	29 3/16	30 1/8	31 1/16	32	32 15/16	33 7/8	34 13/16	35 3/4
3/16" Bar	18 15/16	19 7/8	20 13/16	21 3/4	22 11/16	23 5/8	24 9/16	25 1/2	26 7/16	27 3/8	28 5/16	29 1/4	30 3/16	31 1/8	32 1/16	33	33 15/16	34 7/8	35 13/16

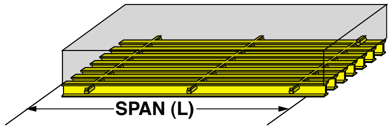
GATORDECK T-5020 – Load & Deflection Table

LOAD = Lbs. / Sq. Ft.

CONCENTRATED LOAD



UNIFORM LOAD



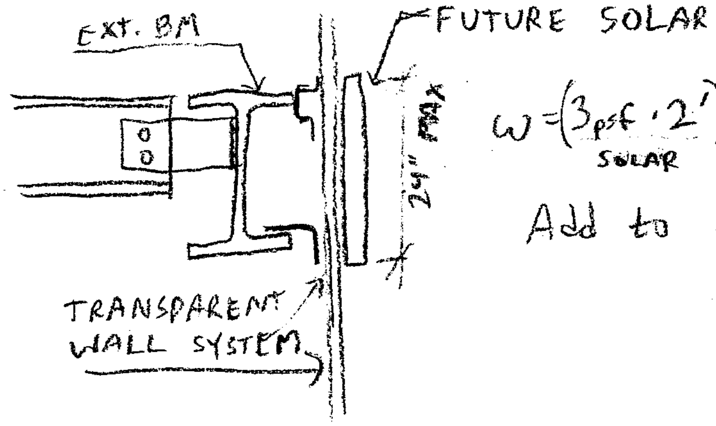
SPAN INCHES	LOAD TYPE									LOAD FOR	
										.25-inch deflection	.375-inch deflection
24	UL UL deflection	500 .025	750 .038	1000 .007	1250 .010	1500 .013	2000 .016	3000 .023	4000 .029		
	CL CL deflection	500 .021	750 .031	1000 .042	1250 .052	1500 .062	2000 .083	3000 .125	4000 .166		
30	UL UL deflection	250 .032	500 .065	750 .097	1000 .129	1500 .194	2000 .258	2500 .323	3000 .387		
	CL CL deflection	250 .021	500 .042	750 .063	1000 .083	1500 .125	2000 .167	2500 .208	3000 .250		
36	UL UL deflection	100 .025	250 .063	500 .125	750 .188	1000 .250	1250 .313	1500 .375	2000 .500		
	CL CL deflection	100 .014	250 .035	500 .069	750 .104	1000 .139	1250 .174	1500 .208	2000 .278		
42	UL UL deflection	100 .045	125 .057	250 .114	500 .227	750 .341	1000 .454	1250 .568		550	825
	CL CL deflection	100 .021	125 .027	250 .054	500 .107	750 .161	1000 .214	1250 .268		116	1750
48	UL UL deflection	100 .080	250 .200	300 .240	400 .320	500 .400	600 .480			313	469
	CL CL deflection	100 .032	250 .080	300 .095	400 .127	500 .159	600 .239			786	1179
54	UL UL deflection	100 .120	125 .150	250 .300	300 .360	400 .480				208	313
	CL CL deflection	100 .045	250 .112	300 .134	400 .178	500 .223				561	841
60	UL UL deflection	75 .146	100 .195	200 .389	250 .486					129	193
	CL CL deflection	100 .062	250 .156	500 .312	750 .469					400	600
66	UL UL deflection	75 .200	100 .267	125 .333	250 .400					94	141
	CL CL deflection	100 .080	200 .160	300 .240	400 .320					313	469
72	UL UL deflection	75 .281	100 .375	125 .469						67	100
	CL CL deflection	100 .103	250 .256	400 .410						244	366
78	UL UL deflection	50 .260	75 .390							48	72
	CL CL deflection	100 .129	250 .322							194	292

Load Takeoffs

EXTERIOR BAY LOAD TAKEOFF		
Area-Applied Loads(psf)	fiberglass decking	Application Level
Fiberglass Decking	3	Decking
Fireproofing	1	Decking
Corrosion Resistant	1	Decking
MEP	5	Decking
Misc	3	Decking
Beams	4.2	Beams
Cross Braces	1.4	Beams
Girders	4.7	Girders
Columns	4.7	Columns
TOTALS (psf)		
Deck	13	
Beams	19.6	
Girders	24.2	
Columns	28	
Linearly-Applied Loads(plf)		
PVC Pipe System	1	Ext. Girders
Spinach Biomass	1.2	Ext. Girders
Movable Rack System	60	Ext. Girders
Rack Support	25	Ext. Girders
Glass/Plastic Curtain Walls	21	Ext. Beams
Live Load (psf)	80	

INTERIOR BAY LOAD TAKEOFF		
Area-Applied Loads(psf)	metal decking	Application Level
Metal Decking	22	Decking
Fireproofing	1	Decking
Corrosion Resistant	1	Decking
MEP	8	Decking
Misc	3	Decking
Beams	3	Beams
Cross Braces	1.5	Beams
Girders	4.5	Girders
Columns	2	Columns
TOTALS (psf)		
Deck	35	
Beams	39.5	
Girders	44	
Columns	46	
Live Load (psf)	135	

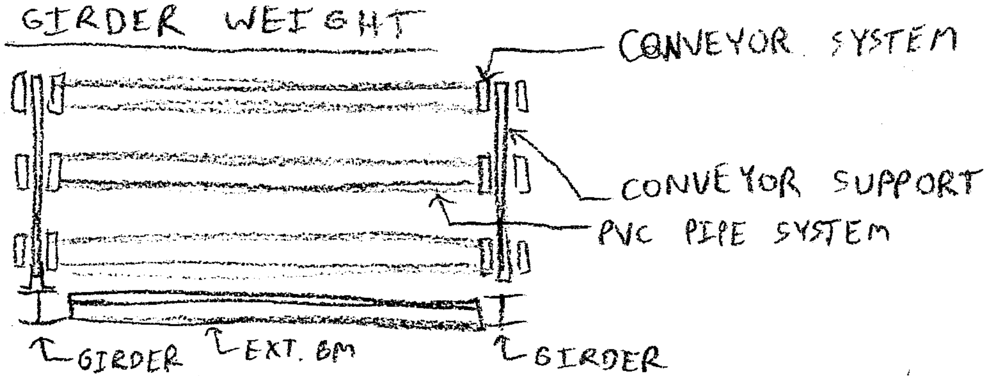
WALL WEIGHT



$$w = \underset{SOLAR}{(3\,psf \cdot 2')} + \underset{POLYETHELYNE}{(12' \cdot 59\,pcf \cdot \tfrac{1'}{12"} \cdot \tfrac{1}{4}")} = 21\,plf$$

Add to ext. bm. load as DL

GIRDER WEIGHT



wt. of pipes: $\left[\underset{PVC}{0.5} + \underset{WATER}{\left(\pi\left(\tfrac{1}{2}\right)^2 \cdot \tfrac{1}{144} \cdot 62.4\right)} + \underset{BIOMASS}{1.2}\right] \times \underset{layers}{6} = 12.24 \rightarrow \underline{13\,plf}$

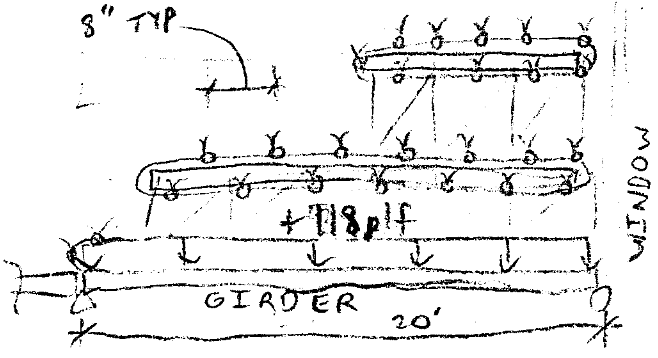
conveyor wt: 3 layers × 10 plf × 2 columns/girder = $\underline{60\,plf}$

+ Support wt: 25 plf

---

98 plf added to girder as Dead Load
1.2 × 74 = 88.8 → Add $\underline{118\,plf}$ factored DL

TYPICAL LAYOUT/



CORE BAYS

Water storage live load:

- Assume water is stored in tanks no larger than 2' in diameter and no more than 6' in height.

- Internal Volume $= \pi \left(\frac{2}{1}\right)^2 \cdot 6 = 18.85\ ft.^3$
- Water weight $= 62.4\,pcf \cdot 18.85\ ft.^3 = \boxed{1176\ lb.}$

  Distribute to 2' × 2' area: $\boxed{W = 294\ psf} \rightarrow$ Check locally

- Assume water tanks and other heavy objects cover a maximum of 25% of the total area inside the core bays

AVERAGE LIVE LOAD

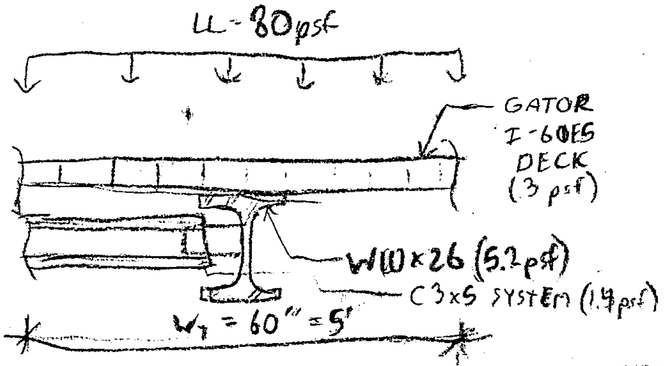
$(80\,psf \times 0.75) + (294\,psf \times 0.25) = 134\,psf$

Use 135 psf to design for beams, girders and columns.

FOR SEISMIC WEIGHT, INCLUDE 25% OF WATER WEIGHT IN DEAD WEIGHT*

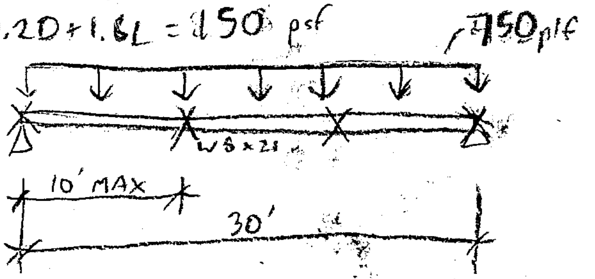
$294 \times 0.25 \times 0.25 = \boxed{18.4\ psf}$

* Tanks will almost always have some water, and be contained to one location due to anchorage to the floor. Similar live load conditions in the code also specify provisions like this for heavy load cases such as library stacks.



DL = 19.6psf TOTAL

1.2D + 1.6L = 150 psf



$L_b = 10'$

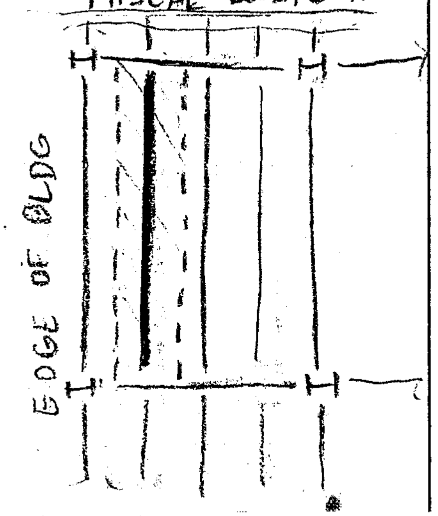
$$M_u = \frac{wL^2}{8} = \frac{.750 \cdot 30^2}{8} = \boxed{84.4 \text{ k-ft.}}$$

CAPACITY OF W10x26 @ 10' UNBRACED LENGTH = $\boxed{94\text{k-ft}}$ ✓
(REF. AISC TABLE 3-10)

REACTIONS (SYMMETRIC)

$$R = \frac{\text{TOTAL LOAD}}{2} = 750\,\text{plf} \cdot 30' \cdot \tfrac{1}{2} = \boxed{11.25\text{k}} \rightarrow$$ GRAVITY LOAD ON GIRDERS/COLUMNS

TYPICAL LOCATION



TYP. EXT. BM

G3

$A_T(\text{EXT. BM}) = \frac{1}{2}\left[A_T(\text{INT. BM})\right] = \frac{1}{2}(5' \cdot 30) =$

$\therefore w_{ext} = \frac{1}{2} w_{int} = \frac{1}{2}(750) = 375 \text{ plf}$

$w_{TOTAL} = 375 + w_{wall}$

$w_{wall} = \underbrace{(6 \text{ plf})}_{\text{FUTURE SOLAR}} + \underbrace{(1.23 \text{ psf} \cdot 12')}_{\text{WALL WT.}} = 21 \text{ plf} \rightarrow 25 \text{ plf factored}$

$w_{TOTAL} = 375 + 25 = 400 \text{ plf}$

$M_u = \frac{w\ell^2}{8} = \frac{.400 \cdot 30^2}{8} = 45 \text{ k-ft.}$ $L_b = 10'$

FOR BENDING, SELECT W8×21 CAPACITY = 61 k-ft. @ $L_b = 10'$

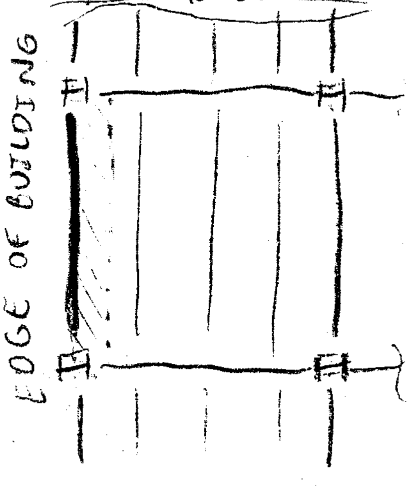
([illegible] 5 CAPACITY ≈ 30 k-ft. [illegible])

(VALUES OBTAINED FROM AISC TABLE 3-10)

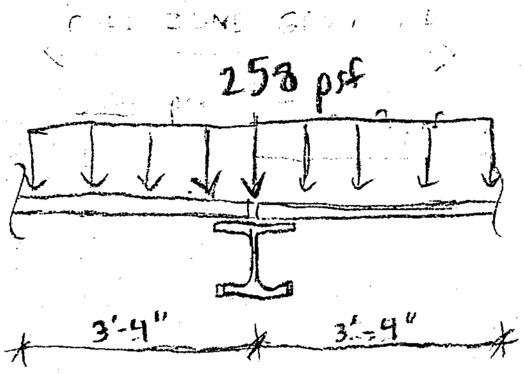
REACTIONS

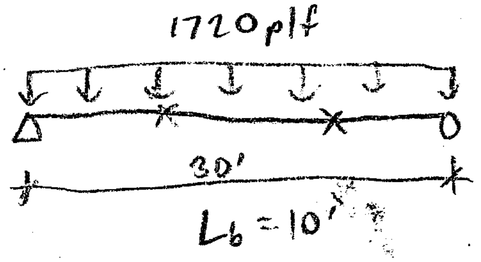
$R = 400 \text{ plf} \cdot 30' \cdot \frac{1}{1000} \cdot \frac{1}{2} = \boxed{6.0 \text{ k}}$ → GRAVITY LOAD ON GIRDERS/COLUMNS

TYPICAL LOCATION



TYP CORE BEAM





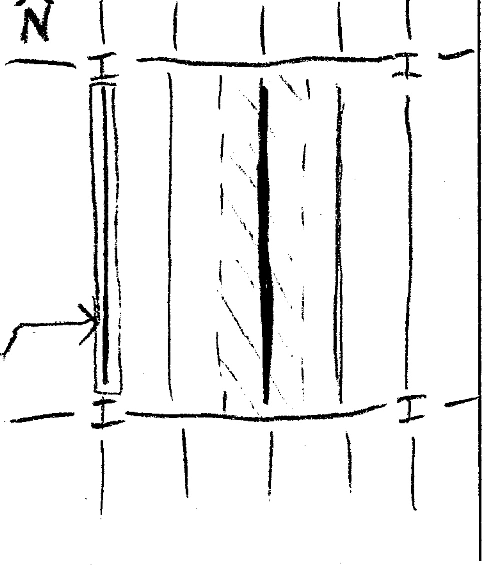
$$M_u = \frac{wl^2}{8} = \frac{1720 \cdot 30^2}{8} \cdot \frac{1}{1000} = 193.5 \text{ k-ft.}$$

SELECT W16x40 FOR CORE BEAMS
CAPACITY = 228 k-ft. PER AISC TABLE 3-10

SHEAR FORCE TO GIRDERS

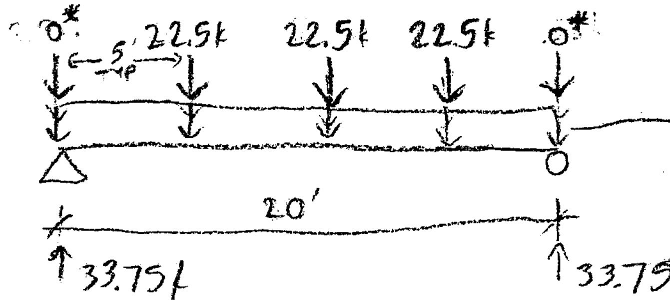
$$R = 1720 \cdot 30 \cdot \tfrac{1}{2} \cdot \tfrac{1}{1000} = \underline{25.8\text{k}}$$

TYPICAL LOCATION



HYBRID LOADING: TREAT AS CORE BEAM TO BE CONSERVATIVE

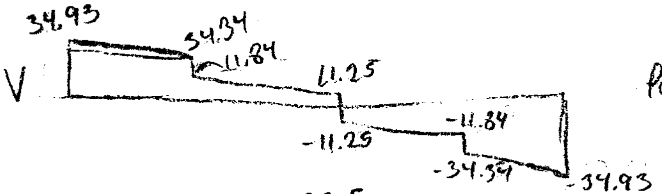
TYP EXT GIRDER



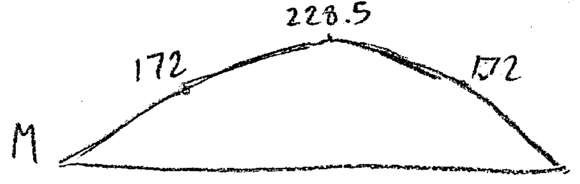
* beam load goes directly to columns

118 plf added dead load

$L_b = 5'\text{-}0''$



Peak $V_u = 34.9$ k



Peak $M_u = 228.5$ k-ft.

LIVE LOAD REDUCTION

$K_{LL} = 2 \quad A_T = 600\text{ SF} \quad L = L_o\left(.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right) = 54.6\text{ psf}$

$$\frac{1.2(23.6) + 1.6(54.6) = 115.68}{1.2(23.6) + 1.6(80) = 156.32} = .740 \times M_u = 169\text{ k-ft.}$$

DESIGN MOMENT $M_u = 169$ k-ft.

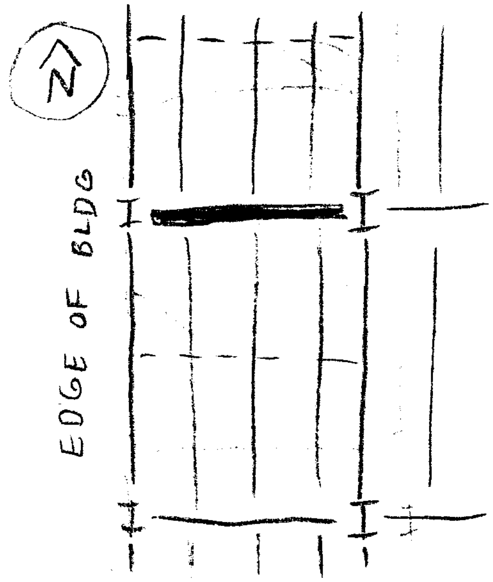
SELECT W12×35 FOR GIRDERS IN EXTERIOR BAYS TO BALANCE LIGHT WEIGHT WITH SECTION DEPTH
CAPACITY @ $L_b = 5'$ = 192 k-ft. PER AISC TABLE 3-10

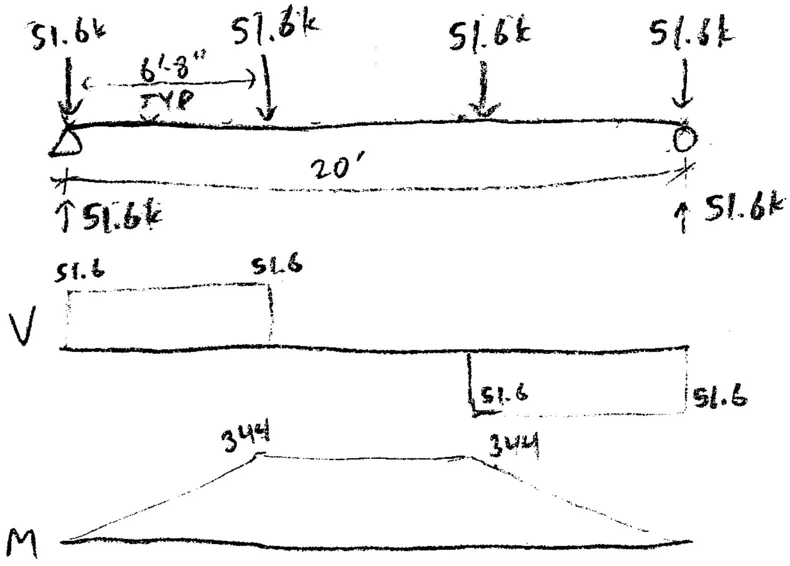
ALTERNATE SECTION SIZES

SECTION	CAPACITY
W14×30	177 k-ft **
W10×39	175 k-ft **
W10×45	206 k-ft
W16×31	195 k-ft

** During a harvest time, the building may be fully occupied, which may exceed member capacity on reduced live load. Members at >90% capacity are not 1st choice.

TYPICAL LOCATION





$L_b = 6.67'$

$V_{max} = 51.6$ k

$M_{max} = 344$ k-ft.

$\phi M_n \geq$

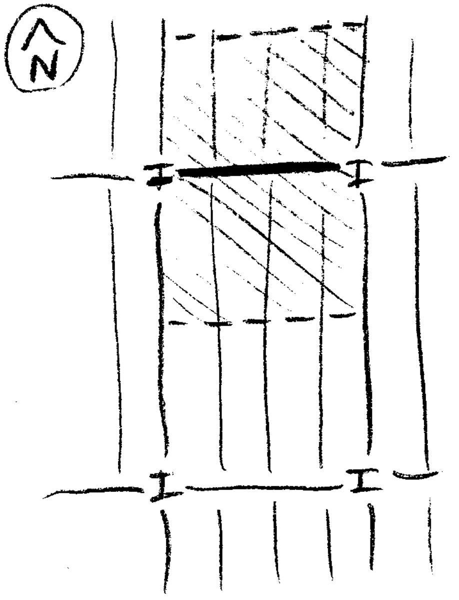
SELECT W21x48, CAPACITY = 390 k-ft.

— Add Girder self-weight, 48 plf

$M = \frac{wl^2}{8} = \frac{48 \cdot 20^2}{8} = 2.4$ k-ft.

Total $M_u = 344 + 2.4 = \underline{346.4\text{k-ft.}} < \underline{390\text{k-ft.}}$ ✓

TYPICAL LOCATION



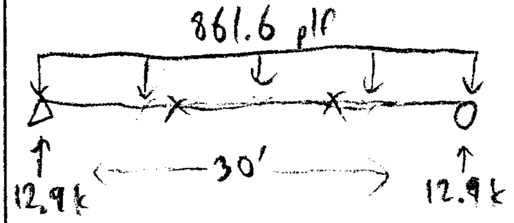
TYP. ROOF BEAM

Spread water loads out over entire areas.

$\left[(135 \times 4_{\text{bays}}) + (80 \times 13_{\text{bays}})\right] \cdot \frac{1}{17} = 93_{\text{psf}}$ AVG. LIVE LOAD

$1.2D + 1.6L = 1.2(19.6) + 1.6(93) = 172.3_{\text{psf}}$

$w = 172.3 \cdot 5 = 861.6_{\text{plf}}$



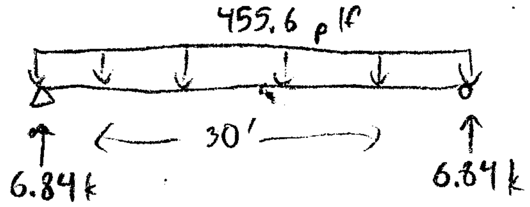
$L_b = 10'$

$$M_u = \frac{w l^2}{8} = \frac{.8616 \cdot 30^2}{8} = 96.9 \text{ k-ft.}$$

$\boxed{\text{SELECT W12×26, CAPACITY = 114 k-ft. @ } L_b = 10'}$

EXT. ROOF BEAM

$w = (172.3 \cdot 2.5) + 25 = 455.8_{\text{plf}}$



$L_b = 10'$

$$M_u = \frac{w l^2}{8} = \frac{.4556 \cdot 30^2}{8} = 51.3 \text{ k-ft.}$$

$\boxed{\text{SELECT W10×28, CAPACITY = 94 k-ft. @ } L_b = 10'}$

TYP. ROOF GIRDER

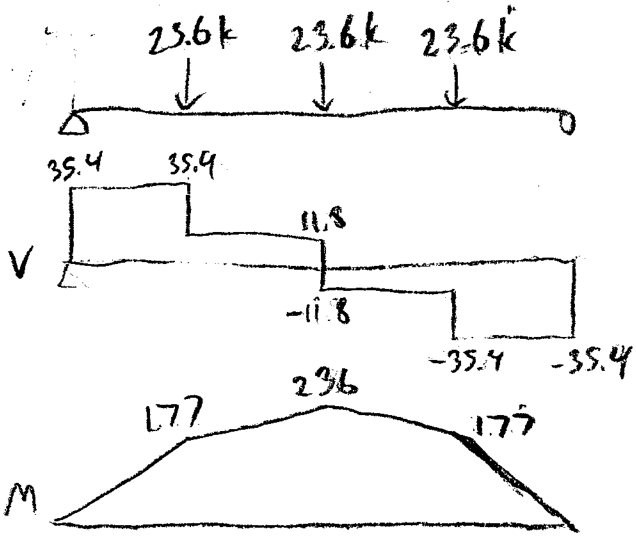
$1.2D+1.6L = 1.2(24.2)+1.6(93) = 177.8$ psf

LIVE LOAD REDUCTION

$L = 93\left(.25+\frac{15}{\sqrt{600}}\right) = 80.2$ psf

$1.2D+1.6L = 157.4$ psf

$w = 157.4 \cdot 5 \cdot 15 = 11.81$ k



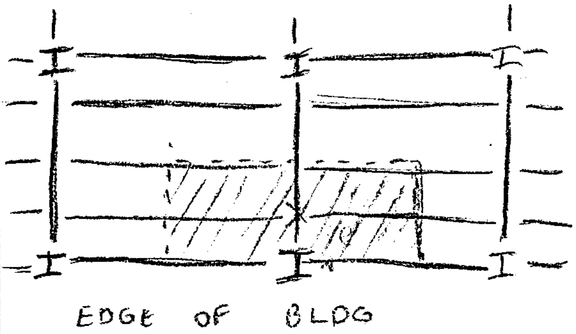
$M_u = 236$ k-ft.

$\boxed{\text{SELECT W16×40, CAPACITY} = 274 \text{ k-ft.}}$

SEE P. G5 FOR DETAILED EXPLANATION OF CHOICE

TYP. EXT. COLUMN

(L [illegible])



LIVE LOAD REDUCTION

$L = L_o\left(.25 + \frac{15}{\sqrt{A_T K_{LL}}}\right) = 54.6$ psf

$K_{LL} = 4$ $A_T = 300$ SF

$P_o = \left[(1.2 \cdot \underline{31}) + (1.6 \cdot 54.6)\right] \cdot 10' \cdot 30' \cdot \frac{1}{1000} = \underline{37.9}$ k / floor

$(P_o \cdot 4)_{(\text{Normal Floors})} + (2 \cdot P_o)_{(\text{top floor})} = 6P_o = \underline{225}$ k

AXIAL LOADING

k = 1.0 L_b = 11'-0" MAX

SELECT HSS 6×6×½

CAPACITY = $\underline{319}$ k (AISC Table 4-4)

ABOVE COLUMN SPLICE

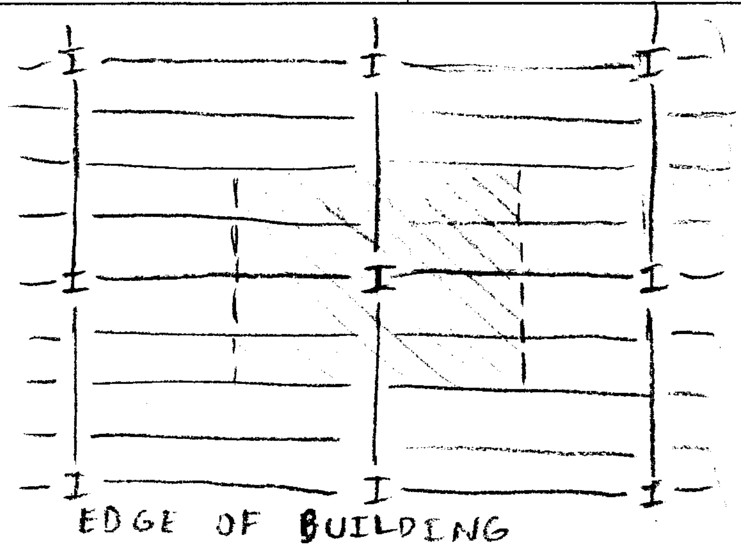
Total load $= P_{TOTAL} - (2 \cdot P_o) = 4P_o = \underline{150}$ k

k = 1.0 L_b = 11'-0" MAX.

SELECT HSS 6×6×¼

CAPACITY = $\underline{175}$ k (AISC Table 4-4)

TYP. CORE COLUMN



LIVE LOAD REDUCTION

$K_{LL} = 4$ $A_T = 600$ sf

$L = L_o(.25 + \frac{15}{\sqrt{K_{LL} A_T}})$

$L = 117$ psf

$P_o = [(1.2 \cdot 40 + (1.6 \cdot 117)] \cdot 20' \cdot 30' \cdot \frac{1}{1000} = 141.12$ k/floor

$(P_o \cdot 4) + 2P_o = 6P_o = 847$ k

AXIAL LOADING

k = 1.0 L_b = 11'-0" MAX.

SELECT W10×88 COLUMNS

CAPACITY = 973k (REF. AISC TABLE 4-1)

ABOVE COLUMN SPLICE

Total Load = $P - (2P_o) = 4P_o = 565$k

k = 1.0 L_b = 11'-0" MAX.

SELECT W10×54 COLUMNS

CAPACITY = 585k (REF AISC TABLE 4-1)

CHECK L_b = 15' AT ROOF

$P = [1.2 \cdot 3 + 1.6(20)] \cdot 20' \cdot 30' \cdot \frac{1}{1000} = 21.4$k

W10×54 CAPACITY @ L_b = 15': 495k ✓

CANOPY CEILING

Assume no live load. Polyethelyne panels cannot support human weight and would be serviced by removing panels from the floor below.

$D = 3\text{psf} \rightarrow 1.4D = 4.2\text{ psf}$

BEAMS

Try $\boxed{WT\ 4 \times 6.5}$. $Z = 1.74\text{ in.}^3$ Assume A36 steel

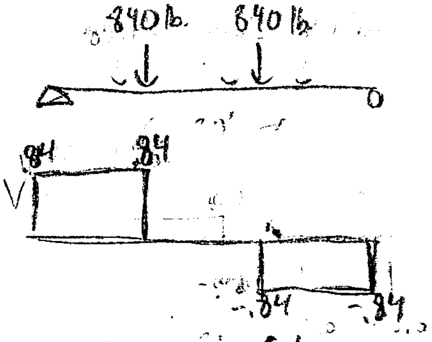
$M_n = 36\text{ ksi} \cdot 1.74 = 62.64\text{ k-in.} = 5.22\text{ k-ft.}$

$\phi M_n = 4.70\text{ k-ft.}$

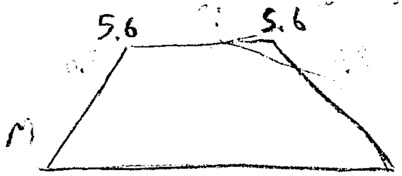
$w_u = 4.2 \cdot 6.67 = 28\text{ plf}$

$M_u = \frac{wl^2}{8} = \frac{0.028 \cdot 30^2}{8} = 3.15\text{ k-ft.} < 4.70\text{k}$ ✓

GIRDERS



$V_u = 0.84\text{ k}$



$M_u = 5.6\text{ k-ft.}$

Try $\boxed{(2)\ L\ 4 \times 3 \times 5/16}$ $Z = 1.28 \cdot 2 = 2.56\text{ in.}^3$

$M_n = 36 \cdot 2.56 = 92.2\text{ k-in.} = 7.68\text{ k-ft.}$

$\phi M_n = 6.91\text{ k-ft.}$

$M_{u_{TOTAL}} = 5.6 + \underbrace{\left(\frac{0.007 \cdot 20^2}{8}\right)}_{\text{ADDED SELF WT.}} = 5.95\text{ k-ft.} < 6.91\text{ k-ft.}$ ✓

SEISMIC LOADS

$S_{ds} = 0.791\,g \qquad S_{d1} = 0.450\,g$

BLDG. TOTAL SEISMIC WEIGHT

$$W_{FLOORS} = 4\left\{\left[\underset{\text{GROW ZONES}}{\underline{28}\ psf\,(30\cdot 20)\cdot 13}\right] + \left[\underset{\text{CORE ZONES}}{\underline{58.4}\ psf\,(30\cdot 20)\cdot 4}\right]\right\} = \underline{1434\,k}$$

$$W_{ROOF} = 2\,W_{FLOOR} = \underline{717.12}$$

$$W_{CANOPY} = (180\cdot 60)\cdot \underline{3}\ psf = 32.4\,k$$

$$+\ W_{WALL} = \underline{1.25}\ psf\left[(2\cdot 180) + 60 + (3\cdot 20) + (2\cdot 30)\right]\cdot 75' = \underline{50.63\,k}$$

$$W_{TOTAL} = \underline{2234}\ k$$

BASE SHEAR

$$C_s = \frac{S_{ds}}{R/I} = \frac{0.791}{6/1.0} = \boxed{0.1318}$$

SEISMIC DESIGN CATEGORY E

R = 6 (ORDINARY STEEL CONCENTRICALLY BRACED FRAME)

$$V_b = C_s \cdot W = 0.1318 \cdot 2234 = \underline{295}\ k$$

SHEAR DISTRIBUTION BY LEVEL

$$V_i = \frac{W_i h_i^k}{\Sigma W_i h_i^k} V_b \qquad k = 1.0$$

$$V_2 = \frac{12' \cdot 371.2 \cdot V_b}{(12\cdot \underline{371.2}) + (24\cdot \underline{371.2}) + (36\cdot \underline{371.2}) + (48\cdot \underline{371.2}) + (60\cdot \underline{728.1}) + (75\cdot \underline{37.5})} = 14.45k$$

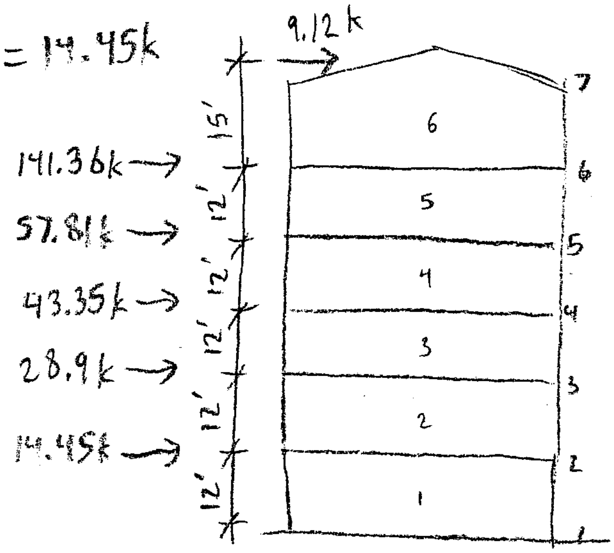
$V_3 = 28.9\,k$

$V_4 = 43.35\,k$

$V_5 = 57.81\,k$

$V_6 = 141.36\,k$

$V_7 = 9.12\,k$



WIND LOADS

BUILDING FACE AREA

E/W: $180 \times 69 = 12{,}420$ sf

N/S: $60 \times 69 = 4140$ sf

$h_{eff} = 75 - \frac{h_1}{2} = 75 - 6 = 69'$

WIND PRESSURE PER ASCE 7-10

EXPOSURE C

BASE WIND SPEED: 110 MPH

$K_d = 0.85$

$GC_{pi} = \pm 0.18$

$K_{zt} = 1.0$ due to lack of significant elevation changes near the site

$$K_z = 2.01\left(\frac{z}{z_g}\right)^{2/\alpha} = 2.01\left(\frac{75}{900}\right)^{2/9.5} = 1.191$$

$$q_z = 0.00256\, K_z K_{zt} K_D V^2$$

$$= 0.00256 \cdot 1.191 \cdot 1.0 \cdot 0.85 \cdot 110^2 = 31.36 \text{ psf}$$

$$p = q\,GC_p - q_i(GC_{pi})$$

$G = 0.85$

$C_p = 0.8$ (windward), -0.5 (leeward)

$$p_w = 31.36(0.8)(0.85) - 31.36(\pm .18) = \underline{27.0 \text{ psf}}$$

$$p_l = 31.36(-0.5)(0.85) - 31.36(\pm .18) = \underline{19.0 \text{ psf}}$$

BASE SHEAR

$$E/W:\ V_b = 12{,}420(|p_w| + |p_l|) \cdot \frac{1}{1000} = \boxed{571.3 \text{ k}}$$

$$N/S:\ V_b = 4140(|p_w| + |p_l|) \cdot \frac{1}{1000} = \boxed{190.4 \text{ k}}$$

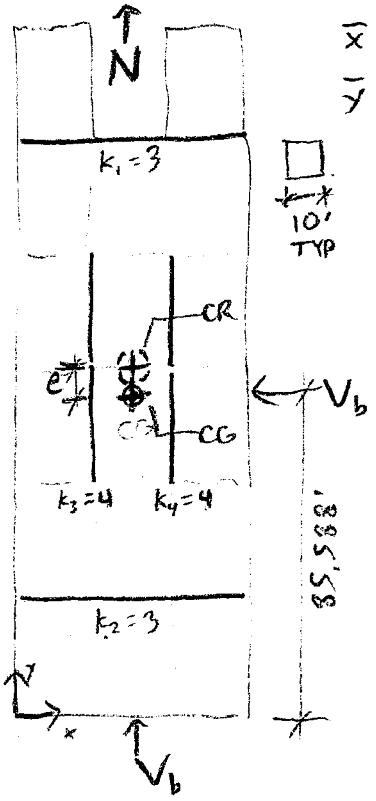
CONCLUSION:

WIND GOVERNS IN E/W DIRECTION,* BUT SEISMIC GOVERNS IN THE N/S DIRECTION

*EXCEPTION: WHEN BASE SHEAR IS DISTRIBUTED BY LEVEL, SEISMIC GOVERNS AT THE TOP FLOOR

$$V_w = 180 \cdot 13.5(27 + 16) \cdot \frac{1}{1000} = 111.8 \text{ k}$$

$$V_s = 141.4 \text{ k}$$



$\bar{x} = 30'$

$\bar{y} = 85.588' = 85'\text{-}7''$

$e_x - 0 + 5\%(60') = 3'$

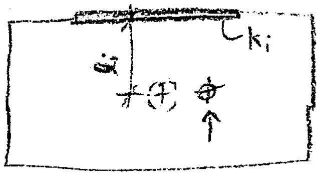
$e_y = 4.412 + 5\%(180') = 13.412'$

N/S) $M_x = V_b e_x = 295 \cdot 3 = 885$ k-ft.

E/W) $M_y = V_b e_y = 571.3 \cdot 13.412 = 7662.3$ k-ft.

TORSIONAL FORCE DISTRIBUTION (N/S)

$$F_i = \frac{M k_i l_i}{\Sigma k_i l_i^2}$$

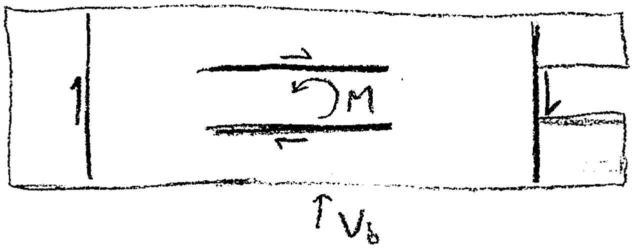


X-DIRECTION

$$F_1 = \frac{M_x \cdot 3 \cdot 94.412}{(3 \cdot 94.412^2) + (3 \cdot 85.588^2) + (4 \cdot 10^2) \cdot 2} = \frac{250,663.9}{49,116.8} = 5.10 \text{ k}$$

$$F_2 = \frac{M_x \cdot 3 \cdot 85.588}{49,116.8} = 4.63 \text{ k}$$

$$F_3 = F_4 = \frac{M_x \cdot 4 \cdot 10}{49,116.8} = 0.72 \text{ k}$$



E/W DIRECTION

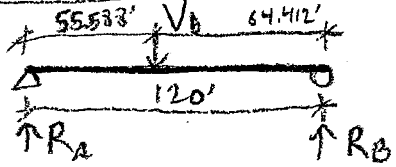
$$F_1 = \frac{M_y \cdot 3 \cdot 94.412}{49,116.8} = 44.19\,k$$

$$F_2 = \frac{M_y \cdot 3 \cdot 85.588}{49,116.8} = 40.06\,k$$

$$F_3 = F_4 = \frac{M_y \cdot 4 \cdot 10}{49,116.8} = 6.24\,k$$

SHEAR FORCE DISTRIBUTION

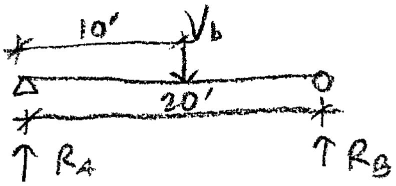
E/W DIRECTION:



$$R_A = \frac{64.412}{120} V_b = 306.7\,k$$

$$R_B = \frac{55.588}{120} V_b = 264.6\,k$$

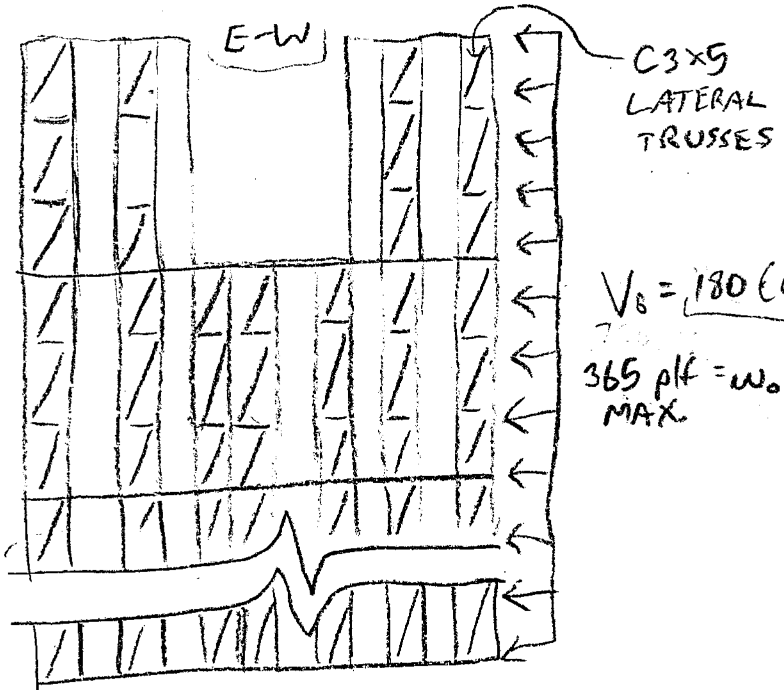
N/S DIRECTION



$$R_A = R_B = \frac{V_b}{2} = 147.5\,k$$

SUMMARY

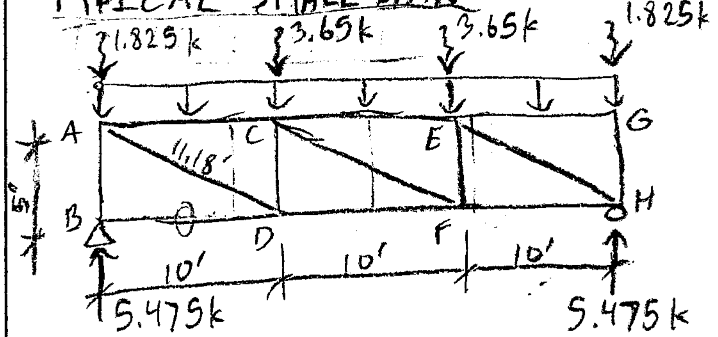
	MAX. V_{DIRECT}	MAX. $V_{TORS.}$	TOTAL
N-S	147.5 k	5.10 k	152.6 k
E-W	306.7 k	44.19 k	350.9 k



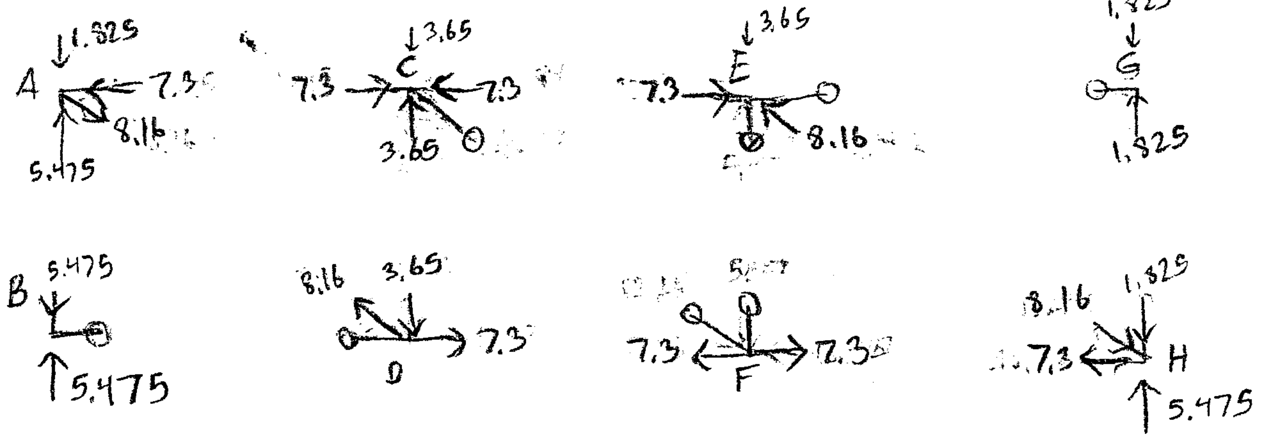
$$V_0 = \underbrace{180(6+7.5)}_{\text{Trib. Area}}\ \overbrace{(27\,psf)}^{\text{Windward face pressure}} = 65.61\ k$$

365 plf $= w_o$
MAX

TYPICAL SMALL SPAN



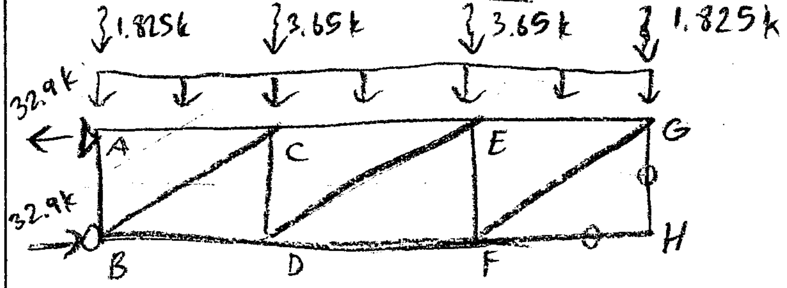
- Check weak-axis bending in beam-chords
- For truss, model as point loads to A, C, E, G

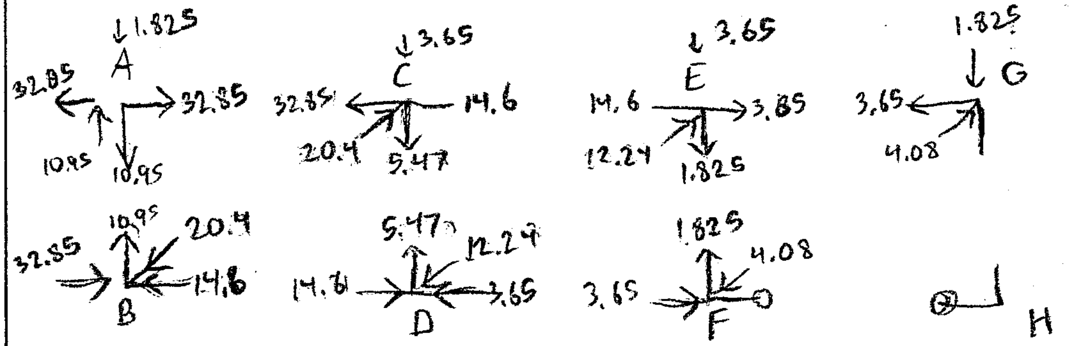


Highest comp. force in C3×5: <u>8.16 k</u>

Highest comp. force in W8×21: <u>7.3k</u>

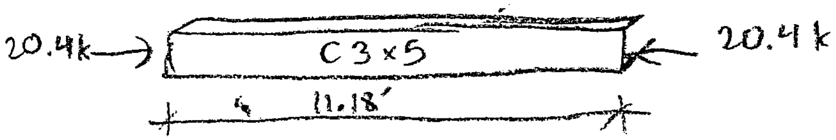
END & BAY: SMALL SPAN

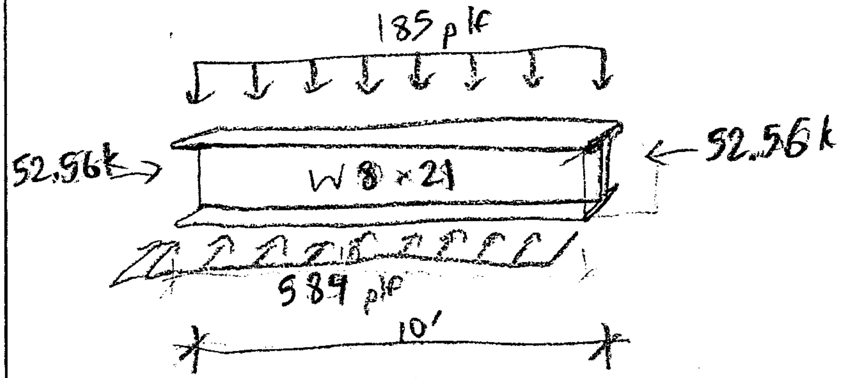




Highest comp. force in C3x5: 20.4 k

Highest comp. force in W8x21: 32.85 k

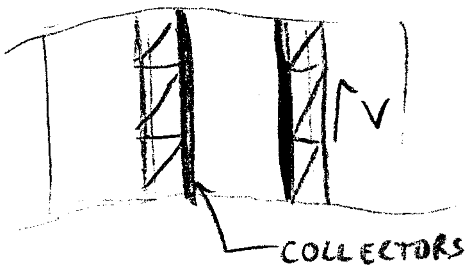




LOAD COMBINATION

1.2D + 0.5L + 1.6W

N/S

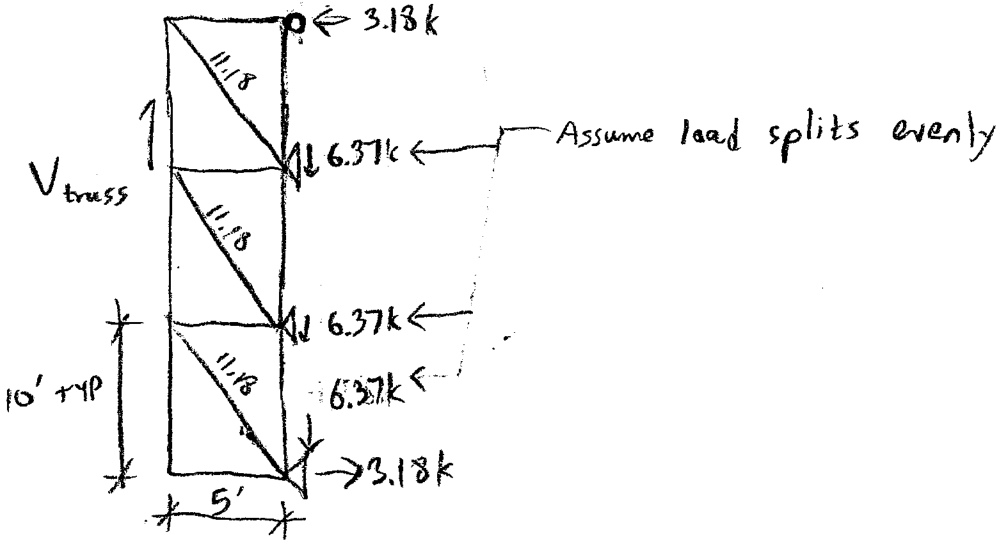


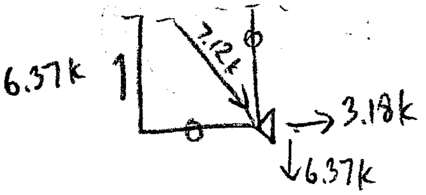
$V_{floor} = 141.4\ k$

$V = C_s \cdot W_{floor} \cdot \frac{141.4}{295}$

$V = \left[0.1318 \cdot \left(28 \cdot 30 \cdot 20 \cdot 6_{bays}\right)\right] \cdot 6\ \text{floors} \cdot \frac{141.4}{295} = \boxed{32.21\ k}$

DISTRIBUTE TO 2 SETS OF TRUSSES: $\boxed{V_{Truss} = 19.11\ k}$





$P_{max} = 7.12 < 20.4$

OTHER BRACES GOVERN DESIGN, USE RESULTS FROM PAGE L6 CALCULATION

C3x5 CAPACITY

$L = 11.18' = 134.2''$

$k = 1.0$

$r = .405''$

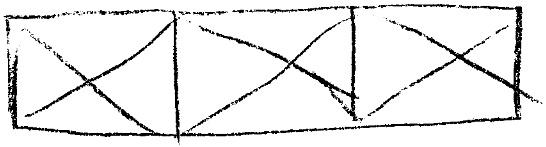
L4x4x7/16 → 12.8 plf

HSS 3x3x3/16 → 6.87 plf

Bars → 5.5 plf (4.8 plf if 7/8" Ø)

HSS 4x2½x3/16 → 6.87 plf

Use 1" Ø Bars in each direction (2 per bay)



$T_n = ((1)/2)^2 \cdot \pi \cdot 36 \text{ ksi} = 28.3 \text{ k}$

$\boxed{\phi T_n = 25.4 \text{ k} > 20.4 \text{ k}}$ ✓

Weight = 5.5 plf ~ 5 plf (channels) ✓

- No need to recalculate weights

	DEMAND	CAPACITY
STRONG-AXIS BENDING	20.8 k-ft.	61 k-ft.
WEAK-AXIS BENDING	7.3 k-ft.	21.3 k-ft.
AXIAL COMPRESSION	52.6k	141.5 k

Strong-Axis $M_u = \frac{w\ell^2}{8} = \frac{0.185 \cdot 30^2}{8} = 20.8$ k-ft.

Weak-Axis $M_u = \frac{w\ell^2}{8} = \frac{.584 \cdot 10^2}{8} = 7.3$ k-ft.

COMPRESSION CAPACITY

$L_y = 10'$ $K = 1.0$ $r_y = 1.26$ in. $L_x = 30'$ $r_x = 3.49$ in.

$\frac{KL_x}{r_x} = \boxed{103}$ $\frac{KL_y}{r_y} = 95$

$4.71\sqrt{E/F_y} = 4.71\sqrt{29{,}000/50} = 113 > 103 \rightarrow F_{cr} = \left[0.658^{F_y/F_e}\right] F_y$

$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 \cdot 29{,}000}{103^2} = 26.9$

$F_{cr} = \left(0.658^{50/26.9}\right) \cdot 50 = 22.97$ ksi

$\phi P_n = 0.9\,(A \cdot F_{cr}) = 6.16 \cdot 22.97 = 141.5$ k

WEAK-AXIS BENDING CAPACITY

$L_b = 10'$ $Z_y = 5.69$ in.^{3}

$M_n = M_p = F_y Z_y = 5.69 \cdot 50 = 285$ k-in. $= 23.7$ k-ft. $\phi = 0.9$

$\phi M_n = 21.3$ k-ft. (REF. AISC TABLE 3-4)

COMBINED FORCES

$$\frac{P_u}{\phi P_n} + \left(\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}}\right)\frac{8}{9} \leq 1.0$$

$$\frac{52.6}{141.57} + \left(\frac{7.3}{21.3} + \frac{20.8}{61}\right)\frac{8}{9} = 0.979 < 1.0 \checkmark$$

W8×21 IS ADEQUATE

W10x26 CAPACITY

	DEMAND	CAPACITY
STRONG-AXIS BENDING	51.3 k-ft.	94
WEAK-AXIS BENDING	7.3 k-ft.	28.1
AXIAL COMPRESSION	52.6 k-ft.	188.9 k

COMPRESSION CAPACITY

$L_y = 10'$, $r_y = 1.33$ in. $L_x = 30'$, $r_x = 4.27$ in. $K = 1.0$

$$\frac{KL_x}{r_x} = \boxed{90.2} \qquad \frac{KL_y}{r_y} = 84.3$$

$$F_e = \frac{29{,}000 \cdot \pi^2}{90.2^2} = 35.2$$

$$F_{cr} = \left(0.658^{50/35.2}\right) \cdot 50 = 27.6 \text{ ksi}$$

$$\phi P_n = 0.9 \cdot 27.6 \cdot 7.61 = \underline{188.9 k}$$

WEAK-AXIS BENDING CAPACITY

$$Z_y = 7.5 \text{ in.}^3$$

$$M_n = M_p = 7.5 \cdot 50 = 375 \text{ k-in.} = 31.25 \text{ k-ft.}$$

$$L_p = 1.76\, r_x \sqrt{\frac{E}{F_y}} = 1.76 \cdot 4.27 \sqrt{\frac{29{,}000}{50}} = 181 \text{ in.} = 15.08' > 10' \checkmark$$

$$\boxed{\phi M_n = 28.1 \text{ k-ft.}}$$ (REF. AISC TABLE 3-4)

COMBINED FORCES

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}}\right) \leq 1.0$$

$$\frac{52.6}{188.9} + \frac{8}{9}\left(\frac{7.3}{28.1} + \frac{51.3}{94}\right) = \underline{0.991 < 1.0} \checkmark$$

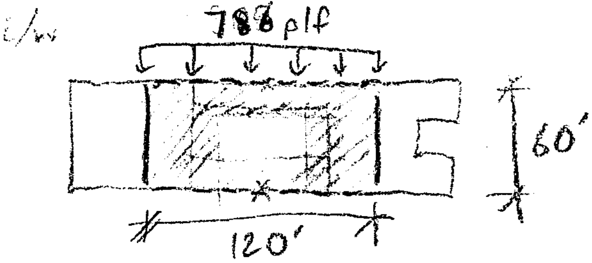
W10X26 IS ADEQUATE

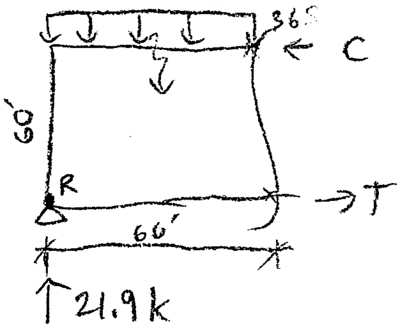
DIAPHRAGM CHORDS

E/W

[---] = CHORD

[▬] = COLLECTOR





$$\Sigma M_R = 60C - (786\cdot 60\cdot 30) = 0$$

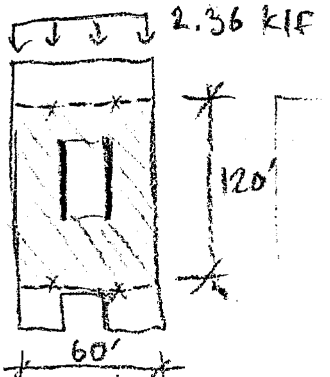
$T = 23.58\ k$

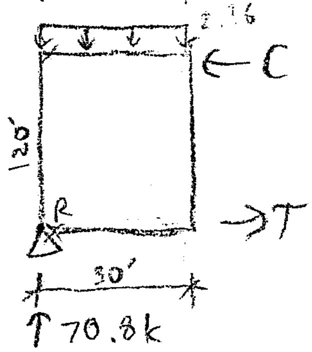
$C = 23.58\ k$

CHECK W8x21 COMPRESSION DEMAND @ CENTER OF

$$\underbrace{8.16}_{\text{REF. P. L4}} + 23.58 = 31.74k < \underbrace{32.85\ k}_{\text{REF. P. L5}} \text{ (GOVERNING)} \checkmark$$

N/S





$$\Sigma M_R = 120C - (2.36\cdot 30\cdot 15) = 0$$

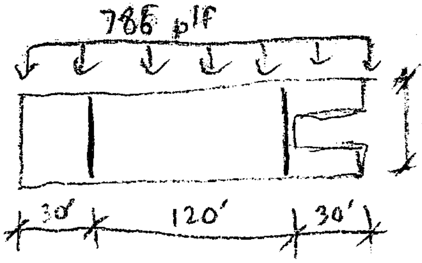
$C = 8.85k$

$T = 8.85k$

CHECK 8.85k ADDED ON W10x26 GIRDERS

↳ SEE NEXT PAGE

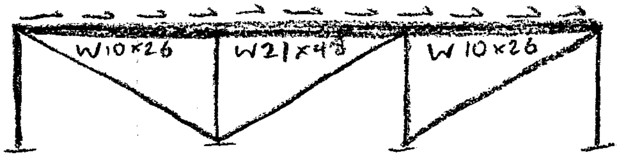
E/W

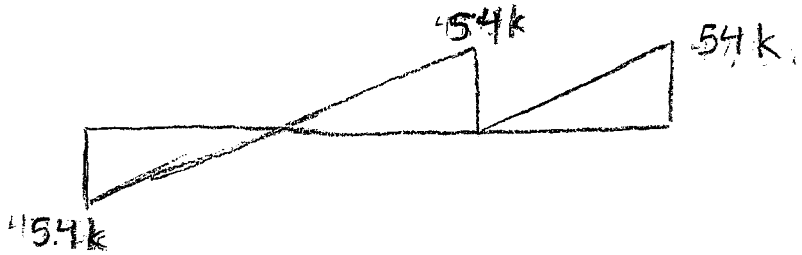


$V_{max} = 141.4 \left(\frac{350.9}{306.7}\right) = 161.8k$ *

* Scaled effect of torsion, Ref. P. L3

$\frac{161.8}{60} = 2.7$ klf





CHECK 54k ADDED ON W10×26 GIRDERS

$K = 1.0 \quad L = 20' \quad r_x = 4.35") \rightarrow \frac{KL}{r} = 55$

$F_e = \frac{\pi^2 \cdot 29{,}000}{55^2} = 94$

$F_{cr} = \left(0.658^{50/94}\right) \cdot 50 = 40$ ksi

$\phi P_n = 0.9 \cdot 40 \cdot 7.61 \text{ in.}^2 = \underline{274 k}$

COMBINED FORCES

$\frac{55}{274} + \left(\frac{88}{116}\right)\frac{8}{9} = 0.875 < 1.0$ ✓

W10×26 IS ADEQUATE AS A COLLECTOR

CHECK W21×48

$K = 1.0 \quad L = 20' \quad r_x = 8.24 \text{ in.} \quad \frac{KL}{r} = 29 \quad F_{cr} = 47$ ksi

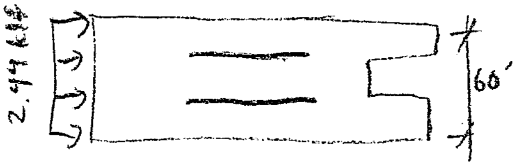
$\phi P_n = 0.9 \cdot 47 \cdot 14.1 = 596$ k

COMBINED FORCES

$\frac{55}{2 \cdot 596} + \frac{346.4}{390} = 0.934 < 1.0$ ✓

W21×48 IS ADEQUATE AS A COLLECTOR

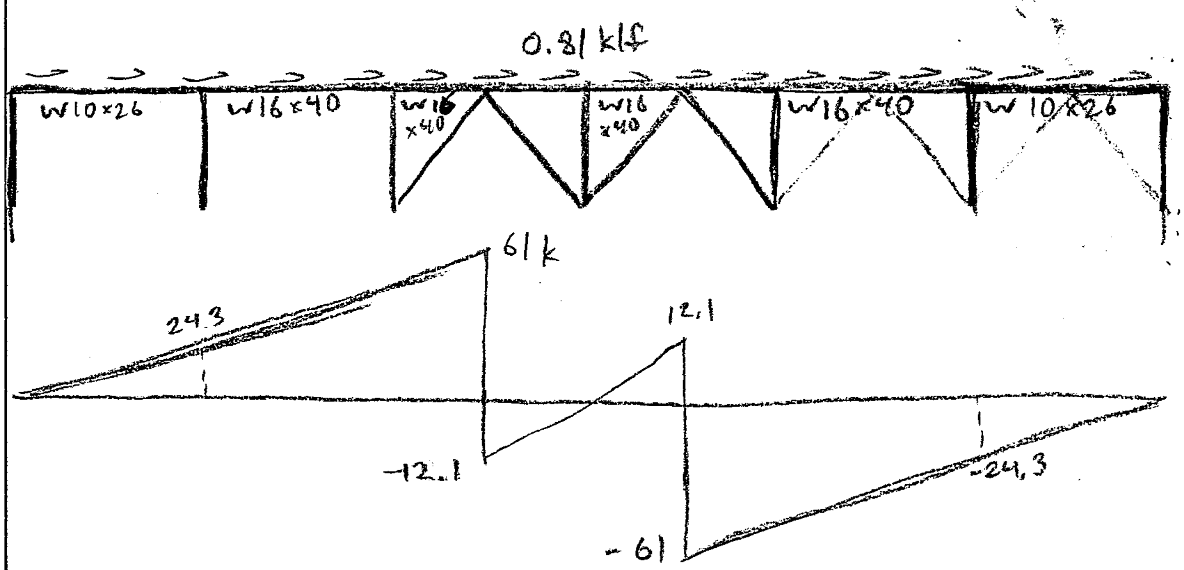
N/S



$$V_{max} = 141.4\left(\frac{152.6}{147.5}\right) = 146.3k$$ *

Scaled effect of torsion, Ref. P. L3

$$\frac{146.3}{180} = 0.81 \text{ klf}$$



CHECK 61K ADDED ON W16×40

$K = 1.0 \quad L = 30' \quad r_x = 6.96 \text{ in.} \rightarrow \frac{KL}{r} = 52$

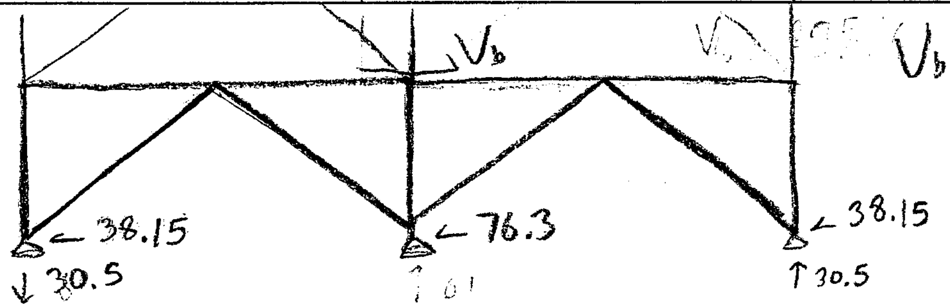
$F_e = \frac{\pi^2 \cdot 29{,}000}{52^2} = 107 \qquad F_{cr} = \left(0.658^{50/107}\right) \cdot 50 = 15.37 \text{ ksi}$

$\phi P_n = 0.9 \cdot 15.37 \cdot 11.8 = \underline{163\,k}$

COMBINED LOADING

$$\frac{61}{163} + \left(\frac{198.7}{228}\right)\frac{8}{9} = 0.95 < 1.0 \checkmark$$

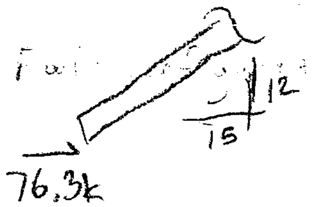
$\boxed{\text{W16×40 IS ADEQUATE AS A COLLECTOR}}$



$V_b = 152.6$ k

$L_{brace} = 19.2'$

Overturn: $F_y = V_b \cdot \frac{h}{L} = 152.6 \cdot \frac{12}{60} = \underline{30.5 \text{ k}}$



$\sqrt{15^2+12^2} = 19.2'$

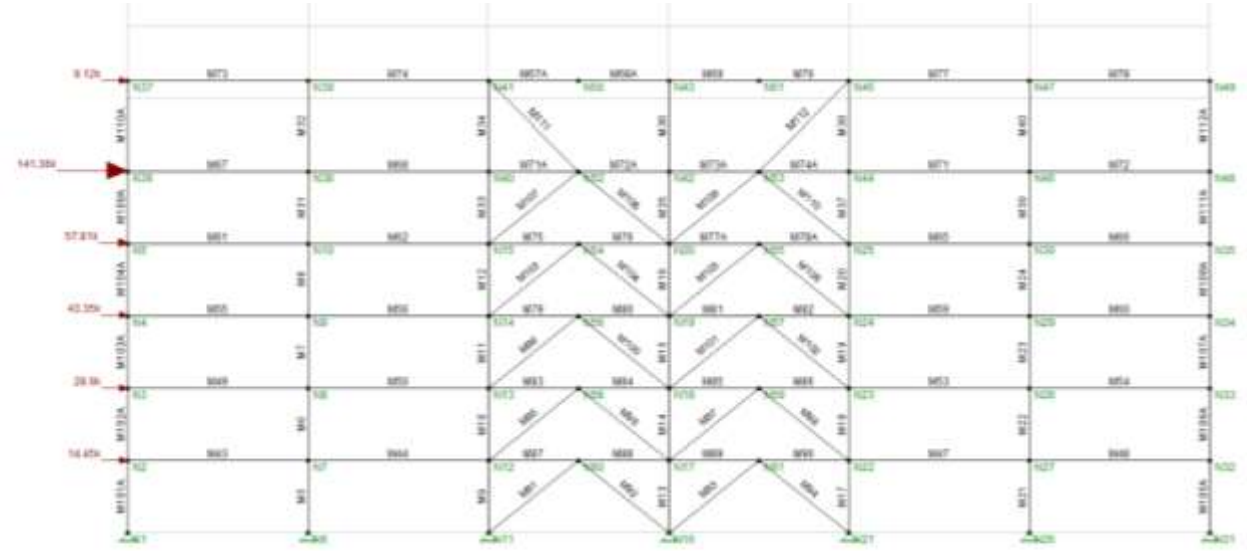
$F_{brace} = 38.15 \cdot \frac{19.2}{15} = \boxed{48.8 \text{k}}$

FOR PRELIMINARY DESIGN, SELECT HSS $4\frac{1}{2} \times 4\frac{1}{2} \times \frac{5}{16}$
CAPACITY = 53.0 k

SEE RISA 2D RESULTS ON NEXT PAGE FOR DETAILED STORY DRIFT ANALYSIS (P. L12)

-Story Drift is adequate

RISA 2D ANALYSIS NORTH-SOUTH



JOINT DEFLECTIONS

LC	Joint Label	X [in]	Y [in]
1	N1	0	0
1	N2	.285	-.078
1	N3	.55	-.144
1	N4	.806	-.196
1	N5	1.047	-.235
1	N6	0	0
1	N7	.269	-.094
1	N8	.52	-.169
1	N9	.761	-.226
1	N10	.986	-.263
1	N11	0	0
1	N12	.25	-.079
1	N13	.489	-.149
1	N14	.717	-.21
1	N15	.921	-.259
1	N16	0	0
1	N17	.239	-.116
1	N18	.47	-.213
1	N19	.685	-.291
1	N20	.872	-.35
1	N21	0	0
1	N22	.239	-.143
1	N23	.467	-.257
1	N24	.675	-.344
1	N25	.847	-.404

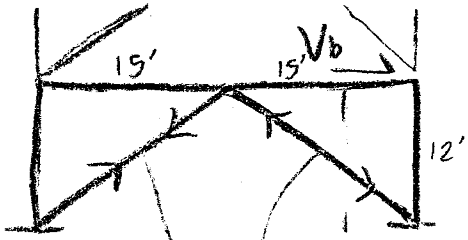
1	N26	0	0
1	N27	.242	-.095
1	N28	.466	-.171
1	N29	.675	-.228
1	N30	.845	-.266
1	N31	0	0
1	N32	.243	-.081
1	N33	.466	-.149
1	N34	.675	-.203
1	N35	.845	-.244
1	N36	1.598	-.285
1	N37	1.208	-.285
1	N38	1.371	-.294
1	N39	1.147	-.294
1	N40	1.155	-.317
1	N41	1.077	-.313
1	N42	1.004	-.412
1	N43	1.071	-.412
1	N44	.957	-.466
1	N45	1.065	-.466
1	N46	.955	-.297
1	N47	1.064	-.297
1	N48	.955	-.293
1	N49	1.063	-.293
1	N50	1.074	-.308
1	N51	1.068	-.425
1	N52	1.05	-.3
1	N53	.958	-.363
1	N54	.871	-.237
1	N55	.835	-.317
1	N56	.67	-.176
1	N57	.649	-.24
1	N58	.445	-.098
1	N59	.434	-.139
1	N60	.206	-.01
1	N61	.201	-.012

MEMBER FORCES

LC	Member Label	Sec	Axial[k]	Shear[k]	Moment[k-ft]
1	M91	1	-90.805	-.028	-.525
		2	-90.805	-.028	-.392
		3	-90.805	-.028	-.26

		4	-90.805	-.028	-.127	
		5	-90.805	-.028	.005	
1	M92	1	98.301	.003	.055	
		2	98.301	.003	.043	
		3	98.301	.003	.03	
		4	98.301	.003	.018	
		5	98.301	.003	.006	
1	M93	1	-88.026	.013	.006	
		2	-88.026	.013	-.056	
		3	-88.026	.013	-.118	
		4	-88.026	.013	-.18	
		5	-88.026	.013	-.242	
1	M94	1	96.559	.003	.064	
		2	96.559	.003	.048	
		3	96.559	.003	.032	
		4	96.559	.003	.016	
		5	96.559	.003	0	
1	M105A	1	159.783		.196	0
		2	159.783		.196	-.588
		3	159.783		.196	-1.176
		4	159.783		.196	-1.765
		5	159.783		.196	-2.353
1	M110A	1	.018	-.22	-3.004	
		2	.018	-.22	-2.18	
		3	.018	-.22	-1.357	
		4	.018	-.22	-.533	
		5	.018	-.22	.291	

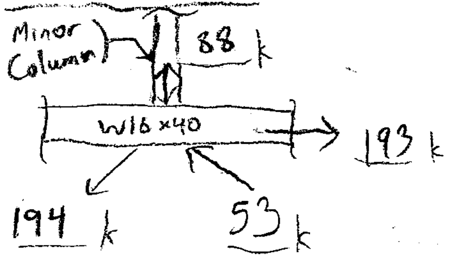
All critical members are under capacity



* Begin by taking braces to yield/buckling strength

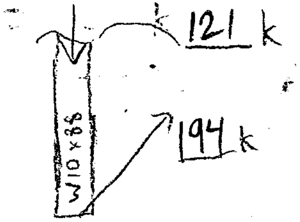
HSS 4½ x 4½ x 5/16 @ 53 k (buckling)
HSS 4½ x 4½ x 5/16 @ 194 k (tensile yield)

TOP OF BEAM



Test Beam for 193 k axial load
Test Minor Column for 88k axial load

BASE CONNECTION



Test Column for 121 k added axial load

COLUMN STRENGTH CHECK: W10x88

1.0D + 0.5L + 1.0E

$$\left[1.0(40) + (0.5)(91.7) + (0.2 \cdot \overset{.791}{D} \cdot S_{ds})\right] \cdot 6 \cdot 20 \cdot 30 \cdot \frac{1}{1000} = \left[(104.83) \cdot 6 \cdot 20 \cdot 30\right] + 121$$

$P_u = 498.4$ k

$\boxed{\phi P_n = 973 \text{ k}}$ ✓

CHECK W10x54

$\boxed{\phi P_n = 564 \text{ k}}$ ✓

CHECK W16×40 STRENGTH

$P_u = 193k$

$M_u = \frac{193.5 + 84.4}{2} = 139\ k\text{-ft.}$

$\phi M_n = 228\ k\text{-ft.}$

$\frac{kl}{r} = 76.4 \rightarrow F_e = \frac{29{,}000 \cdot \pi^2}{76.4^2} = 49$

$F_{cr} = (0.658^{50/49}) \cdot 50 = 32.6$ ksi

$P_n = A \cdot F_{cr} = 11.8 \cdot 32.6 = 384.9k$

$\phi P_n = 346.4k$

COMBINED LOADING

$\frac{P_u}{\phi P_n} + \frac{8M_u}{9\phi M_n} = 1.10 > 1.0$ ✗ → Try W16×50

$P_n = 14.7 \cdot 32.6 = 431.3k \rightarrow \phi P_n = 431.3k$

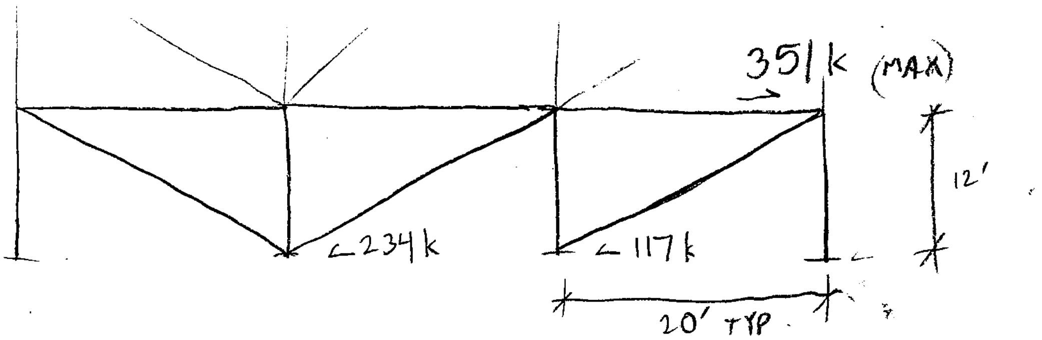
$\frac{P_u}{\phi P_n} + \frac{8M_u}{9\phi M_n} = 0.989 \leq 1.0$ ✓

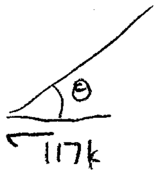
CHANGE TO W16×57 FOR ADDED STRENGTH

SIZE MINOR COLUMN

$P_u = 88k$

SELECT HSS 4×4×3/8, CAPACITY = 104 k ✓
REF. AISC TABLE 4-4





$$F_{brace} = \frac{117}{\cos\theta} = 117\left(\frac{23.32}{20}\right) = 136k$$

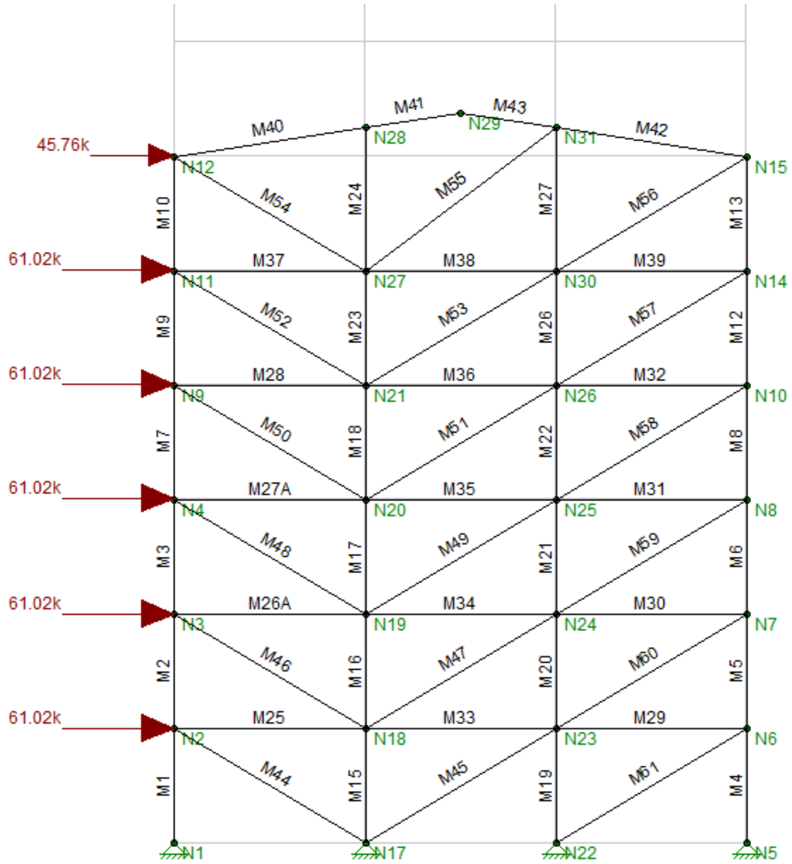
$$L = 23.32'$$

SELECT HSS 6 x 6 x 5/8 FOR PRELIMINARY DESIGN
CAPACITY = 150k @ 24' in compression (Ref. AISC Table 4-4)

SEE RISA 2D RESULTS ON NEXT PAGE FOR DETAILED STORY DRIFT ANALYSIS (p. L15)

- Story drift is adequate.
- Outside columns changed ~~to~~ HSS 6x6x 5/8
- Outside foundations are now drilled caissons

RISA 2D ANALYSIS IN EAST-WEST DIRECTION



JOINT DEFLECTIONS

LC	Joint Label	X [in]	Y [in]
1	N1	0	0
1	N2	.21	.05
1	N3	.512	.064
1	N4	.777	.058
1	N5	0	0
1	N6	.175	-.177
1	N7	.424	-.322
1	N8	.673	-.43
1	N9	1.01	.045
1	N10	.894	-.503
1	N11	1.216	.025
1	N12	1.489	.045
1	N14	1.098	-.582
1	N15	1.208	-.587
1	N17	0	0
1	N18	.29	-.107
1	N19	.549	-.194
1	N20	.784	-.265
1	N21	.99	-.321

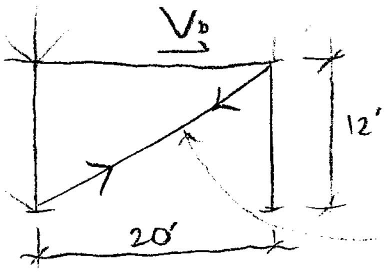
1	N22	0	0
1	N23	.225	-.135
1	N24	.486	-.245
1	N25	.727	-.332
1	N26	.94	-.397
1	N27	1.185	-.39
1	N28	1.538	-.389
1	N30	1.127	-.466
1	N31	1.242	-.466
1	N29	1.395	.513

MEMBER FORCES

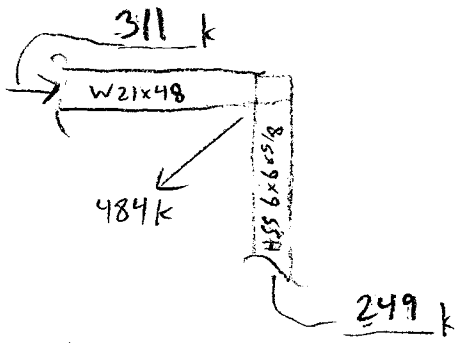
LC	Member Label	Sec	Axial[k]	Shear[k]	Moment[k-ft]
1	M1	1	-97.905	.089	0
		2	-97.905	.089	-.266
		3	-97.905	.089-	.532
		4	-97.905	.089	-.798
		5	-97.905	.089	-1.064
1	M4	1	347.332	0	0
		2	347.332	0	0
		3	347.332	0	0
		4	347.332	0	0
		5	347.332	0	0
1	M44	1	187.415	-.141	-1.328
		2	187.415	-.141	-.506
		3	187.415	-.141	.317
		4	187.415	-.141	1.139
		5	187.415	-.141	1.961
1	M45	1	-149.225	-.023	-1.017
		2	-149.225	-.023	-.881
		3	-149.225	-.023	-.746
		4	-149.225	-.023	-.61
		5	-149.225	-.023	-.474
1	M61	1	-71.697	.026	0
		2	-71.697	.026	-.15
		3	-71.697	.026	-.299
		4	-71.697	.026	-.448
		5	-71.697	.026	-.597

Based on data from this model, make the following structural changes:

- Change Exterior columns to HSS 6x6x5/8 (Capacity= 360k)
- Change Exterior foundations to drilled caissons similar to interior foundations



HSS 6×6×5/8, CAPACITY = 484 k (tension)



Test Beam for 311 k axial load

Test Column for 249 k axial load

COLUMN

$$\left[[1.0(40)+0.5(117)+(0.2S_{ds}(40))]\cdot 6\cdot 10\cdot 30\cdot \frac{1}{1000}\right]+249=P_u$$

$P_u = 437.7$ k

$\emptyset P_n = 360$k X

CHANGE TO HSS 7×7×5/8, FULL HEIGHT (CAPACITY = 470k ✓)
REF. AISC TABLE 4-4

CHECK BEAM

$\emptyset P_n = 596$ k (REF. P. L9 FOR CALCULATION)

$\emptyset M_n = 390$ k-ft.

$P_u = 311$ k

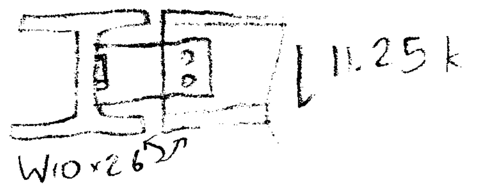
$M_u = 156$ k-ft.

COMBINED LOADING.

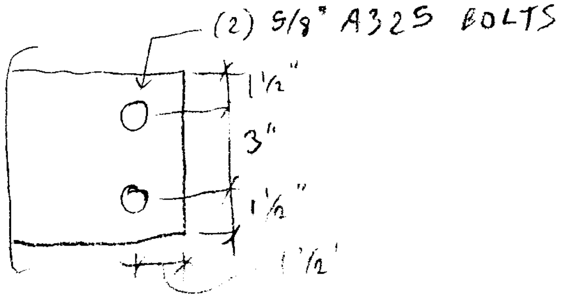
$$\frac{P_u}{\emptyset P_n}+\frac{8M_u}{9\emptyset M_n}=0.877<1.0 \checkmark$$

W21×48 IS ADEQUATE

EXT. BEAM GIRDER CONNECTION (GRAVITY)



- Check shear tab thickness
- Check bolt strength
- Check for block shear tearout
- Check welds to girder



BOLTS

Strength of (1) A325 Bolt: 12.4 k > 11.25 k ✓

BENDING

Start w/ 3/8" Thick Steel A36 Plate

$e_{max} = 9" \rightarrow M_{max} = 11.25k \cdot 9" = 102$ k-in.

$Z_R = \frac{bh^2}{4} = \frac{3/8 \cdot 6^2}{4} = 3.375$ in.^{3}

$M_n = Z F_y = 3.375 \cdot 36 = 121.5$ k-in. $\rightarrow \phi M_n = 109$ k-in. ✓

BLOCK SHEAR

$A_{nv} = [4.5" - (1.5 \cdot 11/16)] \cdot 3/8" = 1.30$ in.^{2} $A_{nt} = (1\frac{1}{2} - \frac{1}{32}) \cdot 3/8 = .433$ in.^{2}

$R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} = 70.4k$

OR $= 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} = 61.6k$ $\phi = 0.75$

$\boxed{\phi R_n = 46.2\text{ k} > 11.25\text{k}}$ ✓

WELDS

Use 70 ksi welds, 3/16" ($F_{EXX} = 70$ ksi)

$A_w = l \cdot 3/16"$

$\frac{11.25k}{0.75} = 15k = R_{n\,max}$

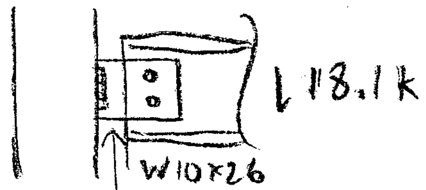
$R_n = 0.6(70)(\frac{\sqrt{2}}{2})(\frac{3}{16}) l = 15$ (REF. AISC FORMULA J2-1)

$5.568 l = 15$

$l_{min} = 2.69"$

$\boxed{\text{Use } 3/16 \times 3" \text{ 70 ksi WELDS}}$

EXT. GIRDER-COLUMN CONNECTION (GRAVITY)



SEE PREVIOUS PAGE FOR CONNECTION DETAIL

BOLTS Strength of (2) A325 Bolts: 24.8 k > 18.1 k ✓

BENDING Start w/ 1/2" Thick A36 Steel Plate $e_{max} = 4''$

$M_u = 18.1 \cdot 4 = 72.4 \text{ k-in.}$

$Z_{pl} = \frac{bh^2}{4} = \frac{1/2 \cdot 6^2}{4} = 4.5 \text{ in.}^3$

$M_n = 4.5 \cdot 36 = 162 \text{ k-in.} \rightarrow \underline{\phi M_n = 145.8 \text{ k-in.} > 72.4 \text{ k-in.}}$ ✓

BLOCK SHEAR $\phi R_n = 46.2\text{k} \cdot (1/2)/(3/8) = 61.6\text{k} > 18.1\text{k}$ ✓

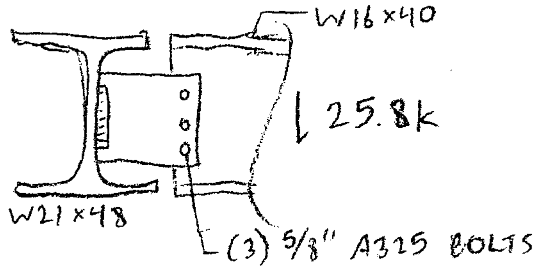
↳ FROM PREVIOUS PAGE

WELDS FROM PREVIOUS PAGE, $5.568 l = \frac{18.1}{.75} = 24.1 \text{ k}$

$l_{min} = 4.33''$

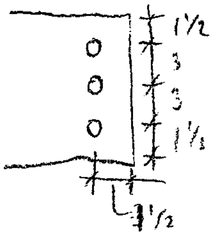
$\boxed{\text{USE } 3/16'' \times 5'' \text{ 70 ksi WELDS}}$

INT. BEAM-GIRDER CONNECTION (GRAVITY)



$e_{max} = 8"$

BOLTS



Strength of A325 Bolt (5/8"): 12.4k

$12.4 \cdot 3 = 37.2k > 25.8k$ ✓

PLATE BENDING

Start w/ ½" Thick Steel A36 Plate

$M_u = 8 \cdot 25.8k = 206.4$ k-in.

$Z_\ell = \frac{bh^2}{4} = \frac{0.5 \cdot 9^2}{4} = 10.13 \text{ in.}^3$

$M_n = Z f_y = 10.13 \cdot 36 = 364.5$ k-in. → $\boxed{\phi M_n = 328 \text{ k-in.} > 206.4 \text{ k-in.}}$ ✓

BLOCK SHEAR

$A_{nv} = [7.5 - (2.5 \cdot \frac{11}{16})] \cdot \frac{1}{2}" = \underline{2.89 \text{ in.}^2}$ $A_{nt} = (1\frac{1}{2} - \frac{11}{32}) \cdot \frac{1}{2}" = \underline{0.578 \text{ in.}^2}$

$R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} = 134.1$ k

OR $= 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} = \underline{114.5 \text{ k}}$ ←

$\boxed{\phi R_n = 85.9k > 25.8k}$ ✓

WELDS

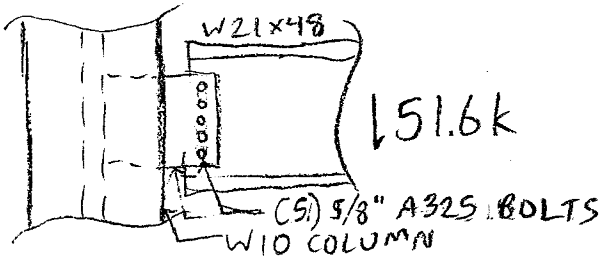
$R_n = \frac{25.8}{.75} = 34.4k$

$R_n = 0.6(70)\left(\frac{\sqrt{2}}{2}\right)\left(\frac{3}{16}\right) \ell = 34.4$

$\ell_{min} = 6.18"$

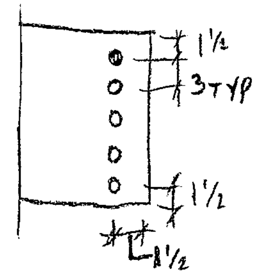
$\boxed{\text{USE } 3/16" \times 7" \text{ 70 ksi WELDS}}$

INT. GIRDER-COLUMN CONNECTION (GRAVITY)



$e_{max} = 9"$

BOLTS



Strength of (1) 5/8" A325 Bolt: 12.4k

$12.4 \cdot 5 = 62\text{k} > 51.6\text{k}$ ✓

BENDING

Start w/ ½" Thick Steel A36 Plate

$M_u = 9 \cdot 51.6 = \underline{464.4 \text{ k-in.}}$

$Z_R = \frac{bh^2}{4} = \frac{0.5 \cdot 15^2}{4} = 28.125 \text{ in.}^3$

$M_n = Z_R F_y = 28.125 \cdot 36 = 1012.5 \text{ k-in.} \rightarrow \boxed{\phi M_n = 911.3 \text{ k-in}} > 464.4 \text{ k-in.}$ ✓

BLOCK SHEAR

$A_{nv} = [13.5 - (4.5 \cdot {}^{11}/_{16})] \cdot {}^{1}/_{2}" = 5.2 \text{ in.}^2$ $\quad A_{nt} = (1\tfrac{1}{2} - \tfrac{11}{32}) \cdot {}^{1}/_{2}" = 0.578 \text{ in.}^2$

$R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} = 214.5 \text{ k}$

$\text{OR} = 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} = 179.3 \text{ k} \leftarrow$

$\boxed{\phi R_n = 134.5 \text{k} > 51.6 \text{k}}$

WELDS

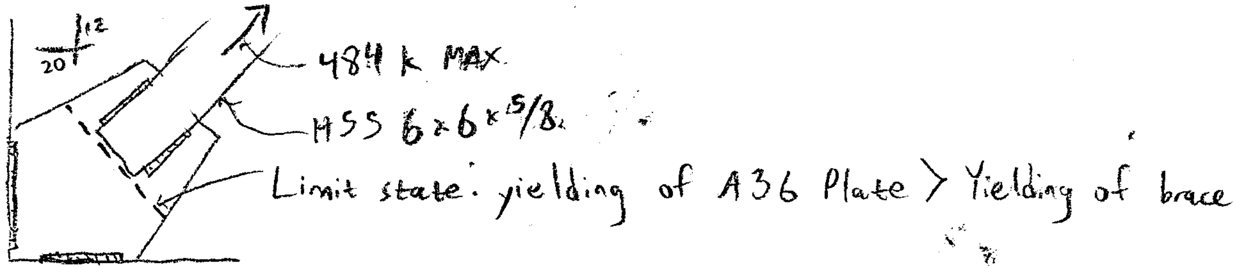
$R_n = 0.6 (70) (\frac{\sqrt{2}}{2}) (\frac{3}{16}) l_{min} = R_{n_{max}}$

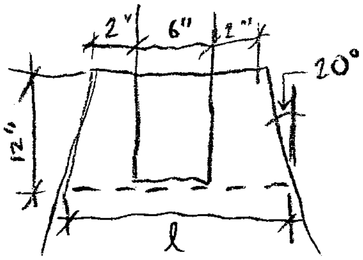
$R_{n_{max}} = \frac{51.6}{.75} = 68.8$

$l_{min} = 12.355"$

$\boxed{\text{USE } {}^{3}/_{16}" \times 13" \text{ 70 ksi WELDS}}$

GUSSET PLATE DESIGN (E-W BRACES)





$\tan 20 = \frac{x}{12} = .364$

$x = 4.367''$

$l = 10 + 2x = 18.73''$

$A_g = 18.73\,t$

$P_u = 1.1\,P_{Br} = 1.1(484) = 532\text{ k}$

$532 = (18.73\,t)\cdot 36\text{ ksi}\cdot 0.9$

$t = 0.876''$ min. $\rightarrow$ $\boxed{\text{Use min. } \underline{1}'' \text{ thick plate}}$

WELDS

$-F_x = 1.1\cdot 484\cdot\frac{20}{23.3} = 457\text{ k}$ $\qquad \frac{457}{0.75} = 609\text{k} = R_{n\,max}$

$609 = 0.6(70)\left(\frac{\sqrt{2}}{2}\right)\left(\frac{3}{4}\right) l\cdot(2\text{ sides})$

$l_{min} = 13.67''$ $\rightarrow$ $\boxed{\text{Use } \underline{3/4}'' \times 14'' \text{ WELDS EACH SIDE}}$

$-F_y = 1.1\cdot 484\cdot\frac{12}{23.3} = 274$ $\qquad \frac{274}{0.75} = 365.6\text{k}$

$365.6 = 0.6(70)\left(\frac{\sqrt{2}}{2}\right)\left(\frac{3}{4}\right) l\ (2)$

$l_{min} = 8.21''$ $\rightarrow$ $\boxed{\text{Use } 3/4'' \times 9'' \text{ WELDS EACH SIDE}}$

$-F_{Br} = 1.1\cdot 484 = 532\text{ k}$ $\qquad \frac{532}{0.75} = 709\text{ k}$

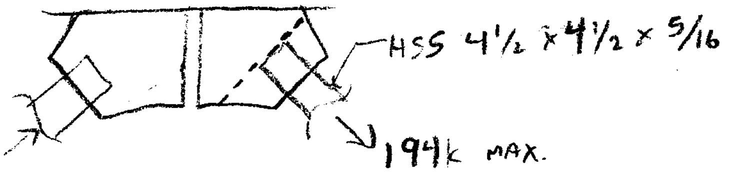
$709 = 0.6(70)\left(\frac{\sqrt{2}}{2}\right)\left(\frac{3}{4}\right) l\,(4)$

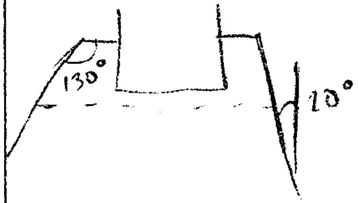
$l_{min} = 7.96''$ $\rightarrow$ $\boxed{\text{Use } 3/4'' \times 9'' \text{ WELDS ALL SIDES}}$

BLOCK SHEAR TEAROUT

$\phi R_n = 0.75\left[(.6\cdot 36\cdot 24) + (1.0\cdot 58\cdot 6)\right] = 649.8\text{ k} > 532\text{k}$ ✓

GUSSET PLATE DESIGN (MID BEAM)





$l = 10 + 12\tan 20° + 12\tan 40° = 24.4''$

$A_g = 24.4t$ (start w/ $t = \frac{1}{2}''$)

$P_u = 1.1(194) = 213.4k$

$24.4 \cdot 0.5 \cdot 36\text{ksi} \cdot 0.9 = 395.3k > 213.4k$ ✓

WELDS

$-F_y = 1.1 \cdot 194 \cdot \frac{15}{19.2} = 166.6\ k$ Start w/ 15" of ½" weld each side

$-F_x = 1.1 \cdot 194 \cdot \frac{12}{19.2} = 133.3\ k$

$F_{EXX} = 0.6(70)\left(\frac{\sqrt{2}}{2}\right)\left(\frac{1}{2}\right)(12)(2\text{ sides}) = 445.4\ k$ either direction

$\phi F_{EXX} = 334\ k$

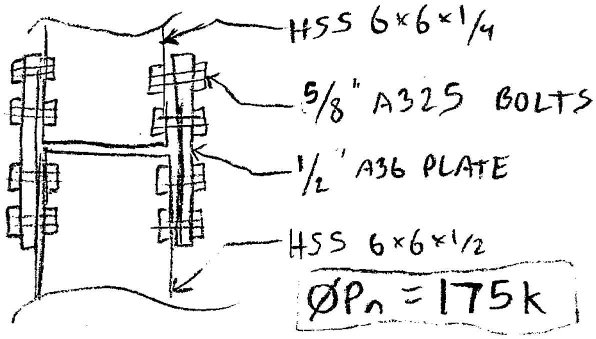
COMBINED LOADING: $\frac{166.6}{334} + \frac{133.3}{334} = 0.898 < 1.0$ ✓

$\boxed{\text{Use } \tfrac{1}{2}'' \times 15''\ 70\text{ KSI WELDS EACH SIDE}}$

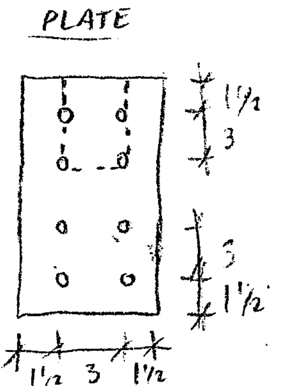
BLOCK SHEAR TEAROUT

$\phi R_n = 0.75[(12 \cdot 36 \cdot 0.6) + (58 \cdot 1.0 \cdot 3)] = 324.9k > 213.4k$ ✓

COLUMN SPLICE AT 4TH FLOOR (HSS)



$\boxed{\phi P_n = 175k}$



Bolt Group Capacity:

BLOCK SHEAR TEAROUT

$A_{nv} = [4.5-(1.5\cdot 11/16)]\cdot 4\cdot 1/2 = 7.6\text{ in.}^2$ $\quad A_{nt} = \left(3-\frac{11}{16}\right)\cdot \frac{1}{2}\cdot 2 = 2.31\text{ in.}^2$

$R_n = 0.6F_uA_{nt} + U_{bs}F_uA_{nt} = 398k$

$R_n = 0.6F_yA_{gv} + U_{bs}F_uA_{nt} = \underline{328k} \leftarrow$

$\boxed{\phi R_n = 246k} > \underline{175k}$ ✓

HSS BOLT GROUP TEAROUT

$A_{nv} = 3.8\text{ in.}^2$ $\quad A_{nt} = 1.16\text{ in.}^2$

$R_n = 0.6F_uA_{nt} + U_{bs}F_uA_{nt} = 254k$

$R_n = 0.6F_uA_{nt} + U_{bs}F_uA_{nt} = 209k \leftarrow$

$\boxed{\phi R_n = 157k} < 175k$ ✗

Locate 1st bolt 2" from edge of HSS

$F_y = 46\text{ ksi}$

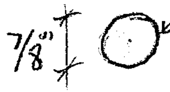
$F_u = 74\text{ ksi}$

$A_{nv} = 3.8\left(\frac{5}{4.5}\right) = 4.22\text{ in.}^2$ $\quad A_{gv} = 5''$

$R_n = 0.6F_yA_{gv} + U_{bs}F_uA_{nt} = 287k$

$\boxed{\phi R_n = 215k} > 175k$ ✓

DIAPHRAGM BRACE CONNECTIONS (WELDED)



3/16" WELD ALL AROUND (70 ksi)

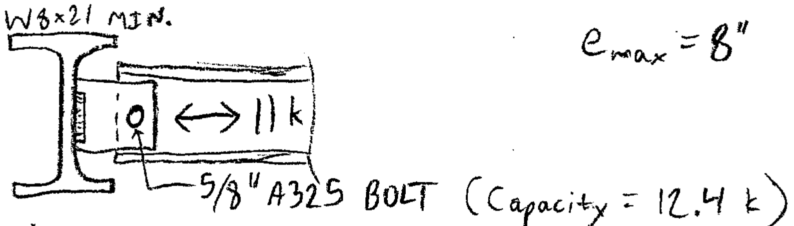
MAX. DEMAND = 20.4 k

$\ell = C = 2\pi r = 7/8\,\pi = 2.75" \cdot 1.1_{(\text{angled})} = 3.025"$

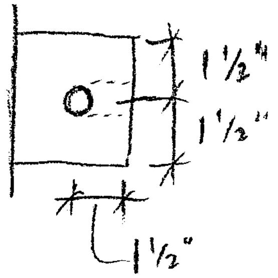
$R_n = 70(\sqrt{2}/2)(3/16)(3.025) = 28.1\text{ k}$

$\phi R_n = 0.75 \cdot 28.1\text{k} = 21.05\text{ k} > 20.4\text{ k}$ ✓

DIAPHRAGM ANGLE SECTION BRACES



$e_{max} = 8"$



BLOCK SHEAR TEAROUT

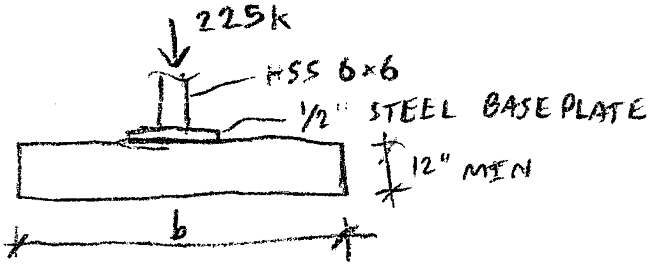
$A_{nv} = 2 \cdot 3/8" [1.5 = 1.125\text{ in.}^2$ $\quad A_{nt} = 3/8 \cdot 11/16 = 0.258\text{ in.}^2$

$R_n = .6 F_u A_{nv} + U_{bs} F_u A_{nt} = 54\text{k}$

$R_n = .6 F_y A_{gv} + U_{bs} F_u A_{nt} = \underline{39\text{k}} \Leftarrow$

$\boxed{\phi R_n = 29.4\text{ k} > 11\text{k}}$ ✓

EXT. PAD FOOTINGS

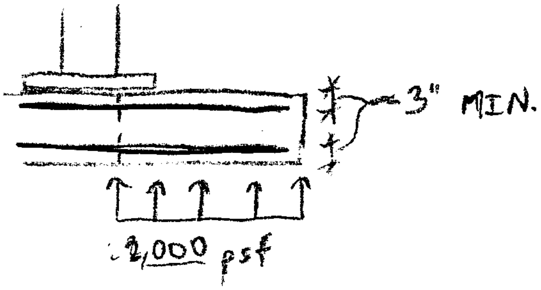


Assume Soil Type D
per 2013 CBC Table 1806.2
$\rho = 2000$ psf

$\frac{225k}{\rho} = 112.5$ sf $\rightarrow$ $b = 10.61'$

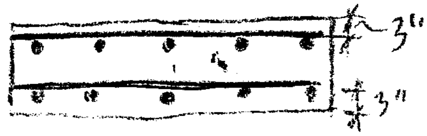
USE 11' SQ. CONC. FTG.

CHECK BENDING



Assume: 4ksi Concrete
60 ksi Rebar
Start #4 bars @ 6"oc each way

$M_u = 2{,}000 \text{ psf} \cdot 0.5' \cdot 5' = 5.0$ k-ft. $= 60$ k-in.



$d = 12 - 3 - \frac{0.5}{2} - 0.5 = 8"$

$A_s f_y = 0.85 f'_c ab$

$0.2 \cdot 60 = 0.85 \cdot 4 \cdot 6 \cdot a$

$a = 0.588"$

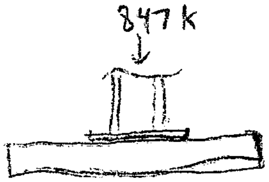
$M_n = A_s f_y \left(d - \frac{a}{2}\right) = 0.2 \cdot 60 \left(8 - \frac{.588}{2}\right) = 92^{4}$ k-in.

$c = \frac{a}{0.85} = 0.692$ $\frac{c}{d} = 0.086$ $\rightarrow$ $\phi = 0.9$

$\phi M_n = 82$ k-in. > 60 k-in. ✓

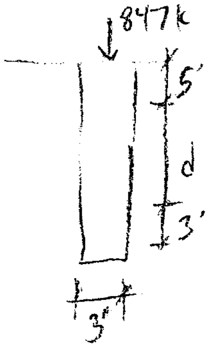
USE #4 BARS @ 6"OC EACH WAY

INT. PAD FOOTINGS



$\rho = 2000$ psf

$\frac{847,000}{2000} = 423.5 \rightarrow$ 20.6' sq. ftgs. → Try 3' ⌀ Caisson



$Q_{ult} = \beta P_v A_{side} + q_b A_{base}$ (Sand)

Assume: $\beta = 1.0$, $q_b = 40$,

Factor of Safety = 2.0

$\alpha = 0.25$ Per CBC 2013 Table 1806.2

$P_v = 150$ psf/ft. Per CBC 2013 Table 1806.2

$Q_{allow} = \frac{Q_{ult}}{2.0} = 847$

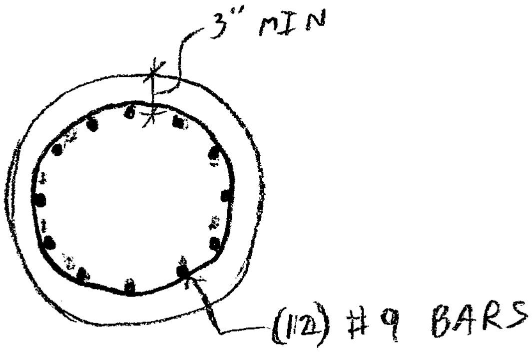
$Q_{ult} = 1694\text{ k} = \left(\frac{150 \cdot d}{2}\right)(3\pi d) + 2000(\pi \cdot 1.5^2)$

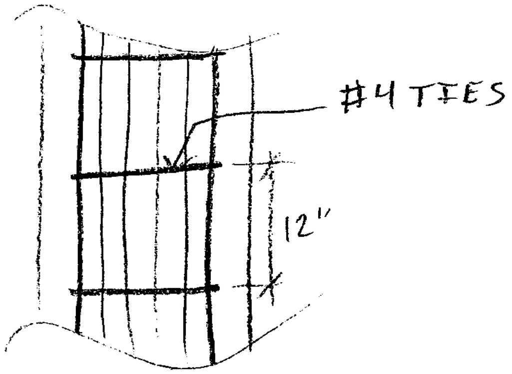
$1679.8 = 706 d^2$

$d = 48.78'$

$48.78 + 5 + 3 = 56.78$

USE 60' DEEP 3' ⌀ CAISSONS WITH REINF. SHOWN BELOW





12×1.0 = 12 in.² > 1.0% MINIMUM REQUIRED BY ACI 318

SECTION III

FUTURE DESIGNS

Analysis of Results

The overall design of the facility largely hinged on allowing natural light to extend deep into the building on every floor. Everything in the structural and architectural design are centered on optimizing that factor in order to maximize the production of crops. The result was a long, slender rectangular building with broad walls of clear windows on all sides. The building is structurally feasible and even lighter than a typical office or residential building of the same size. The lack of a heavy concrete decking system both allowed more light in and reduced the overall weight of the structure.

Geography plays a major role in the dynamics of natural light, effectively dictating the orientation of the building. In low-latitude regions ranging from the tropics all the way to middle temperate zones, sunlight enters the building more effectively from the east and west than from the southern face. Although the numbers would change with any change in latitude relative to San Luis Obispo, the major axis orientation would be the same for most regions.

Economically, it was no secret that the cost of construction would be the dominant barrier to creating a facility like this. The ability of the facility to produce in a cost-effective manner competes against the existing environment in the area. An extremely fertile area like California would likely see poor results when compared to existing farmland because the natural soil is immensely productive already. The only element of California granting an advantage to vertical farms is the high price of land in many areas. A vertical farm could occupy a much smaller physical land area than a traditional farm, thus increasing the efficiency of the space per square foot.

The design would be more viable in regions where traditional farming on a large scale is inefficient or not even an option. Such areas would include desert regions where water is scarce. An enclosed facility with little to no openings for water to evaporate from it would be highly capable of containing and reusing its water supply. An open-air farm in an arid environment would be susceptible to losing much of the water used on it to evaporation, which drives up the already high cost of fresh water in these regions. Another climate in which vertical farming could flourish would be the lands near the poles which are subject to regular freezing temperatures. The harsh outside environment is unfavorable to most crops due to snow, low temperatures, and permafrost conditions in the soil. A contained and heated facility would be far more productive because it has the ability to raise crops at nearly any time of year instead of the short summer growing season. This particular design also works because the natural light from the south would extend deep into the building. An immense south-facing facility could be the most viable design type for these regions. Urban environments are also locations that become favorable to vertical farms, gaining their economic advantage by cutting down on transportation costs from traditional farms located up to several hundred miles away. Land prices in downtown areas would probably be too expensive for vertical farms, but neighborhoods on the outskirts of cities that house mostly industrial buildings and warehouses could be a relatively cheap site location. The crops' journey from farm to market would typically be less than ten miles instead of hundreds, thus slashing transportation costs and even bringing tourist attractions to the first cities that build these facilities.

Research Potential

The patterns and techniques used in the design of this facility come from a largely untapped well of potential research opportunities. The Student Experimental Fields near the Rodeo Arena at the Cal Poly campus offer an easy opportunity to begin that research. There is plenty of open space for testing scale-model versions of the design that are exposed to the actual climate conditions of the area. Future students could use the research area to conduct tests regarding natural lighting conditions, story height, different building systems, different architectural designs, and more. Additional laboratory tests could be conducted to optimize the performance of the conveyor systems, irrigation systems, and possibly the introduction of digital monitoring of the crops and machines. The expansion of research in this field could potentially encompass dozens of majors at Cal Poly, with much left to be discovered by pioneering research.

One of the most readily testable experiments arising from this project is the corroboration of growth patterns under obscured natural light. A model story could be created by sowing a plot of spinach seeds in a predefined area and building an opaque cover directly above it to mimic the story above. That cover could be as simple as a series of plywood planks on 2x2 posts that matches the crop area below. An ordinary, unobstructed plot of spinach plants could be grown nearby and used as a control group. From there, the results would ideally show the point at which spinach plants stop growing effectively in the building. That would be the limit of natural light penetration for the building.

The results of the model story experiment open up several more directions in which research can go. For example, a similar experiment could also introduce a third group that employs the use of conveyor systems in order to empirically measure the extension of natural light depth that they provide. The use of artificial light supplements and hybrid lighting could also be explored with the use of LED components in the darker regions of the buildings. The artificial lighting could be powered by the external solar panels allowed in the original design, which would offset the long-term cost of electricity.

Another branch of structural engineering could potentially adopt projects of this type in the way of integrating them into existing buildings. A retrofit project could attempt to house the necessary growth systems in a building initially designed for other purposes. Future ARCE students could draw upon this design in order to find ways of incorporating it into a mixed-use structure.

Much design work is still needed in order to bring the theorized conveyor systems into reality. Departments such as Mechanical and Electrical Engineering could find research potential in this area. The largest challenge would most likely be the integration of an irrigation system into the conveyors, linking hoses and water-tight connections to the moving components. A modular design in which systems can easily change size or configuration would be a significant advantage for the overall function of the facility. However, this would be difficult to integrate with the irrigation system and would require extensive design work. Despite the amount of work required, the conveyors described in this project are likely a feasible concept and could be implemented into structures in the future.

Furthermore, the design could be put up to real-world conditions not initially considered in the design model. For example, the actual peak amount of sunlight energy and heat does not occur precisely in a cardinal direction, but at an angle somewhere in the southwest quadrant. Models could be created to isolate that angle, and then another iteration of design could have the building oriented toward that angle. This opens up a series of experimental architectural designs that could be developed in order to find configurations that maximize natural light exposure.

Conclusion

Although the economic willpower to invest in a large-scale structure like this has yet to come, the design does have potential applications that are worth the input of resources. The first regions of the world to invest in this type of design would be the regions in which traditional farming is inefficient or even unfeasible. Such areas include deserts plagued by drought, land area near the poles that experience permafrost conditions, and major cities with little to no arable farmland nearby. Vertical farming gains its advantages in these regions by offering immense productivity relative to the alternative, or by saving shipping costs from a distant location. The technological ability to create an efficient facility is already available. The world is now ready for the rise of the vertical farm.

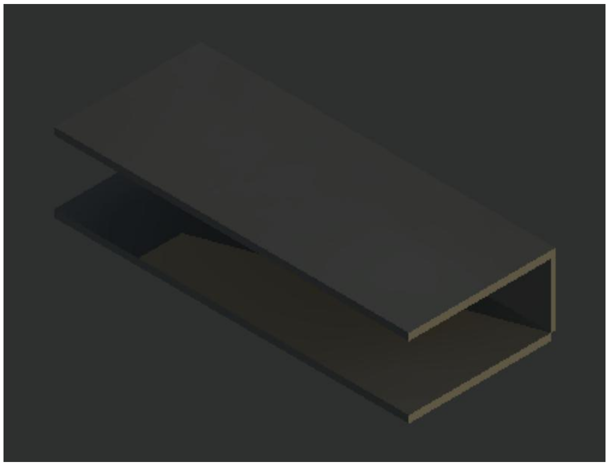
PART IV

APPENDIX

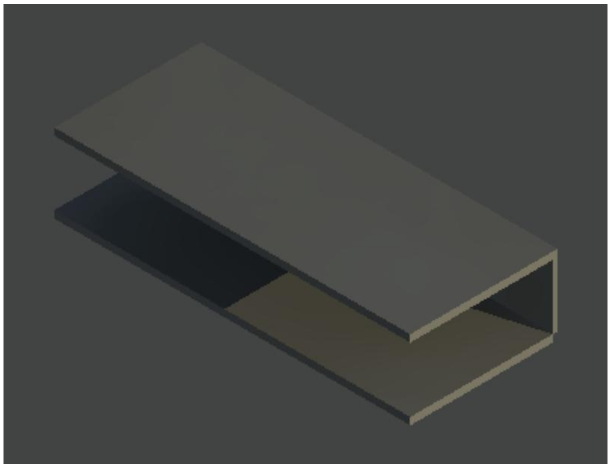
Light Renderings

The following images were obtained using the rendering tool in Revit 2015. A model story measuring 20 feet by 50 feet with a ceiling 10 feet directly above was created in the program and illuminated with a light source designed to approximate the sun's position at a given time of day. The sample shown here is the lighting throughout the day of June 20th, the summer solstice, in San Luis Obispo, CA.

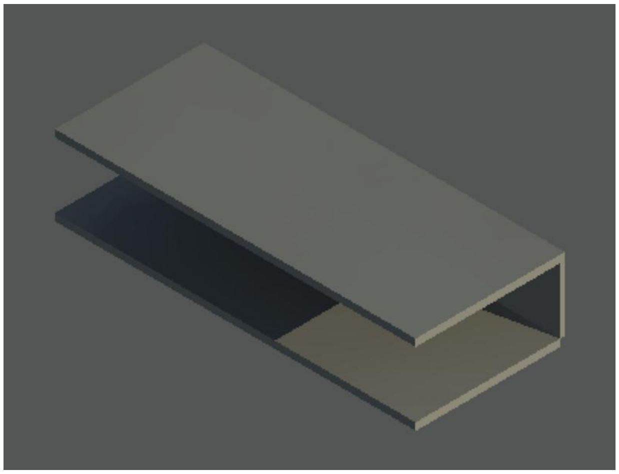
SLO Summer Solstice, 6:00 AM



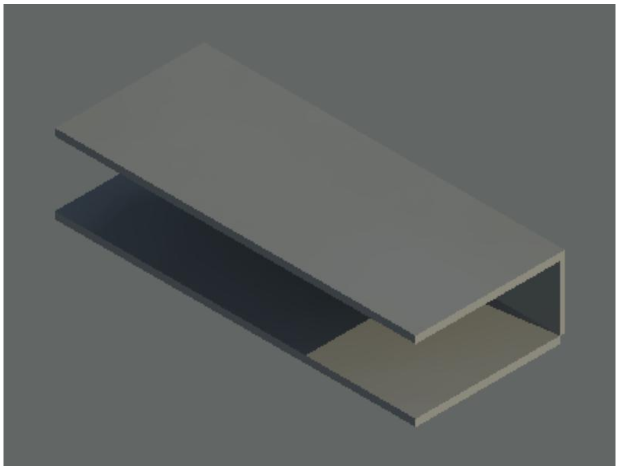
SLO Summer Solstice, 6:30 AM



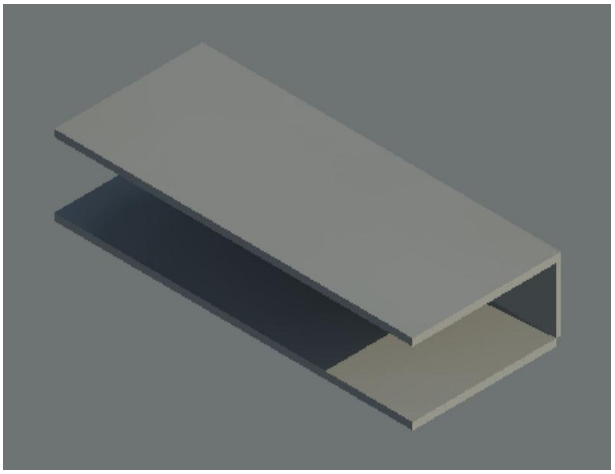
SLO Summer Solstice, 7:00 AM



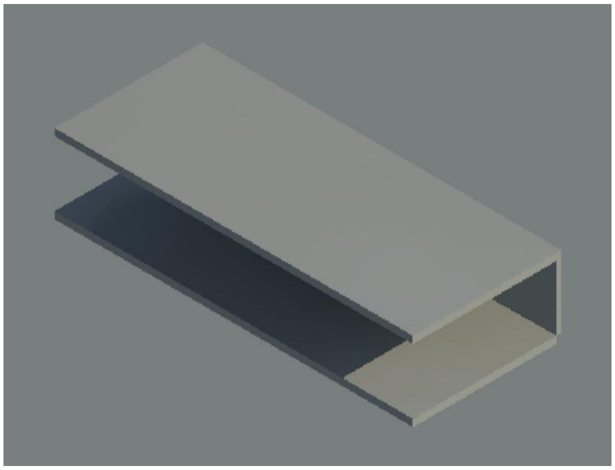
SLO Summer Solstice, 7:30 AM



SLO Summer Solstice, 8:00 AM



SLO Summer Solstice, 8:30 AM



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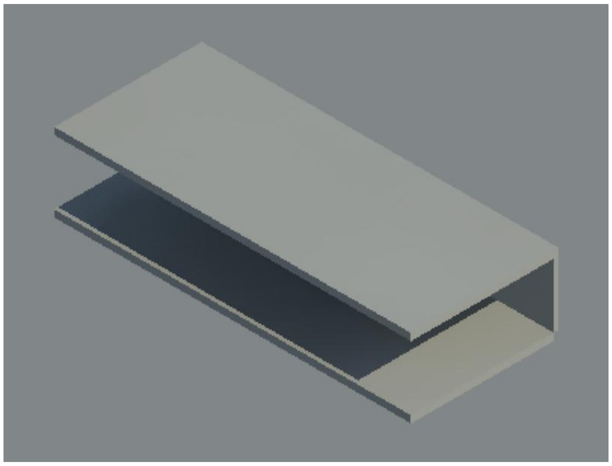

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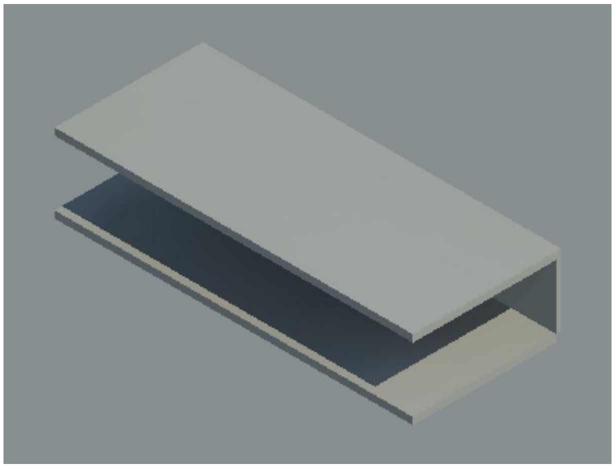

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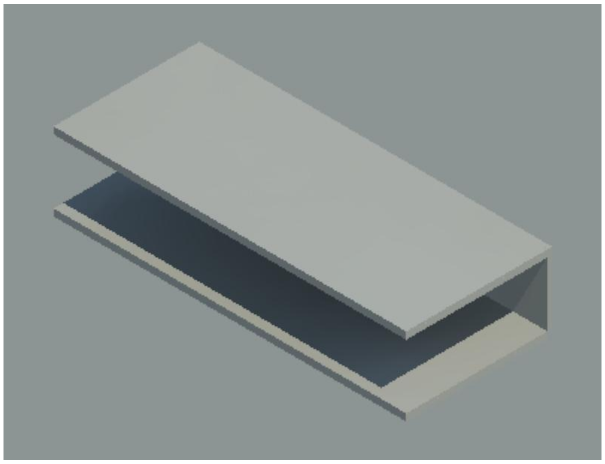

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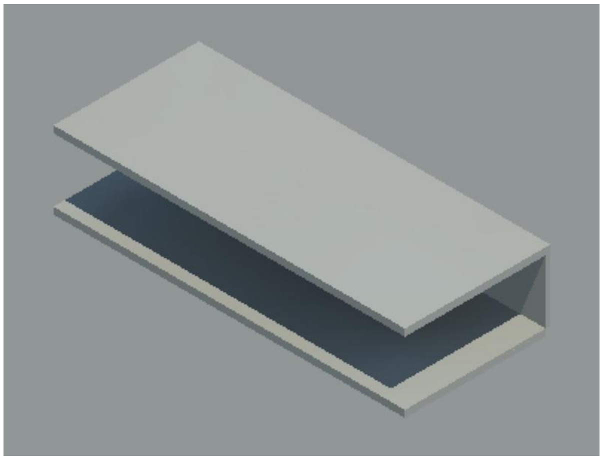

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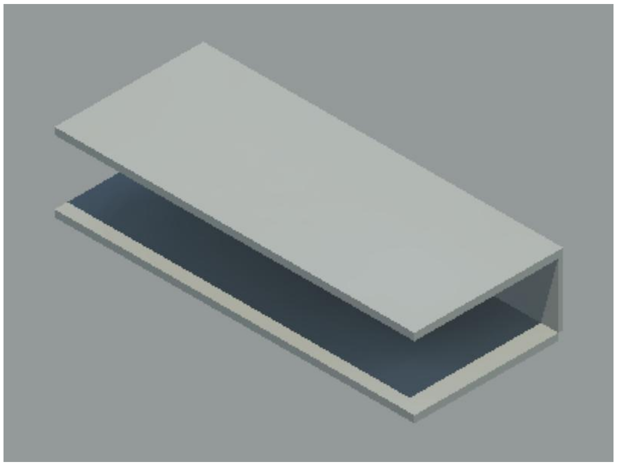

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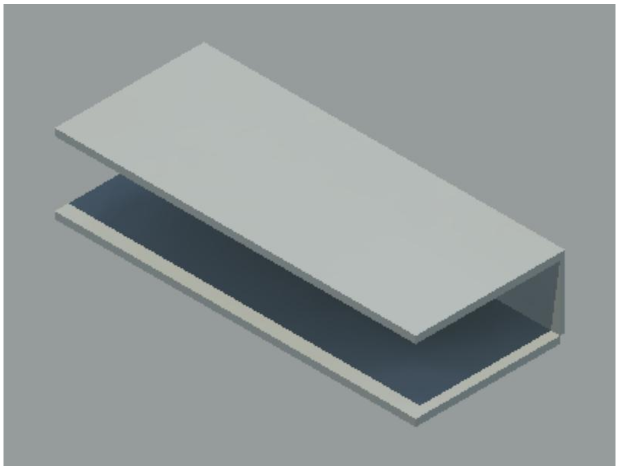
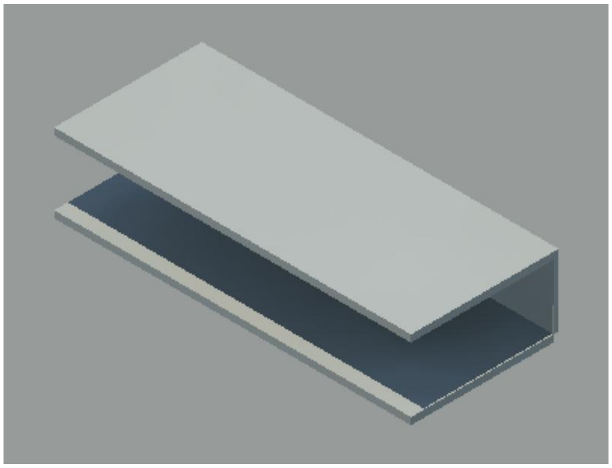


SLO Summer Solstice, 12:00 PM



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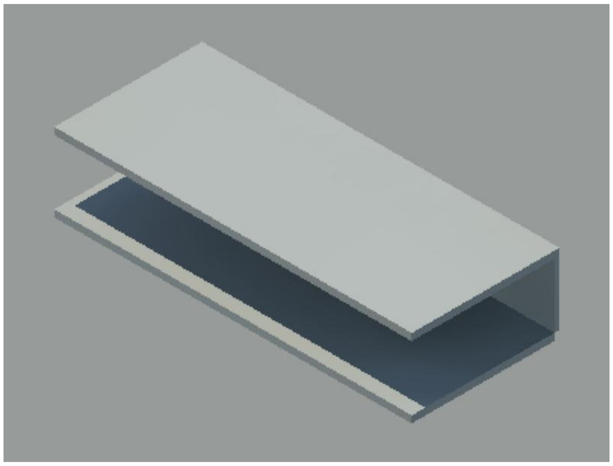

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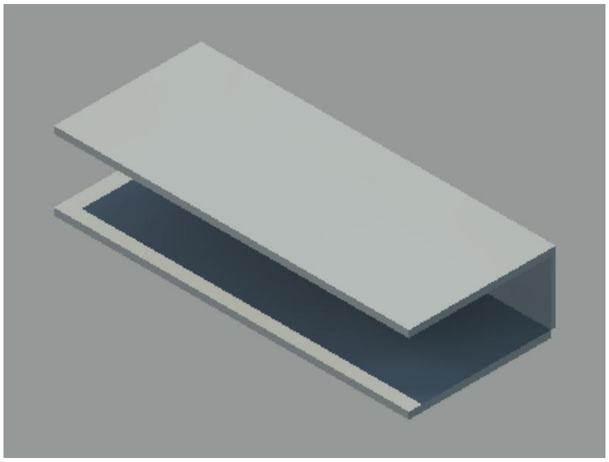
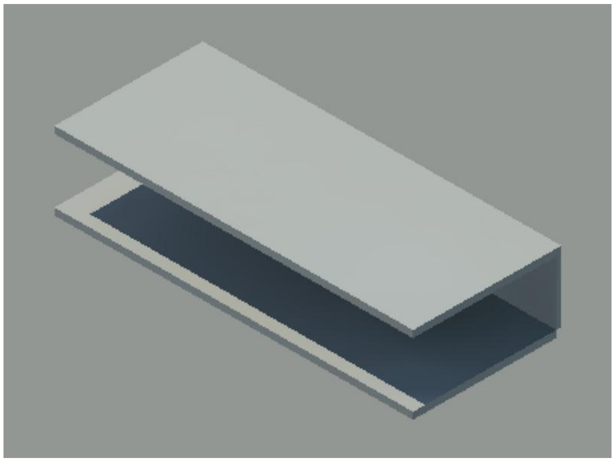


SLO Summer Solstice, 1:30 PM



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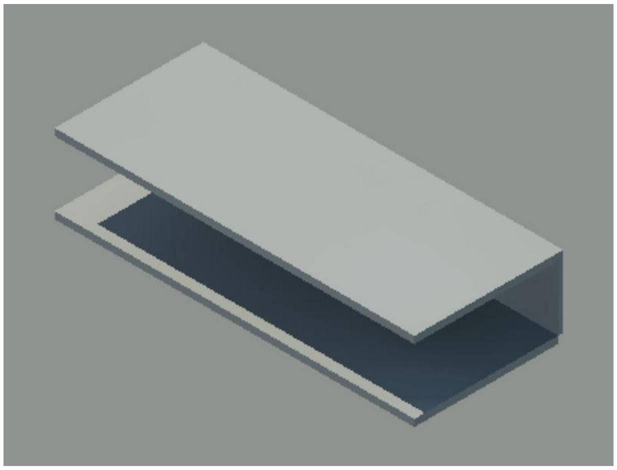

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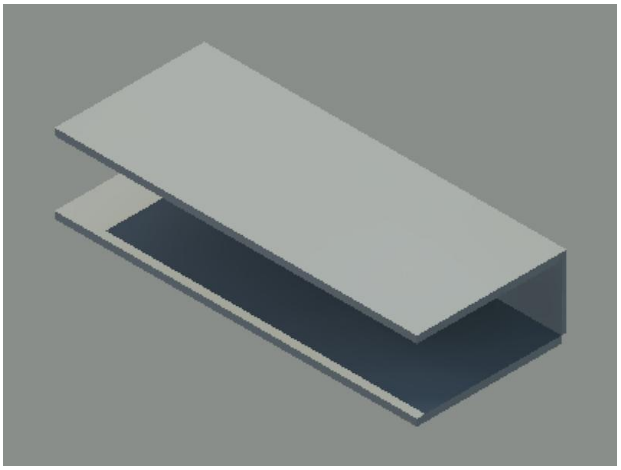

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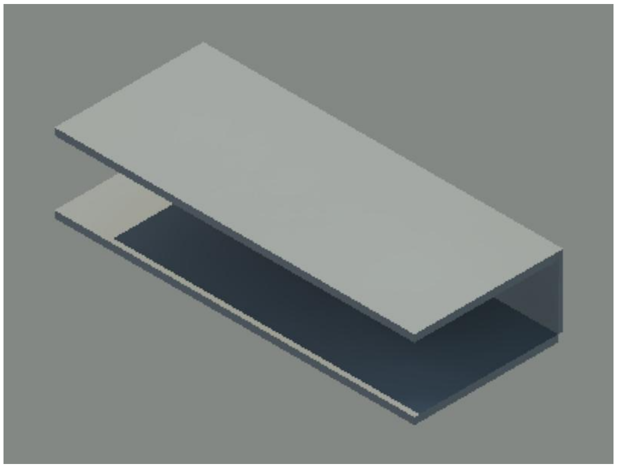

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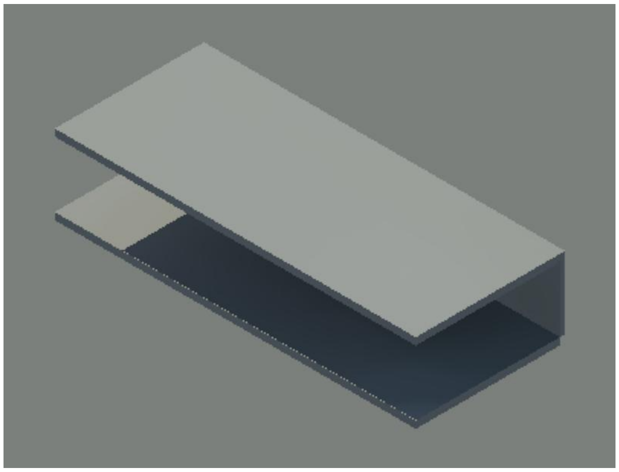

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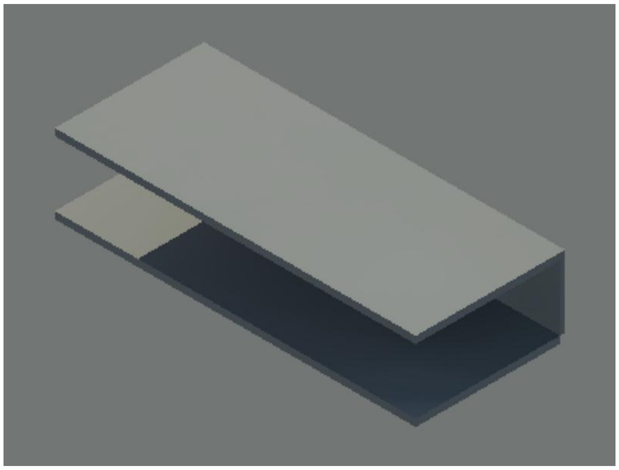
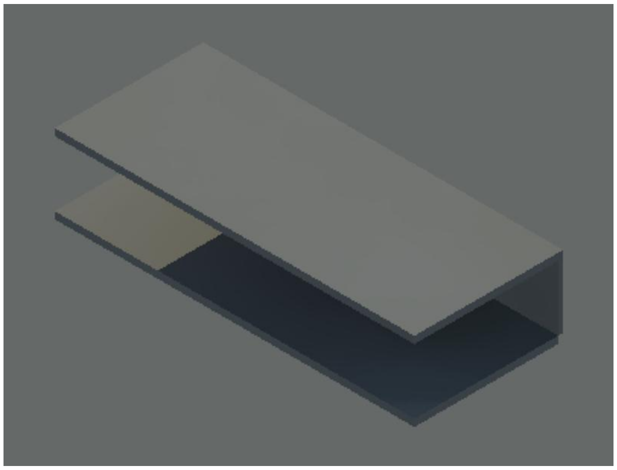


SLO Summer Solstice, 4:30 PM



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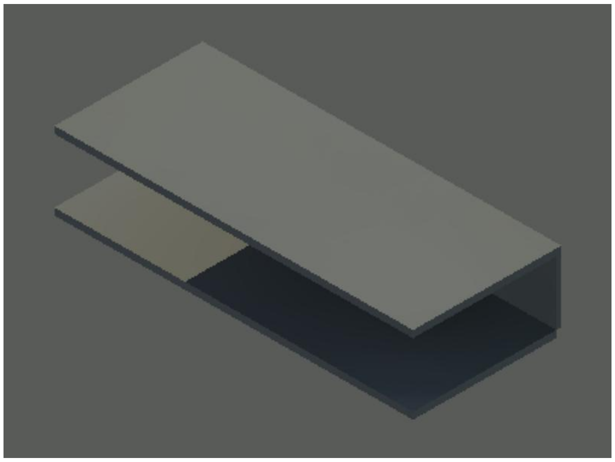
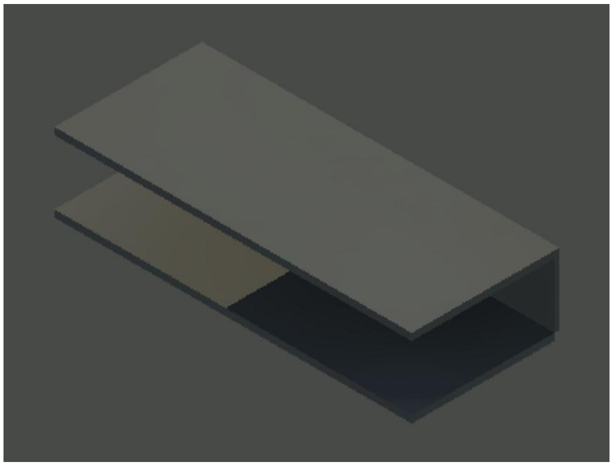
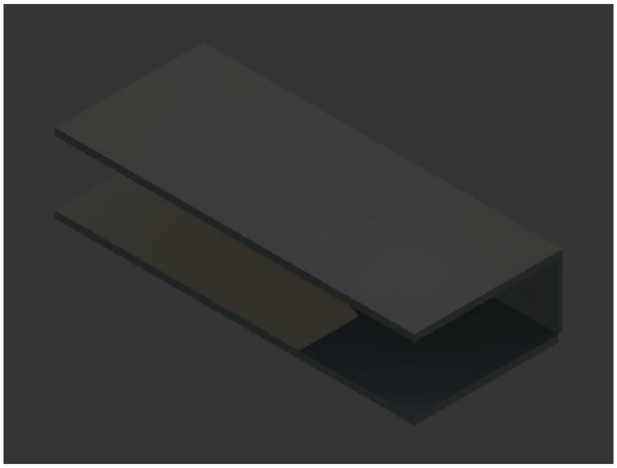


SLO Summer Solstice, 5:30 PM



SLO Summer Solstice, 6:00 PM



SLO Summer Solstice, 6:30 PM

Calculation Tools

The following are outputs from the Excel spreadsheet used to generate the values for total light penetration. Amount of depth assumes a 10 foot clear story height with regard to day and time is calculated and plotted with time intervals of 15 minutes. Values in the 4 left columns are from outside sources and describe the sun's position. The 4 columns on the right calculate the extent of natural light penetration on each face of the building. Each location and day of the year will yield a unique data set. The following is a sample output from the SLO Summer Solstice from sunrise to sunset.

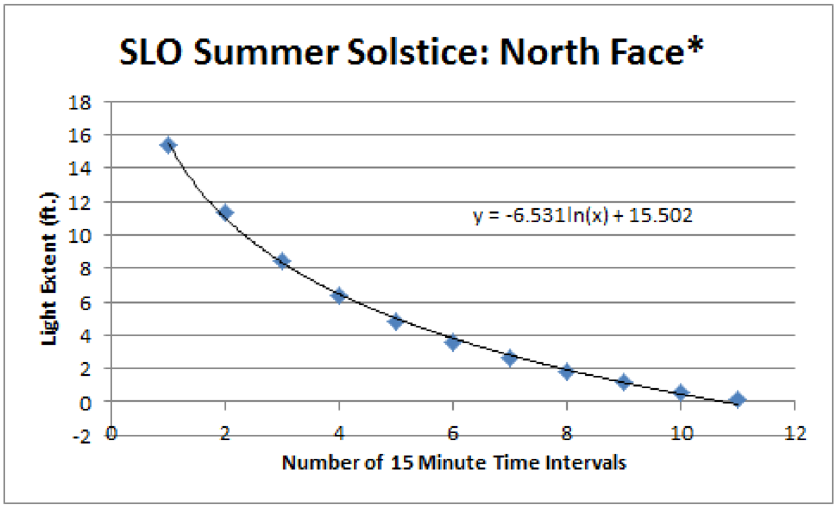
				Light Penetration			
Elevation	Azimuth	Elev (Rd)	Az (Rd)	North Light Penn	South Light Penn	East Light Penn	West Light Penn
-25.23	29.38	-0.44035	0.512777	0	0	0	0
-23.66	32.73	-0.41294	0.571246	0	0	0	0
-21.93	35.96	-0.38275	0.62762	0	0	0	0
-20.07	39.06	-0.35029	0.681725	0	0	0	0
-18.09	42.03	-0.31573	0.733561	0	0	0	0
-15.99	44.87	-0.27908	0.783129	0	0	0	0
-13.79	47.6	-0.24068	0.830776	0	0	0	0
-11.49	50.21	-0.20054	0.876329	0	0	0	0
-9.11	52.72	-0.159	0.920137	0	0	0	0
-6.64	55.14	-0.11589	0.962374	0	0	0	0
-3.95	57.46	-0.06894	1.002865	0	0	0	0
-0.74	59.7	-0.01292	1.041961	0	0	0	0
1.62	61.87	0.028274	1.079834	0	0	0	0
4.2	63.97	0.073304	1.116486	0	0	0	0
6.91	66.01	0.120602	1.152091	0	0	0	0
9.69	68.01	0.169122	1.186997	0	0	0	0
12.53	69.96	0.21869	1.221031	15.4189098	0	42.27132076	0
15.41	71.88	0.268955	1.254542	11.28339001	0	34.48085822	0
18.32	73.77	0.319744	1.287528	8.441229652	0	28.99825855	0
21.27	75.65	0.371231	1.32034	6.366732469	0	24.88695801	0
24.24	77.52	0.423067	1.352978	4.799452974	0	21.68477733	0

27.23	79.39	0.475253	1.385616	3.578035337	0	19.1006461	0
30.25	81.27	0.527962	1.418428	2.602587242	0	16.94864538	0
33.28	83.18	0.580845	1.451764	1.80918101	0	15.12734846	0
36.32	85.13	0.633903	1.485798	1.154863656	0	13.55430242	0
39.37	87.14	0.687136	1.520879	0.608089153	0	12.17203912	0
42.43	89.22	0.740543	1.557181	0.148926376	0	10.93888853	0
45.48	91.4	0.793775	1.59523	0	0.240262166	9.830913922	0
48.54	93.72	0.847182	1.635721	0	0.573208782	8.816211771	0
51.59	96.2	0.900415	1.679005	0	0.856299912	7.882380216	0
54.62	98.9	0.953298	1.726129	0	1.098657858	7.015886921	0
57.63	101.89	1.005832	1.778314	0	1.306016325	6.202860529	0
60.6	105.25	1.057669	1.836957	0	1.482106598	5.436306905	0
63.53	109.13	1.108807	1.904676	0	1.63177551	4.704321013	0
66.38	113.68	1.158549	1.984089	0	1.756343214	4.004860711	0
69.12	119.18	1.206371	2.080082	0	1.859845668	3.330532279	0
71.7	125.97	1.2514	2.198589	0	1.942518142	2.676593306	0
74.04	134.56	1.292241	2.348513	0	2.006672371	2.037731791	0
76.01	145.5	1.326624	2.539452	0	2.0532583	1.411165082	0
77.43	159.17	1.351407	2.778038	0	2.084039776	0.792901021	0
78.12	175.14	1.36345	3.056767	0	2.096133471	0.178227853	0
77.93	191.75	1.360134	3.346666	0	2.093542774	0	0.435457025
76.92	206.89	1.342506	3.610909	0	2.072192495	0	1.050826803
75.24	219.41	1.313185	3.829424		2.035600397	0	1.672652288
73.09	229.3	1.27566	4.002037		1.982482093	0	2.304841124
70.64	237.06	1.2329	4.137474		1.910629801	0	2.9488564
67.98	243.26	1.186474	4.245684		1.819726665	0	3.611821915
65.19	248.33	1.137779	4.334172	0	1.707018204	0	4.296066729
62.31	252.57	1.087514	4.408174	0	1.571970879	0	5.006936503
59.36	256.21	1.036027	4.471704	0	1.41194448	0	5.752683356
56.37	259.41	0.983841	4.527555	0	1.22243608	0	6.538251725
53.35	262.26	0.931132	4.577297	0	1.00205209	0	7.372430351

50.31	264.87	0.878074	4.62285	0	0.742103203	0	8.265980159
47.26	267.27	0.824842	4.664738	0	0.440152815	0	9.230193582
44.2	269.53	0.771435	4.704182	0	0.084380168	0	10.28289298
41.14	271.66	0.718028	4.741357	0.331573295	0	0	11.44228343
38.09	273.71	0.664795	4.777137	0.825495447	0	0	12.73133721
35.04	275.69	0.611563	4.811694	1.413813619	0	0	14.19003635
32	277.62	0.558505	4.845379	2.122040055	0	0	15.86204447
28.98	279.52	0.505796	4.87854	2.986159492	0	0	17.80670116
25.97	281.4	0.453262	4.911352	4.057914511	0	0	20.1253011
22.99	283.27	0.401251	4.94399	5.410191978	0	0	22.94065324
20.03	285.14	0.349589	4.976628	7.164076254	0	0	26.47802987
17.1	287.02	0.298451	5.00944	9.51445924	0	0	31.08189549
14.2	288.92	0.247837	5.042601	12.8140438	0	0	37.38451212
11.33	290.86	0.197746	5.07646	17.7717933	0	0	46.63778845
8.52	292.83	0.148702	5.110843	0	0	0	0
5.76	294.85	0.100531	5.146099	0	0	0	0
3.1	296.93	0.054105	5.182402	0	0	0	0
0.63	299.06	0.010996	5.219577	0	0	0	0

Cumulative Light Exposure Plots

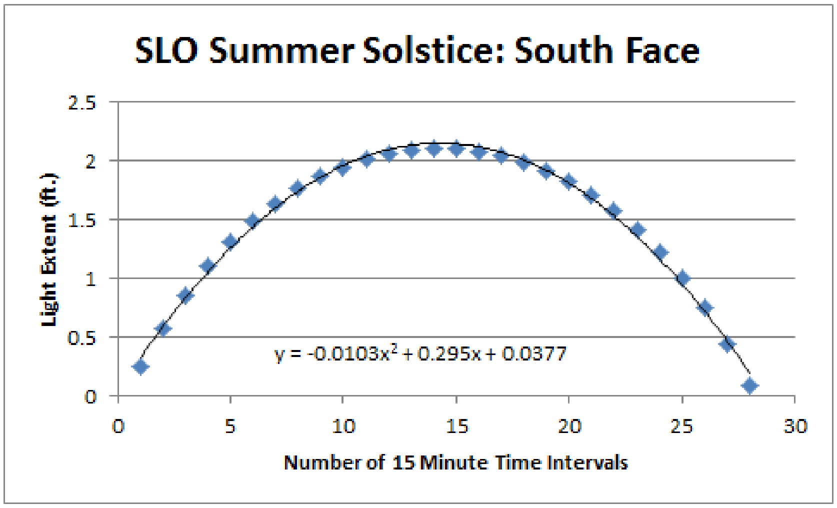
Fig. 1: SLO Summer Solstice: North Face



*This displays only the morning half of northern light exposure. The evening half is a mirror image of it.

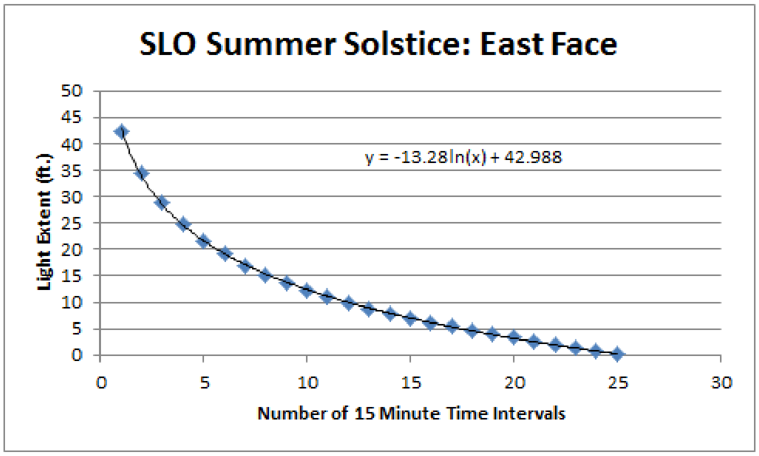
Optimal width of growth zone: 6.0 ft.

Fig. 2: SLO Summer Solstice: South Face



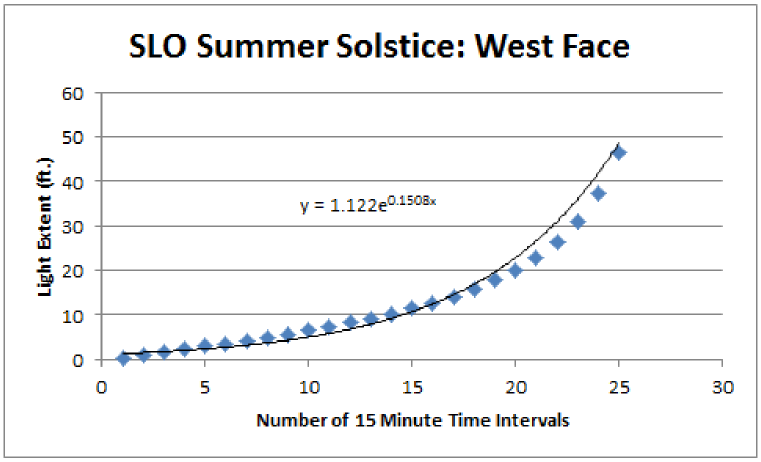
Optimal width of growth zone: 2.59 ft.

Fig. 3: SLO Summer Solstice: East Face



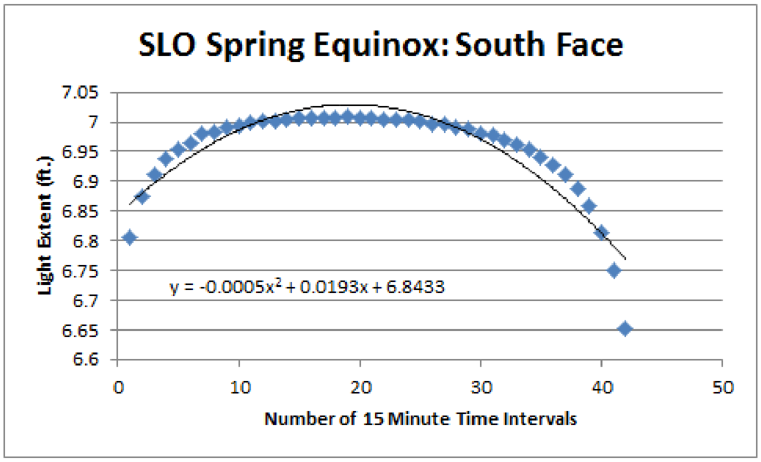
Optimal width of growth zone: 17.61 ft.

Fig. 4: SLO Summer Solstice: West Face



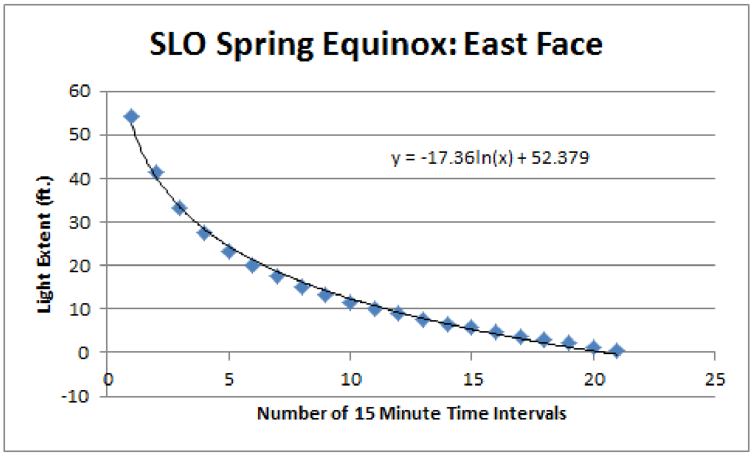
Optimal width of growth zone: 19.63 ft.

Fig. 5: SLO Spring Equinox: South Face



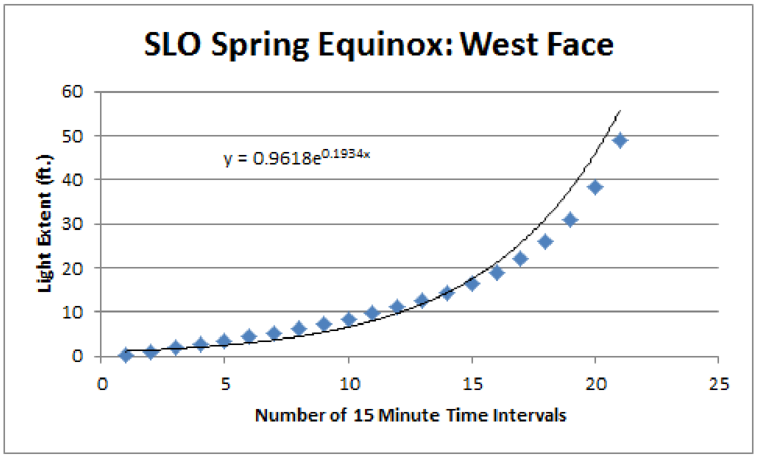
Optimal width of growth zone: 17.83 ft.

Fig. 6: SLO Spring Equinox: East Face



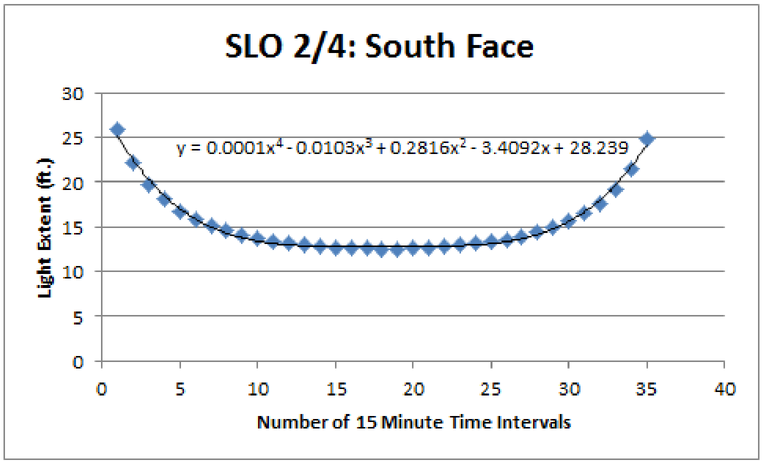
Optimal growth zone width: 17.81 ft.

Fig. 7: SLO Spring Equinox: West Face



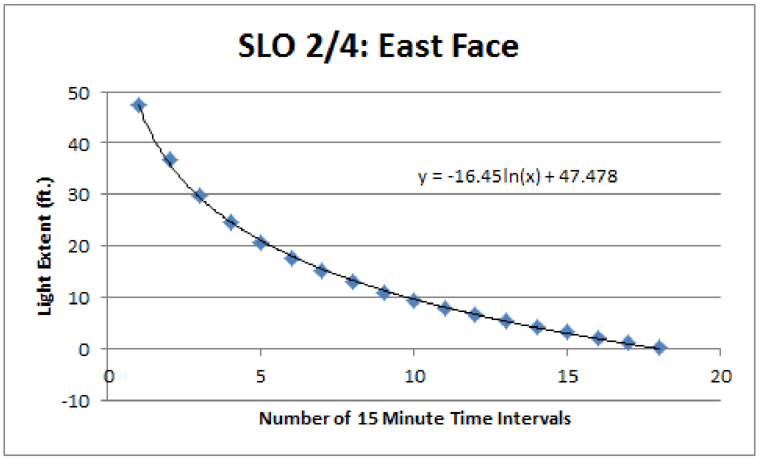
Optimal growth zone width: 17.67 ft.

Fig. 8: SLO Season 2 Beginning (2/4): South Face



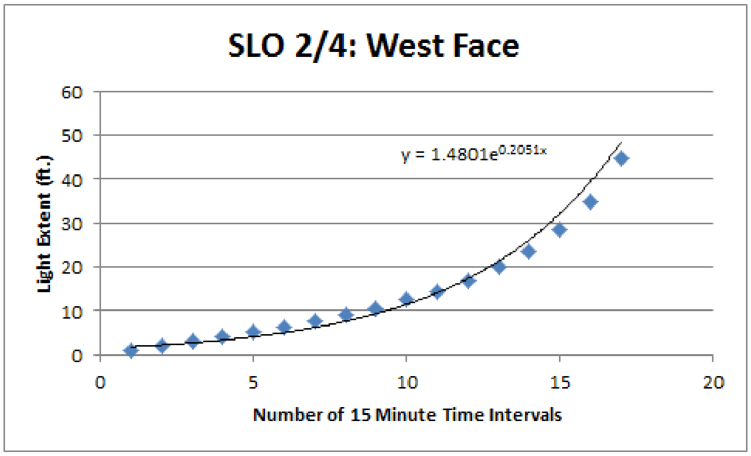
Optimal growth zone width: 31.19 ft.

Fig. 9: SLO Season 2 Beginning (2/4): East Face



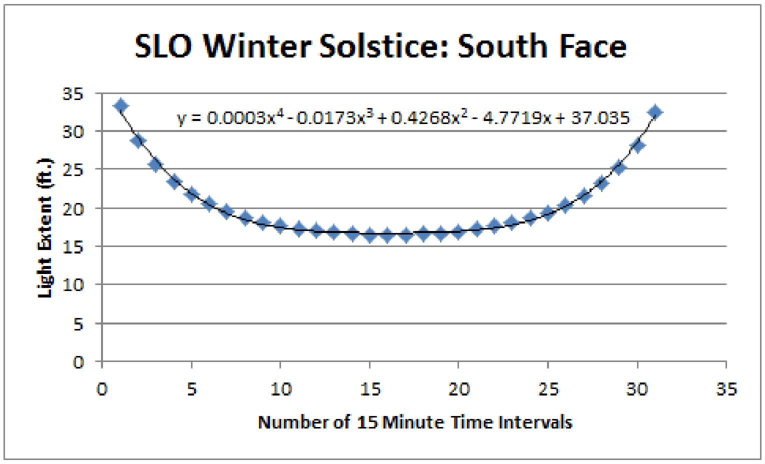
Optimal growth zone width: 14.43 ft.

Fig. 10: SLO Season 2 Beginning (2/4): West Face



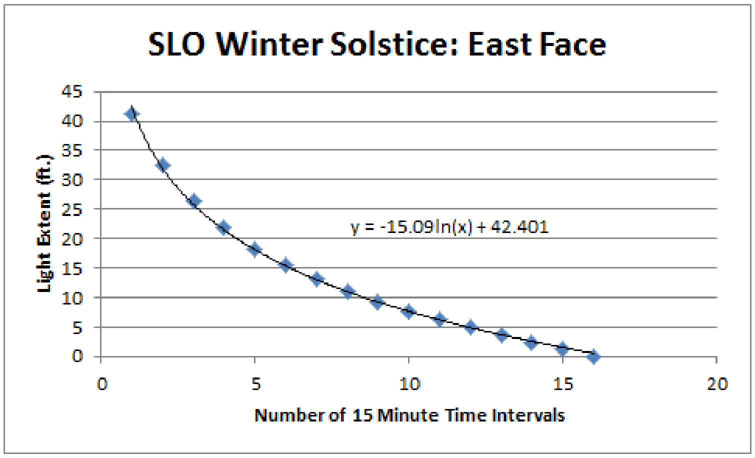
Optimal growth zone width: 14.29 ft.

Fig. 11: SLO Winter Solstice: South Face



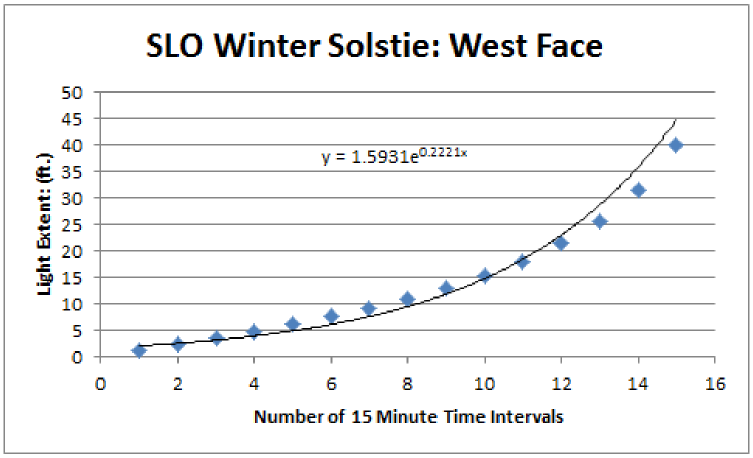
Optimal growth zone width: 61.43 ft.

Fig. 12: SLO Winter Solstice: East Face



Optimal growth zone width: 12.06 ft.

Fig. 13: SLO Winter Solstice: West Face



Optimal growth zone width: 12.09 ft.

References

http://www.gratingdepot.com/sites/default/files/content-pdfs/fiberglass-grating/deckload.pdf

http://www.gratingdepot.com/sites/default/files/content-pdfs/bar-grating/table/p15.pdf

http://www.esrl.noaa.gov/gmd/grad/solcalc/azel.html

http://aa.usno.navy.mil/data/docs/AltAz.php

http://keisan.casio.com/exec/system/1224682277

Works Cited

1. Bhanoo, S. N. (2014, December 3). “Vertical Farms Will Be Big, But For Whom?” Fast Company, Retrieved May 04 2016, from http://www.fastcompany.com/3039087/elasticity/vertical-farms- will-be-big-but-for- who

2. Despommier, D. (2013, June 2). “The Vertical Essay”. Retrieved May 04, 2016, from http://www.verticalfarm.com/?page_id=36

3. Despommier, D. (2014, April 2). Vertical Farming. Retrieved May 04, http://www.eoearth.org/view/article/51cbef257896bb431f69cb45/

4. Mougeot, L. J. (2000). Urban agriculture: definition, presence, potentials and risks. Growing cities, growing food: Urban agriculture on the policy agenda, 1-42

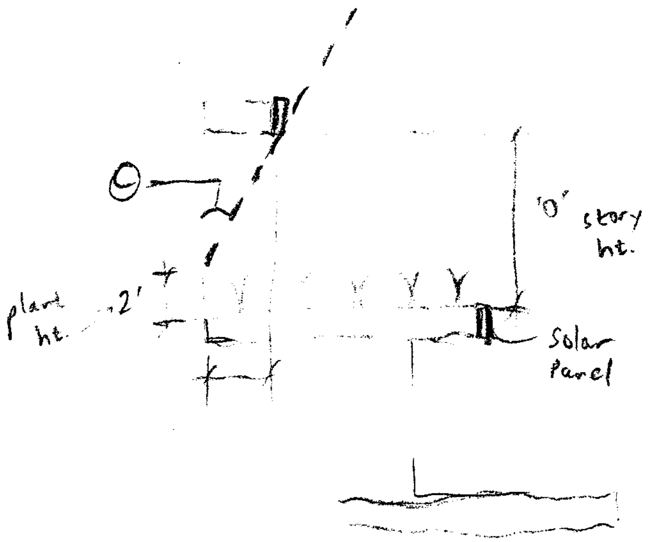
5. Shirk, A. (2015, May 22). “Farming in the Sky”. Retrieved May 06, 2016, from http://www.theatlantic.com/technology/archive/2015/05/farming-in-the-sky/392045/

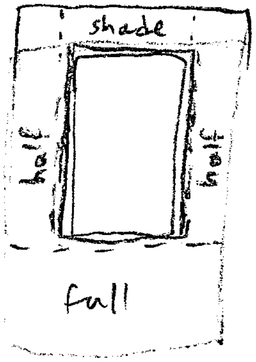
mr

Redenbaugh ch. 12

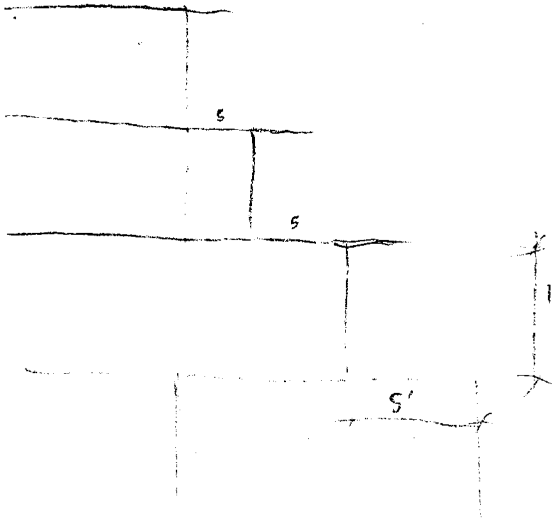
$\phi = 90 - (\text{LAT.} + 23)$

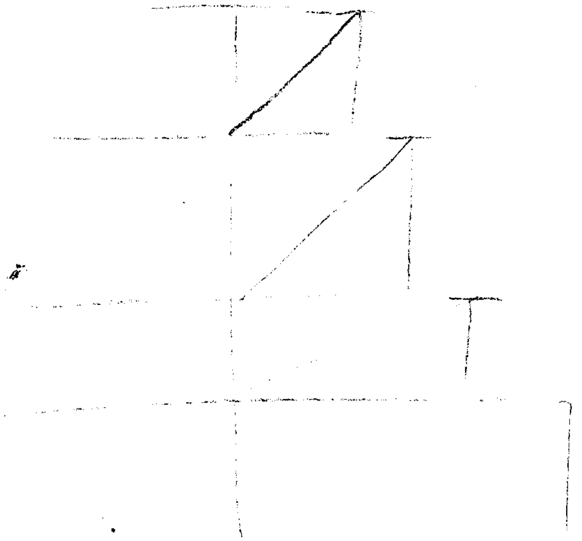
$\theta = \phi(\% \text{ exposure desired})$

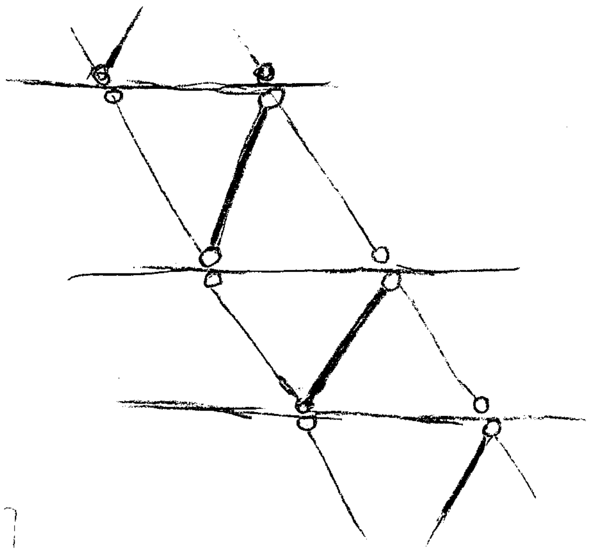


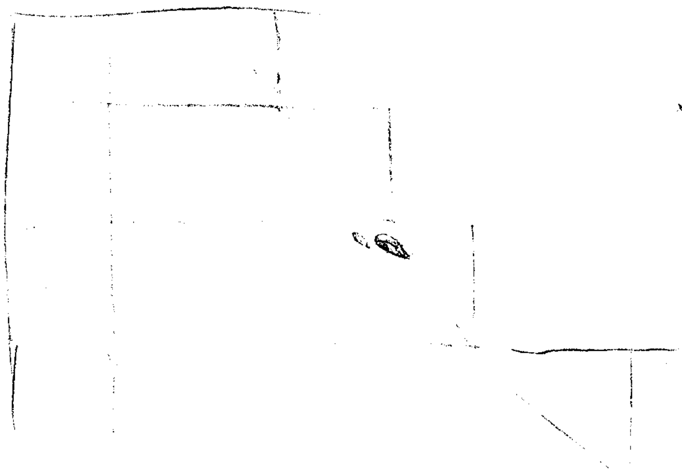


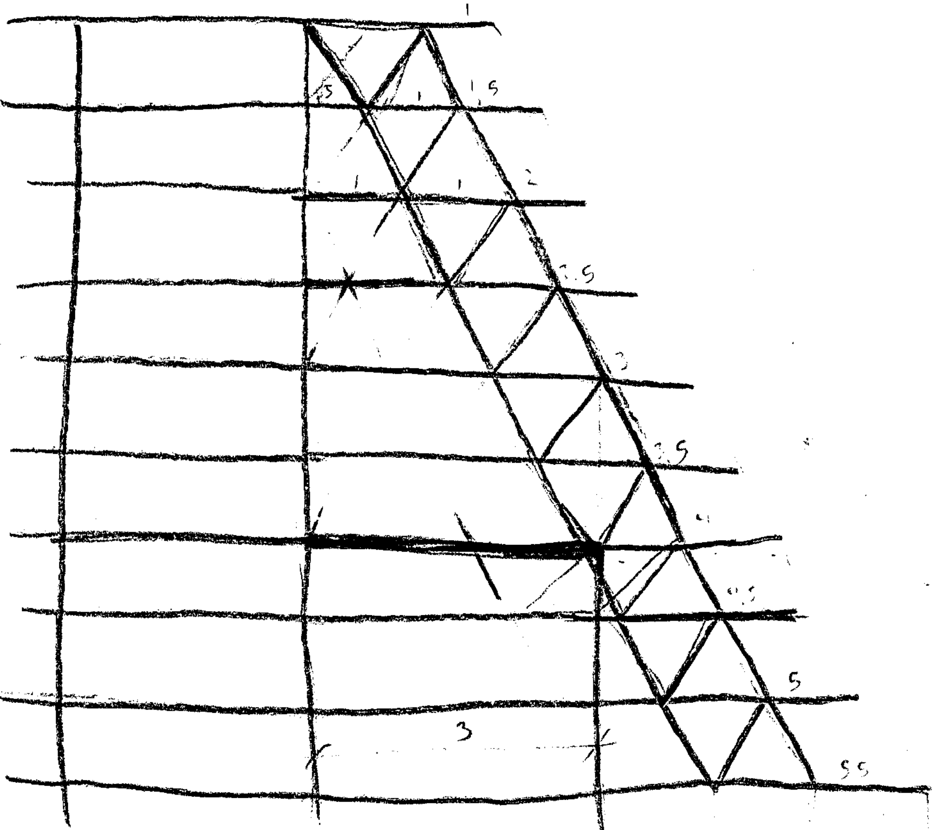
Tiered bldg. vs. straight vertical

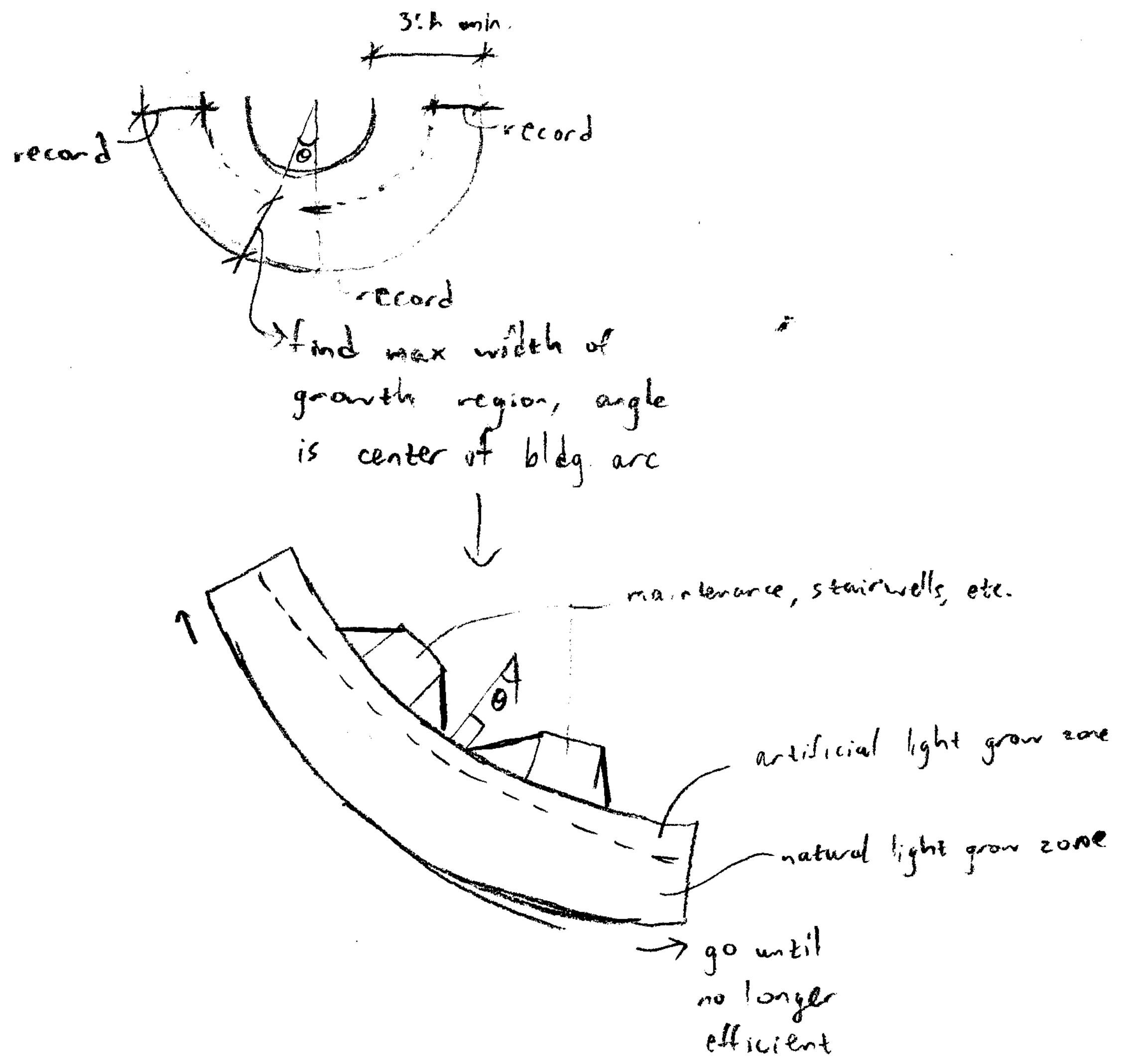

5
5
15'
5'

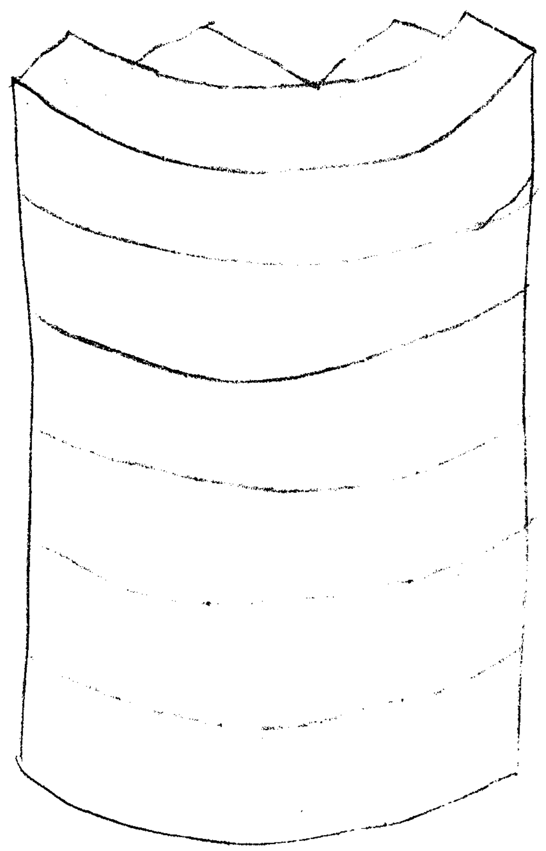




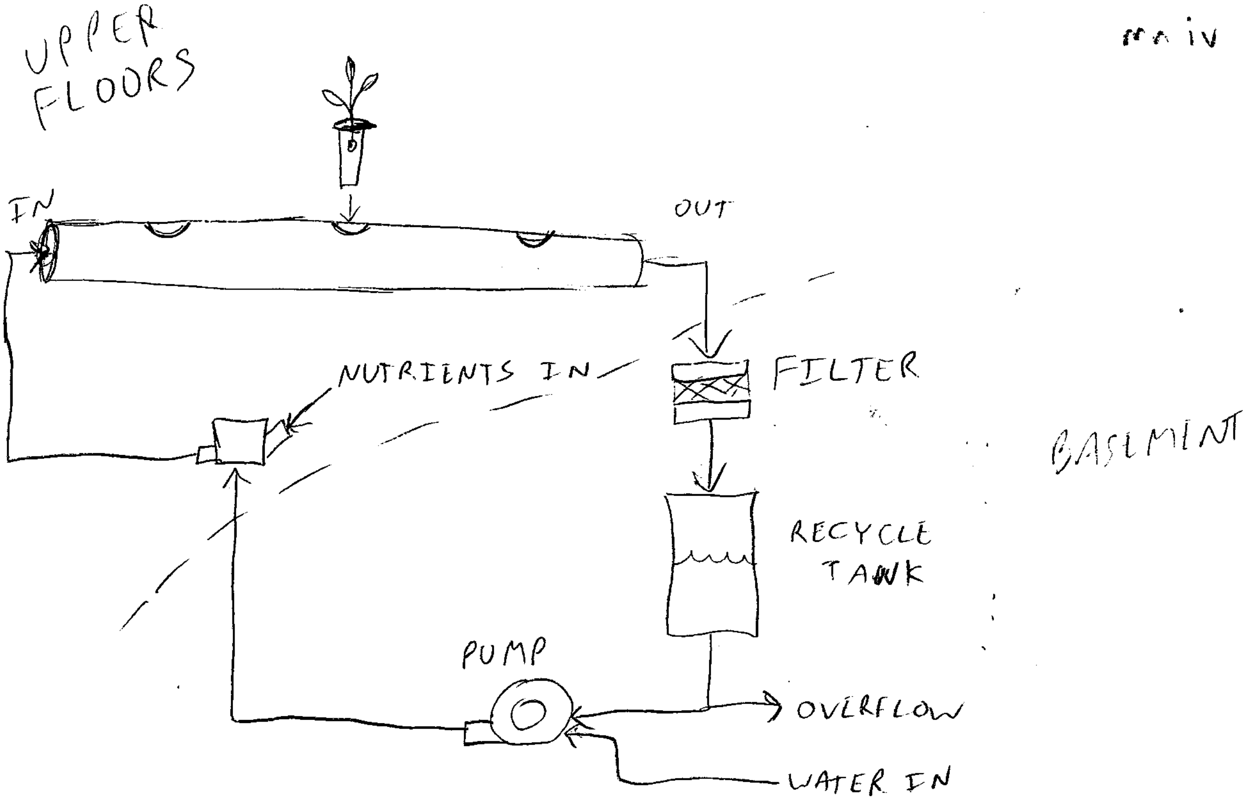


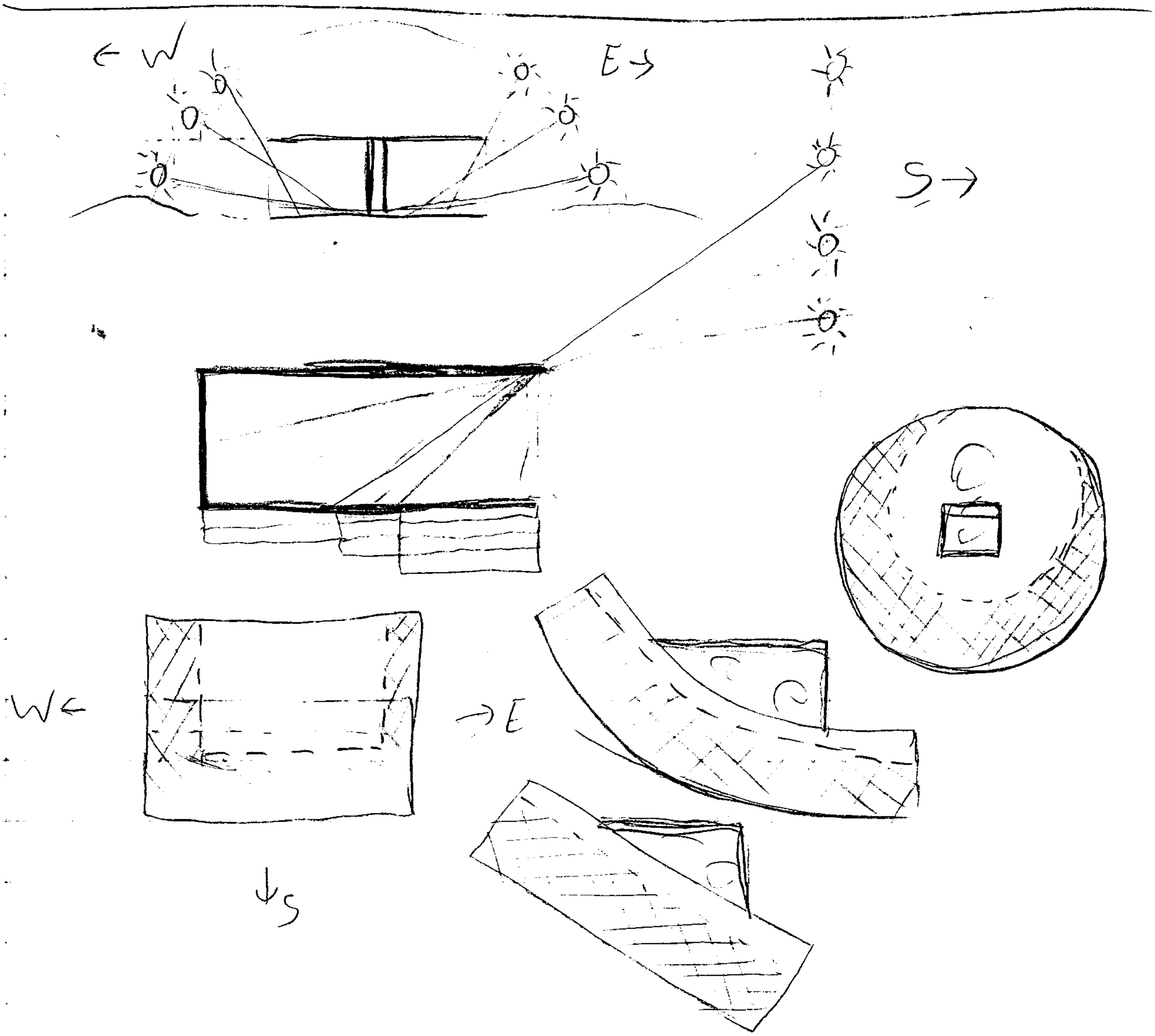

1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
3


3'.6 min.
record
record
record
θ
find max width of growth region, angle is center of bldg arc
maintenance, stairwells, etc.
θ
artificial light grow zone
natural light grow zone
go until no longer efficient

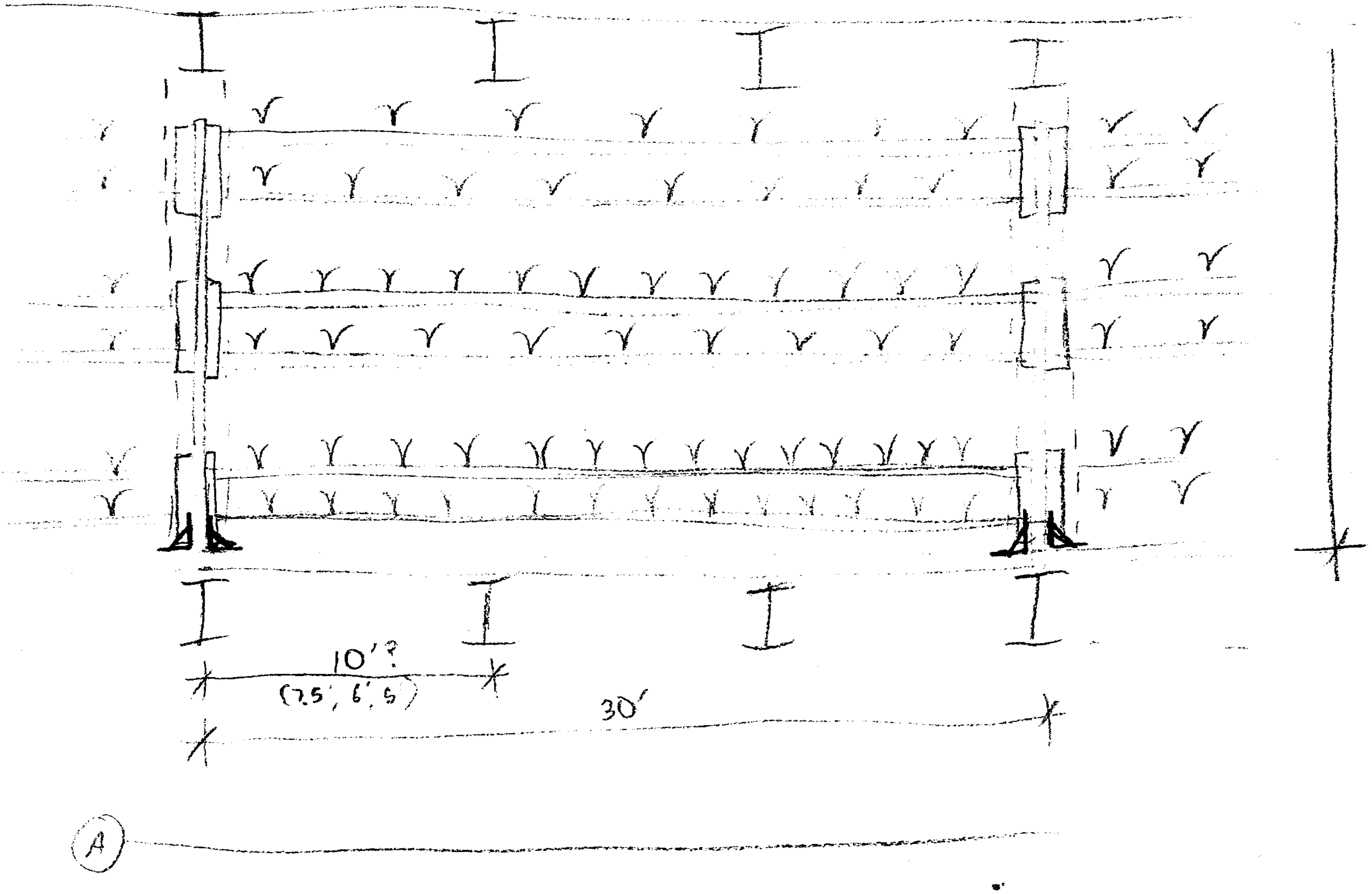


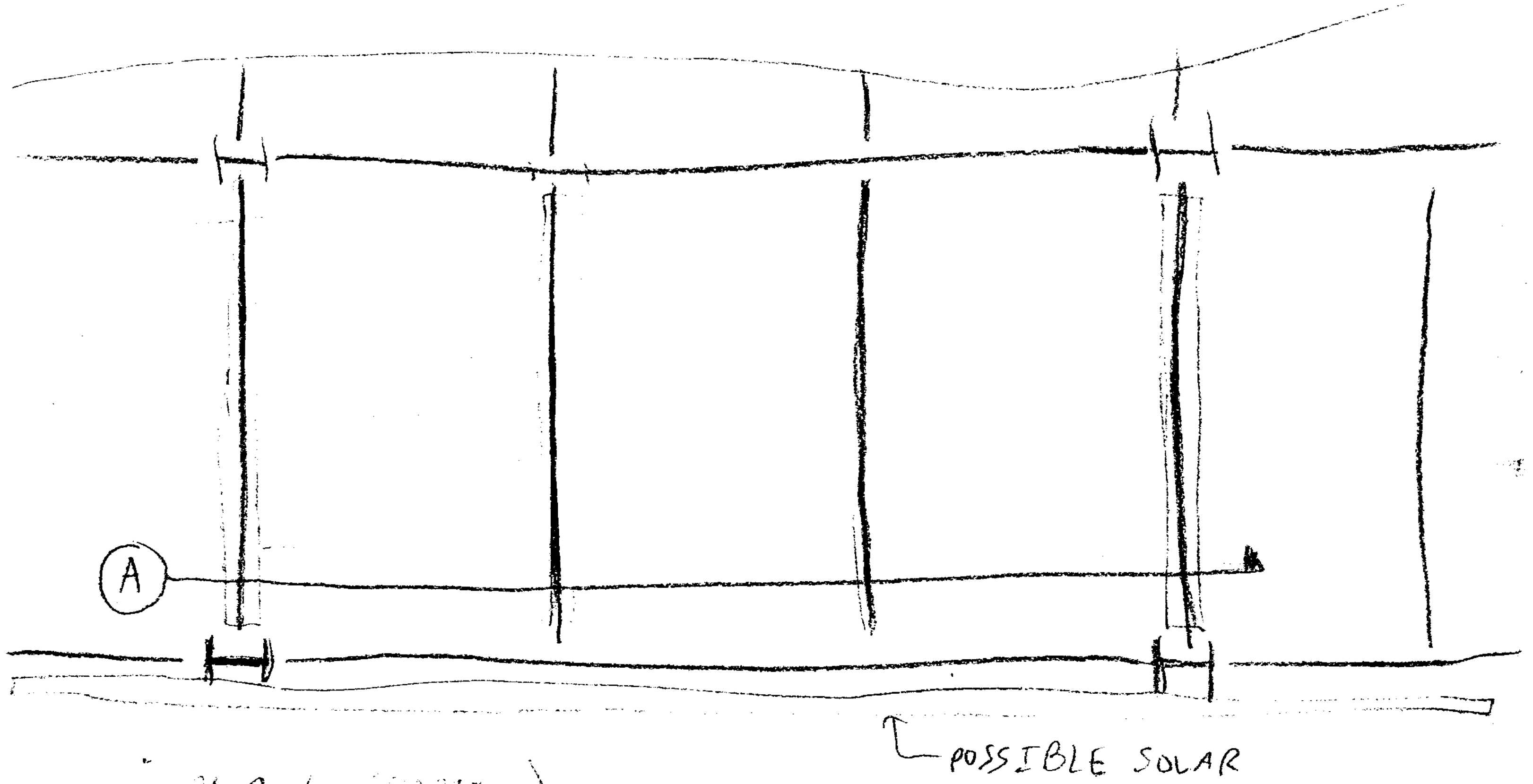
maiv


UPPER FLOORS
IN
OUT
FILTER
NUTRIENTS IN
BASEMENT
RECYCLE TANK
PUMP
OVERFLOW
WATER IN

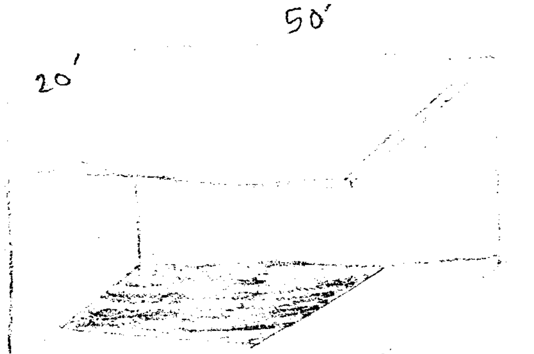

← W
E →
S →
W ←
→ E
↓ S

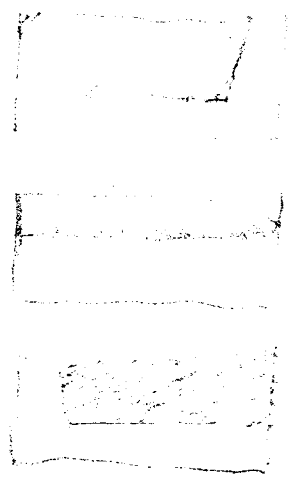
mnV

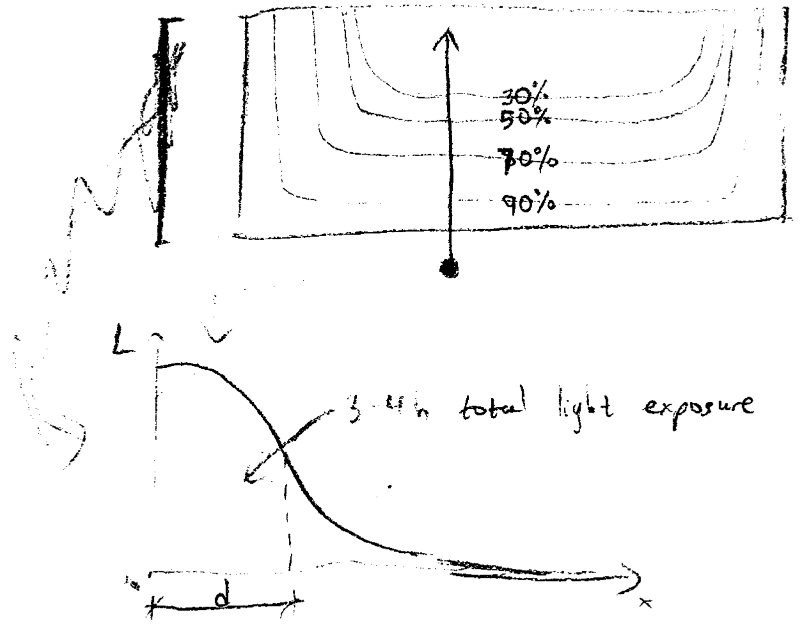


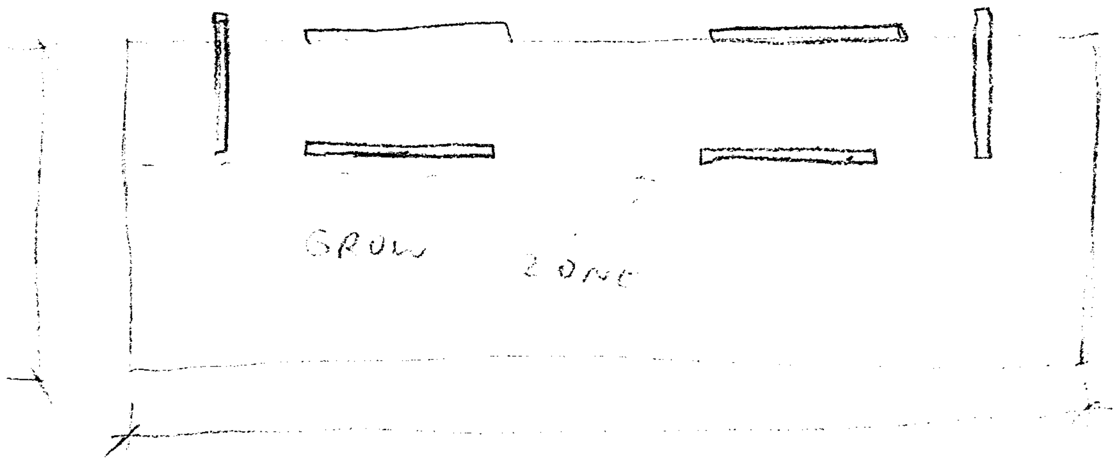


PLAN (PARTIAL)









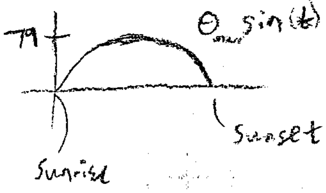
FULL PLAN

equinox

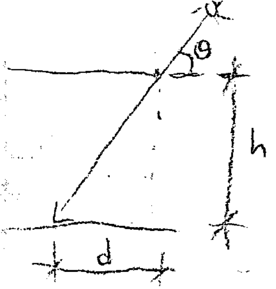
mAvi

SOUTH

WEST



$\Theta_{max} \sin(t) = f(x) = \Theta$



$\frac{h}{d} = \tan\theta$

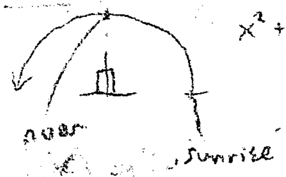
$d = \frac{h}{\tan\Theta}$

$\Theta = \Theta_{max} \sin t$

$d_i = \frac{h}{\tan(\Theta_{max} \sin t_i)}$

$d(t) = \frac{h}{\tan(\Theta_{max} \sin t)}$

$L = \int_{sunrise}^{sunset} d(t)$



$x^2 + y^2 = r^2$

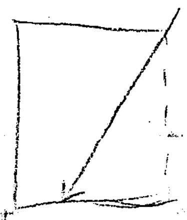
$y = \sqrt{r^2 - x^2}$

$\sim\infty$

$\Delta\Theta$ = constant

↳ Θ: 1st order

$\Theta = \frac{t}{(\text{hours/deg})}$



$d(t) = \frac{h}{\tan(t/\text{hours/deg})}$

$L = \int_{sunrise}^{noon} \frac{h}{\tan(t/\text{hours/deg})}$.

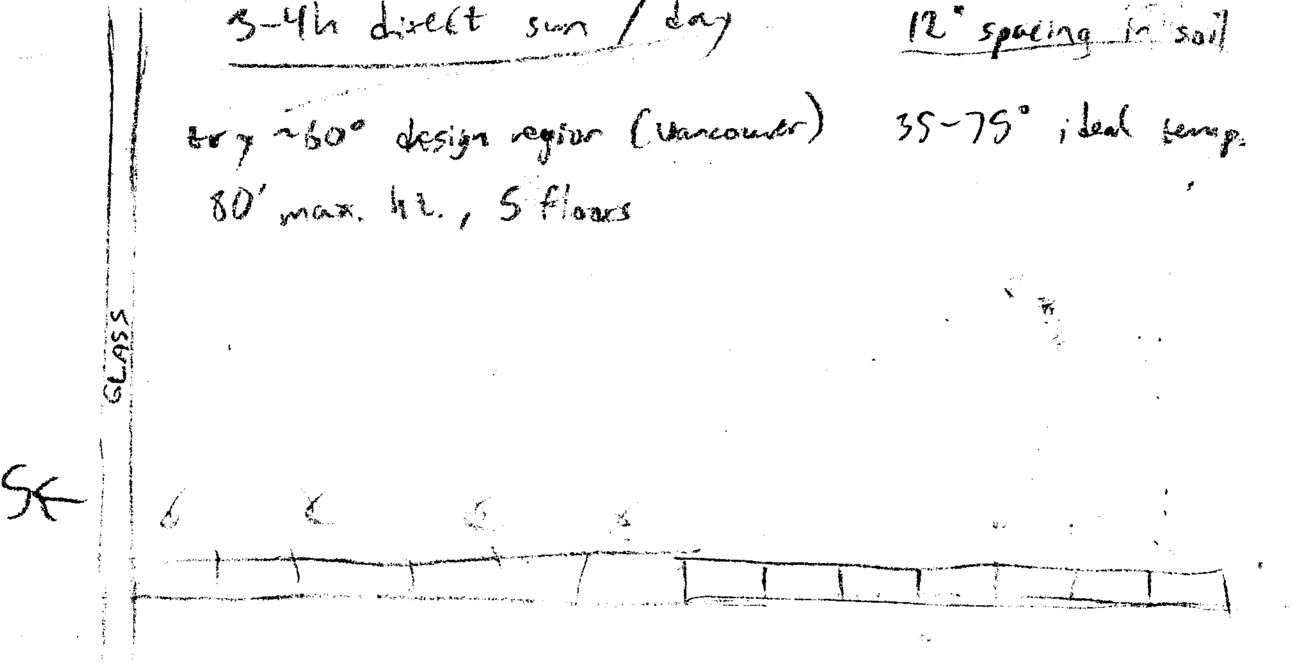
3-4h direct sun / day

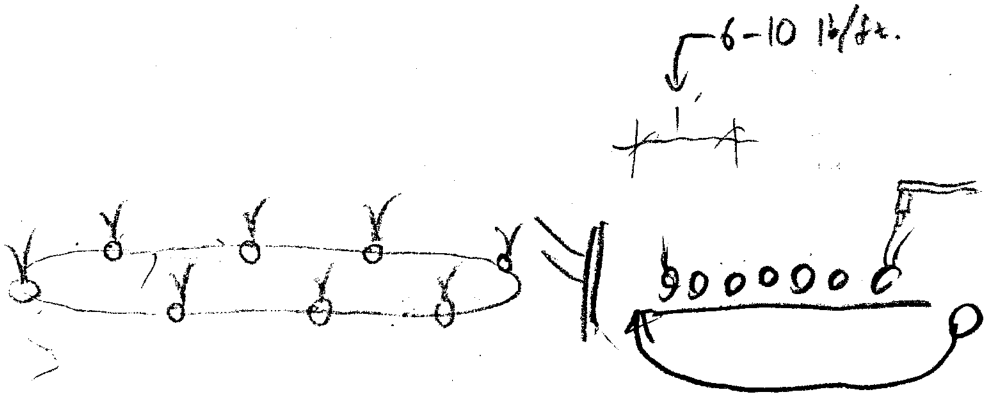
12" spacing in soil

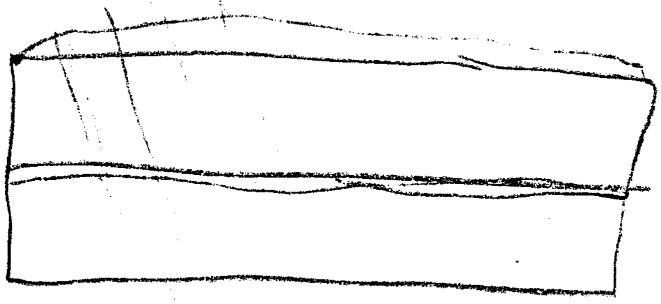
try ~60° design region (Vancouver)

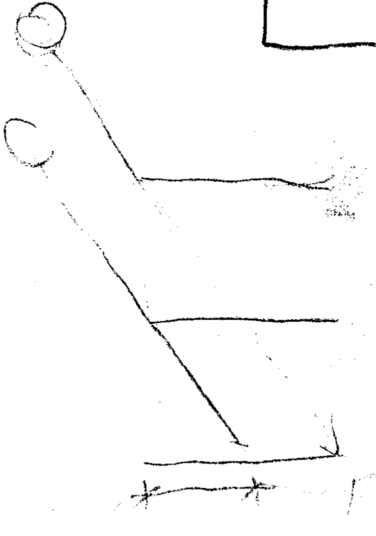
35-75° ideal temp.

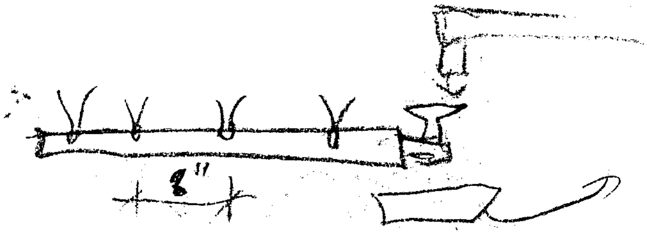
80' max. ht., 5 floors











Summer Solstice, SLO (6/21)

$A_z = 90°$ @ 9:36 AM → $\theta = 43.65°$
$A_z = 270°$ @ 4:33 PM → $\theta = 43.57°$
} → 67.5° @ 4 hr → 4.15'

~~→ 17.42' avg.~~

Winter Solstice, SLO (12/21)

$\theta > 10°$ @ 9:10 AM →
$\theta > 10°$ @ 4:51 PM →
} → 25.78° @ 4 hr → 20.7'

SOURCE: esrl.noaa.gov/gmd/grad/solcalc/azel.html

SLO SS

1hr → 49.725° → 8.47'
2hr → 55.795° → 6.80'
3hr → 61.75° → 5.37'
5hr → 72.655° → 3.12'
6hr → 2.90' (73.80°)
7hr → 2.09'
8hr → 2.09'
9hr → 2.09'

SLO WS

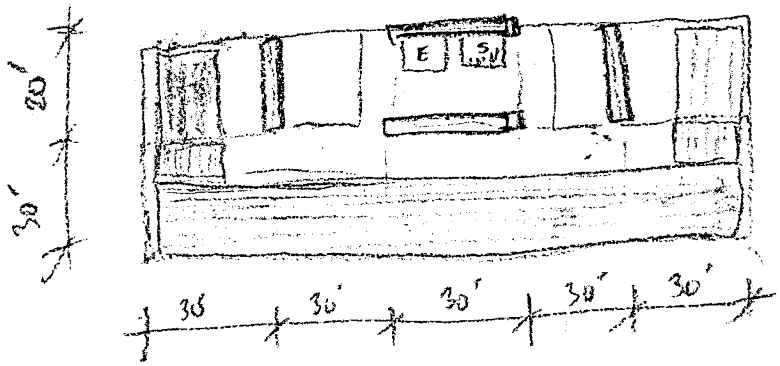
1hr → 14.595° → 38.4'
2hr → 18.82° → 29.34'
3hr → 21.575° → 24.05'
5hr → 28.32° → 18.56'
6hr → 30.14° → 17.23'
7hr → 31.14° → 16.55'
8hr → 31.30° → 16.45'
9hr → 16.45'

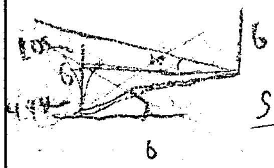
SLO SEQ (3/21)

1h → 15.945° → 35'
2h → 21.895° → 24.88'
3h → 27.705° → 19.04'
4h → 33.31° → 15.22'
5h → 38.62° → 12.52'
6h → 43.52° → 10.53'
7h → 47.85° → 9.05'
8h → 51.405° → 7.98'
9h → 53.955° → 7.28'

SPRING EQ

$A_z \geq 90°$ @ 7:13 AM, $\theta > 10°$ @ 7:56 AM
$A_z = 270°$ @ 7:05 PM $\theta > 10°$ @ 6:23 PM





Stagger ×4

EAST/WEST FACING | | | mn ix

Summer Solstice, SLO (6/21) $d = 10'/\tan\theta$

1hr → 25.38'
2h → 15.11'
3h → 9.77'
4h → 6.30'
5h → 3.79'

6h → 2.24'
7h → –
8h → –
9h →
θ>10° @ 6:46 AM, peak θ @ 1:05 PM

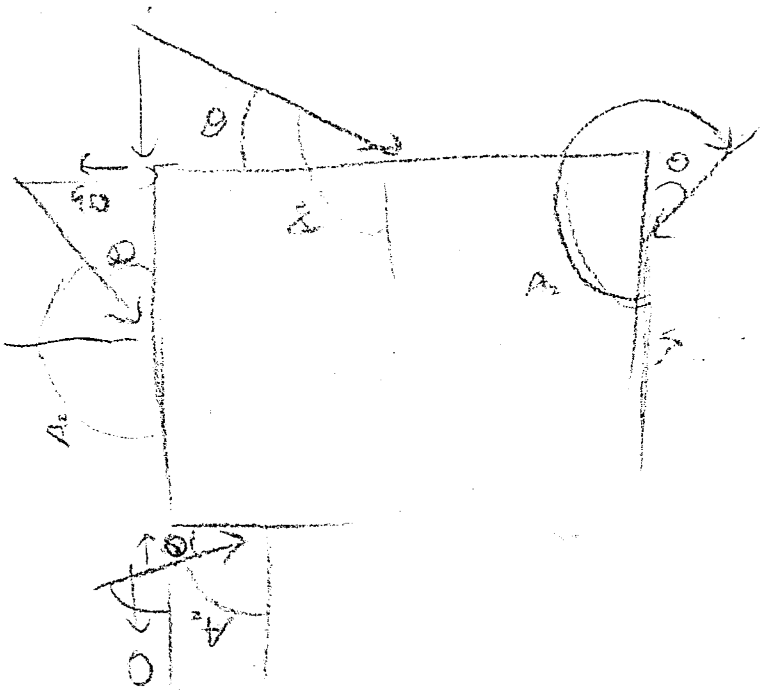
Spring Equinox, SLO (3/21)
θ>10 @ 7:57AM peak θ @ 1:10 PM
1h → 24.69'
2h → 15.15'
3h → 10.51'
4h → 7.99'
5h → 6.97'

Winter Solstice, SLO (12/21)
1h → 29.4'
2h → 20.79'
3h → 17.38'
4h → 16.54'
5h → –

θ>10° @ 9:11 AM
peak θ @ 1:01 PM

keisan.casio.com/exec/system/1224682277

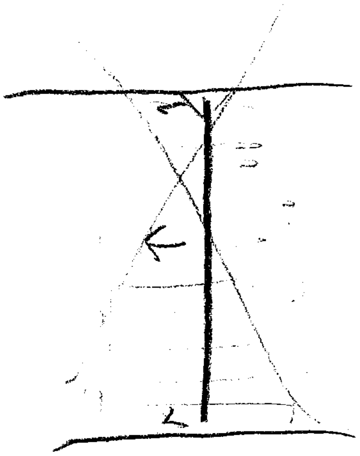
aa.usno.navy.mil/data/docs/AltAz.php

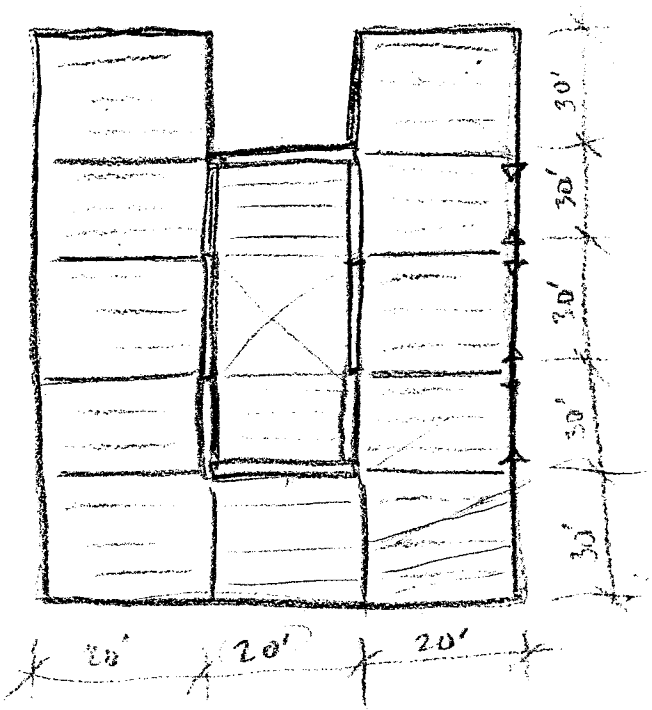


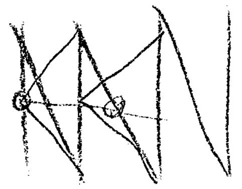
mn x

STRUCTURAL DESIGN QUESTIONS

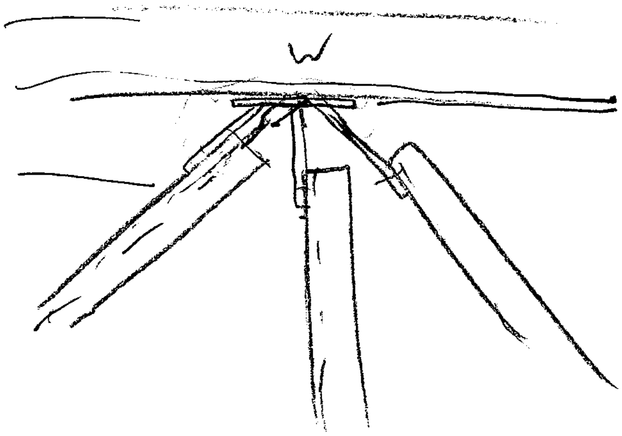
- Lateral truss Diaphragm
  - Btwn every beam or 1 brace / bay?
  - Lateral shear connections for beams?
- Beam Framing
  - Change direction on some bays?
  - Frame steel beam/girder into conc. wall?
  - Need conc. col's embedded into walls?
- Lateral System
  - Braced Frames vs. Conc. Shearwalls
  - Placement + redundancy

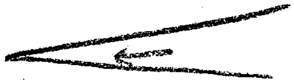


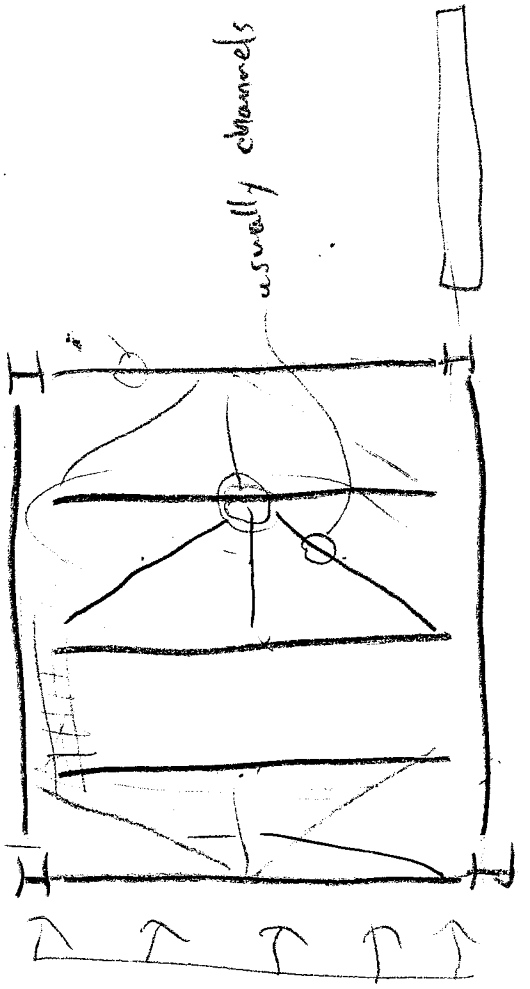




mr xi



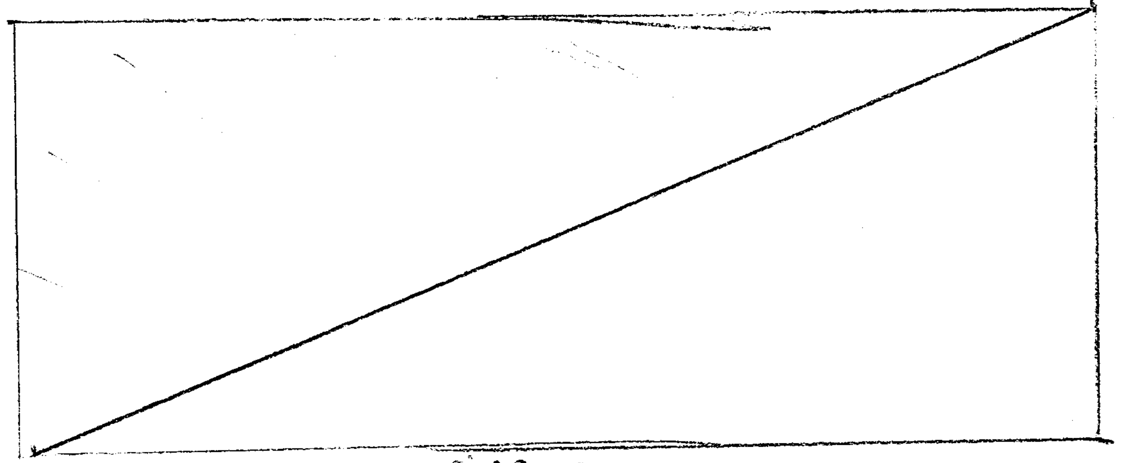




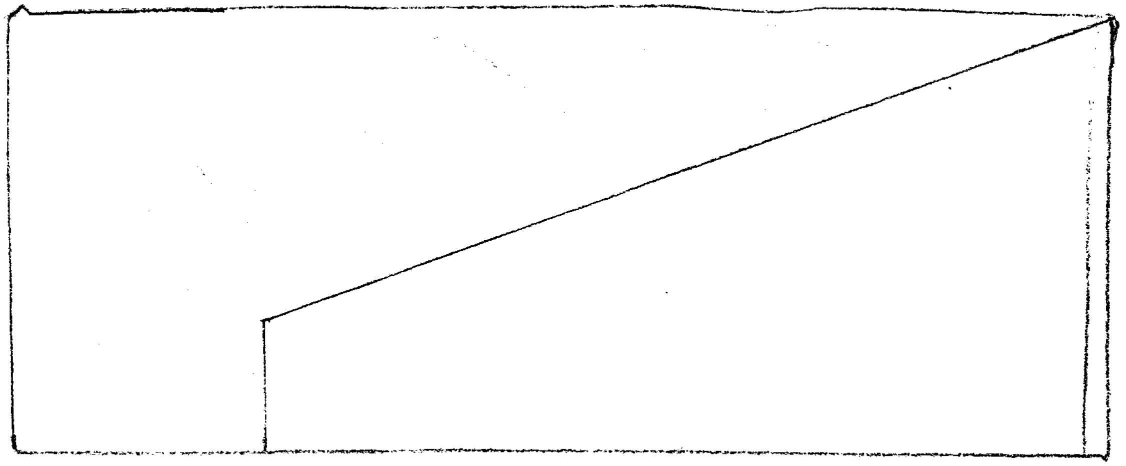
Shear tab on channel → W sec.

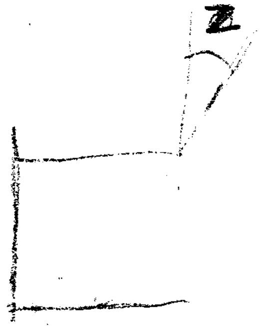
5 5/8 × 2 1/4

5:45 AM

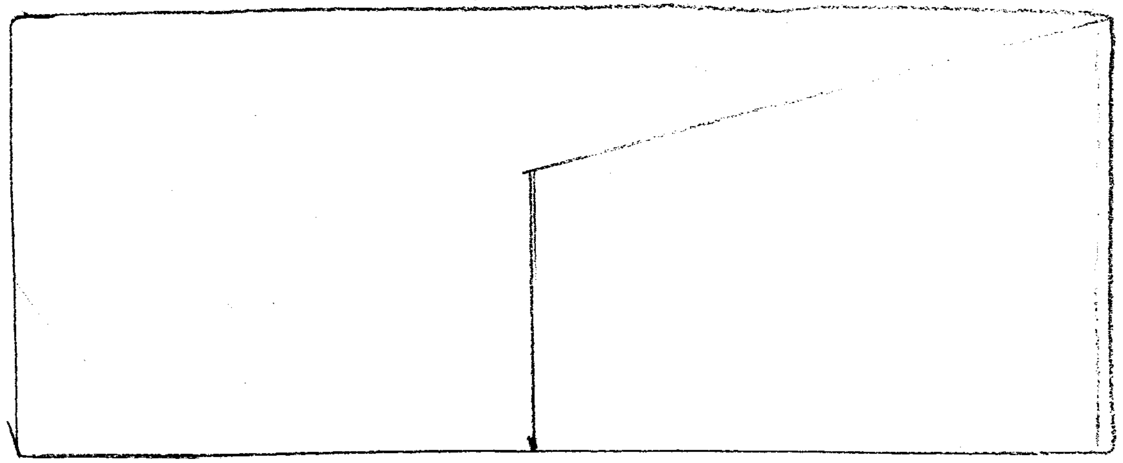


6:00 AM

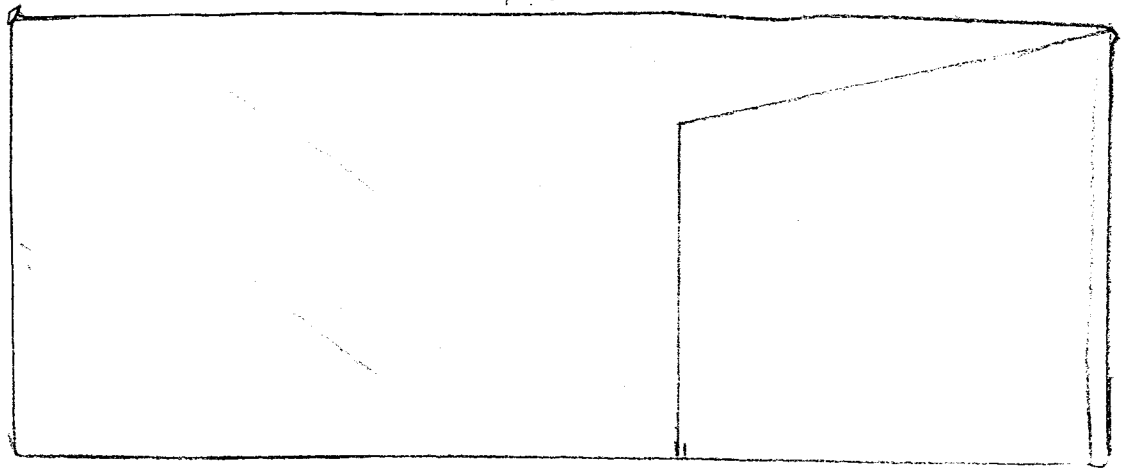




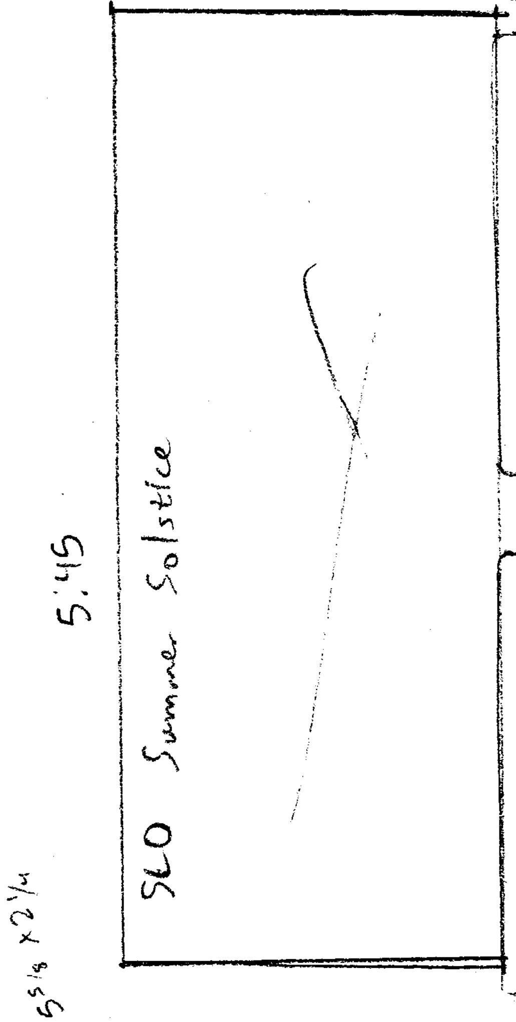
6:30 AM



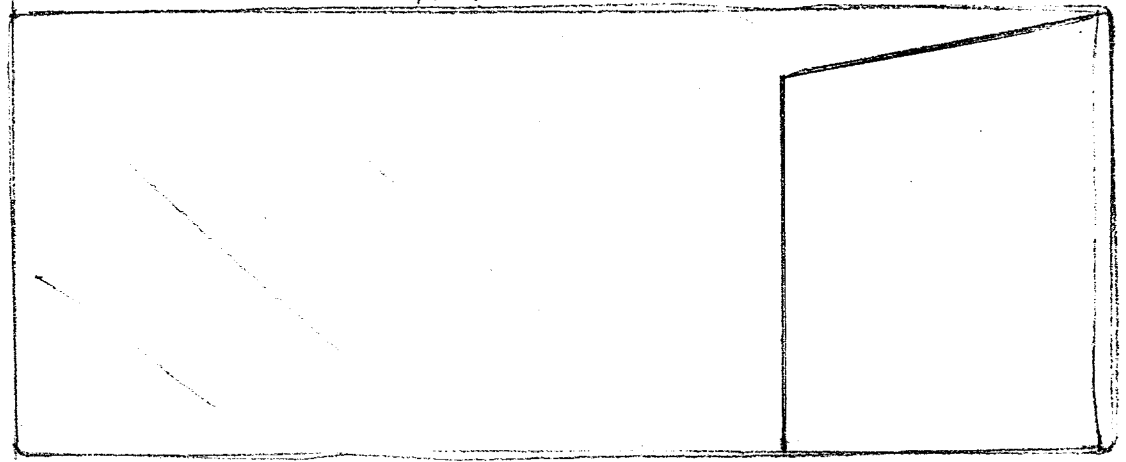
7:00 AM



SLO Summer Solstice



7:30



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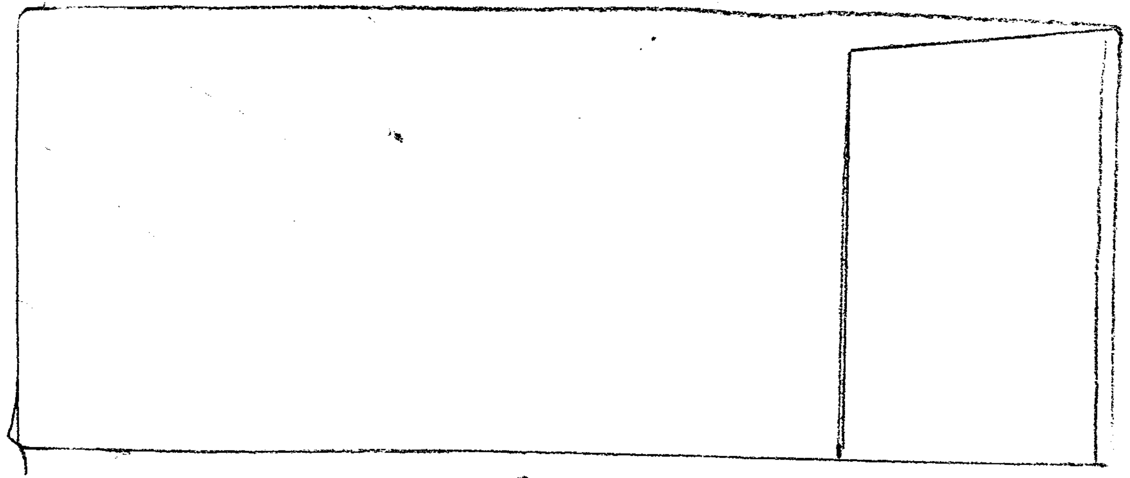

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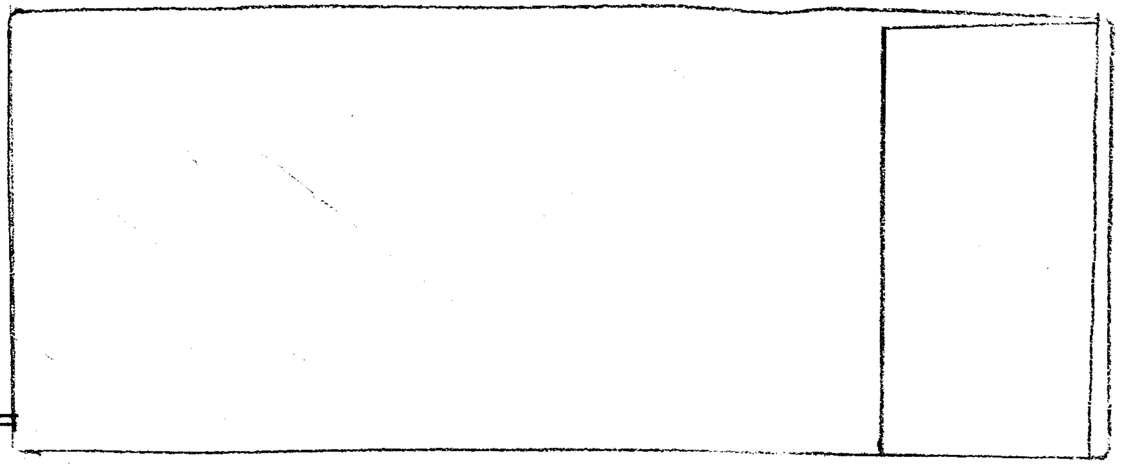

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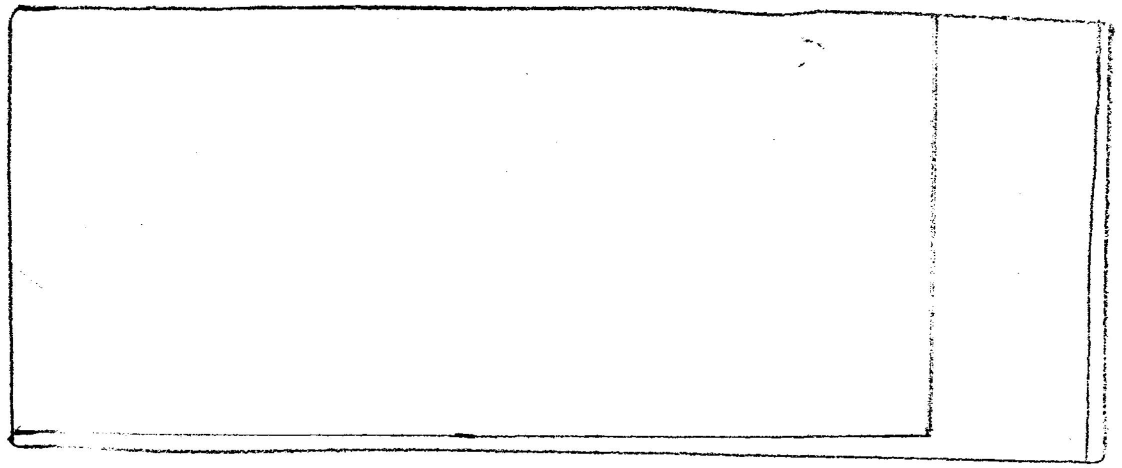

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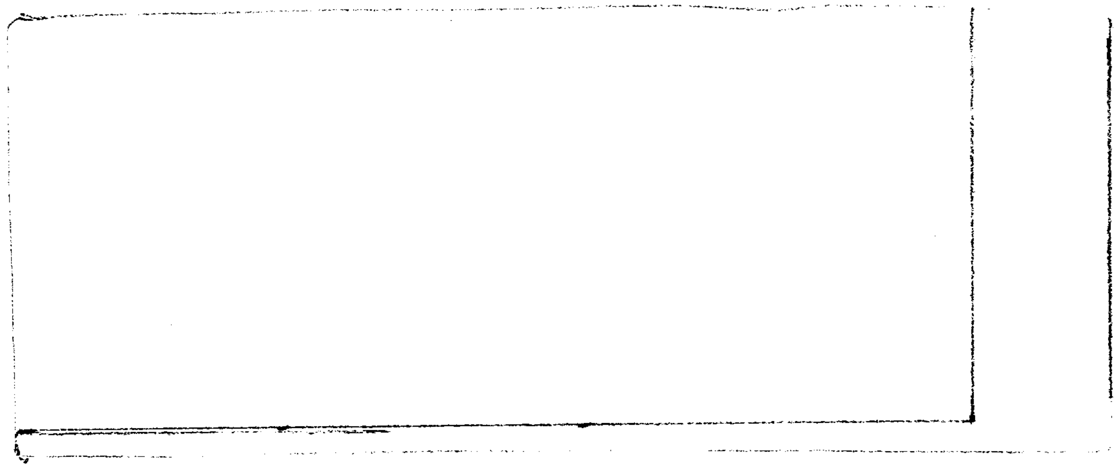
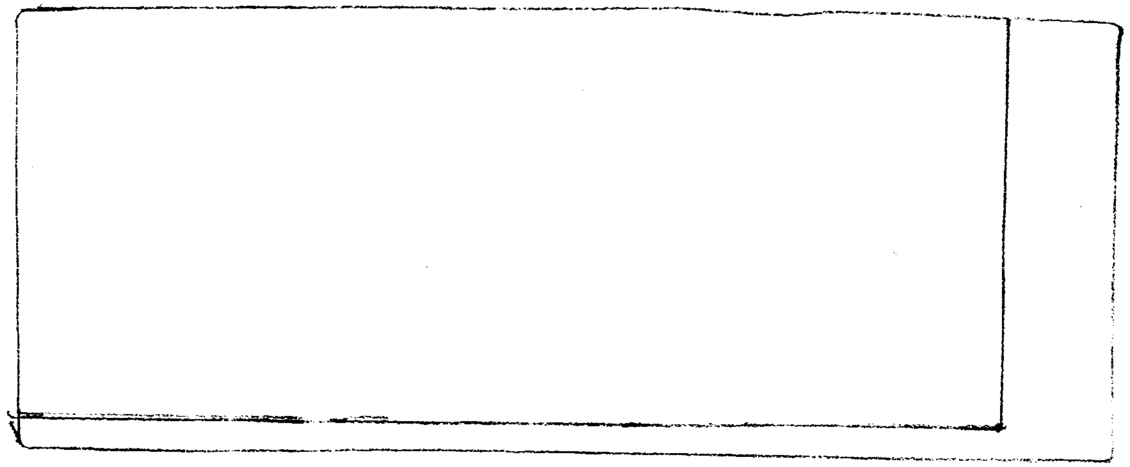
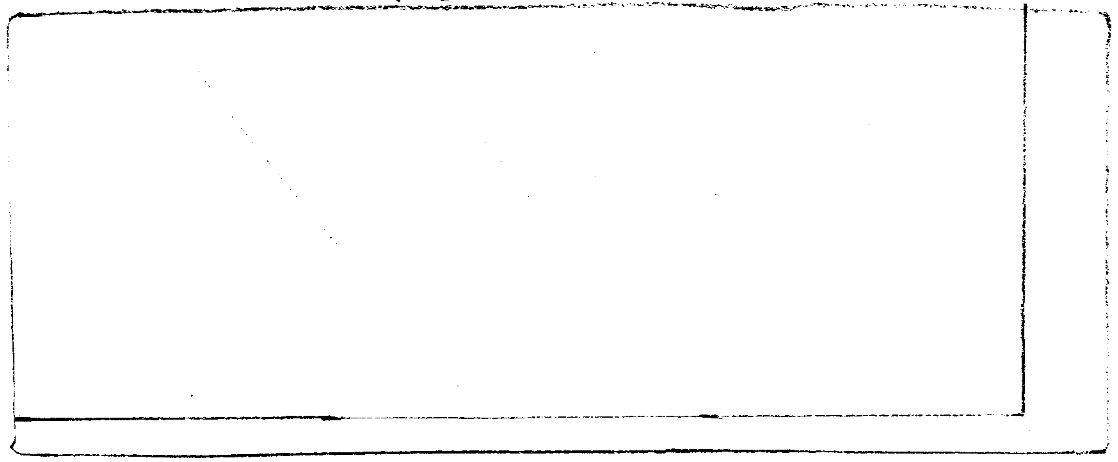


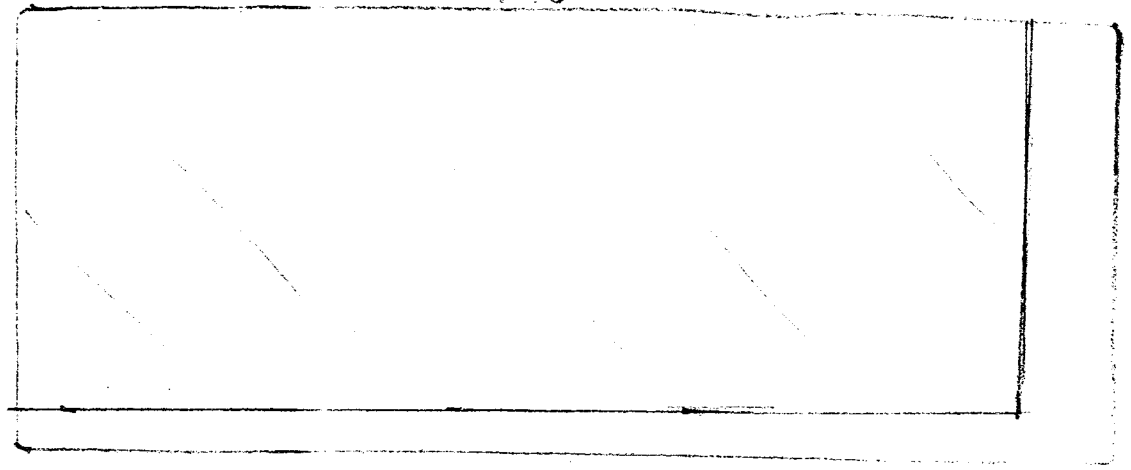
10:00



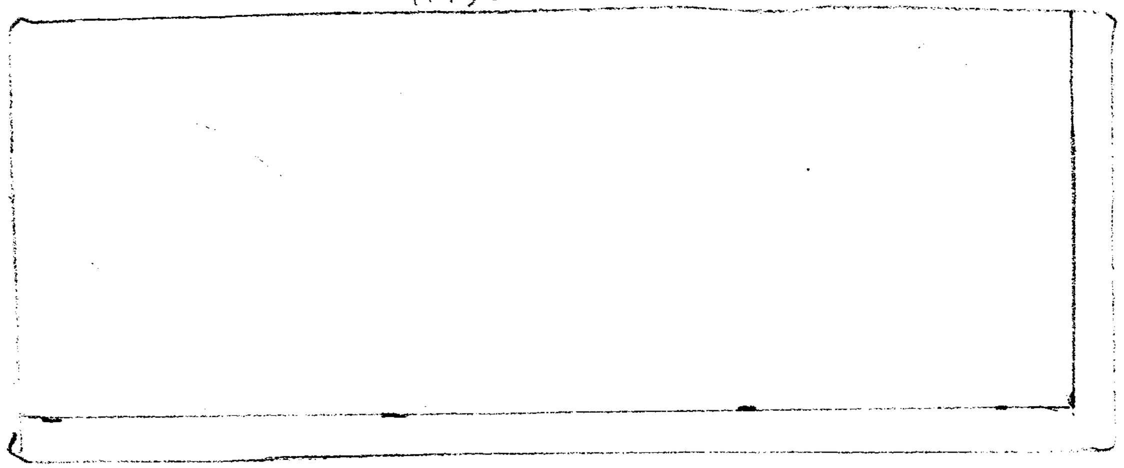
10 30



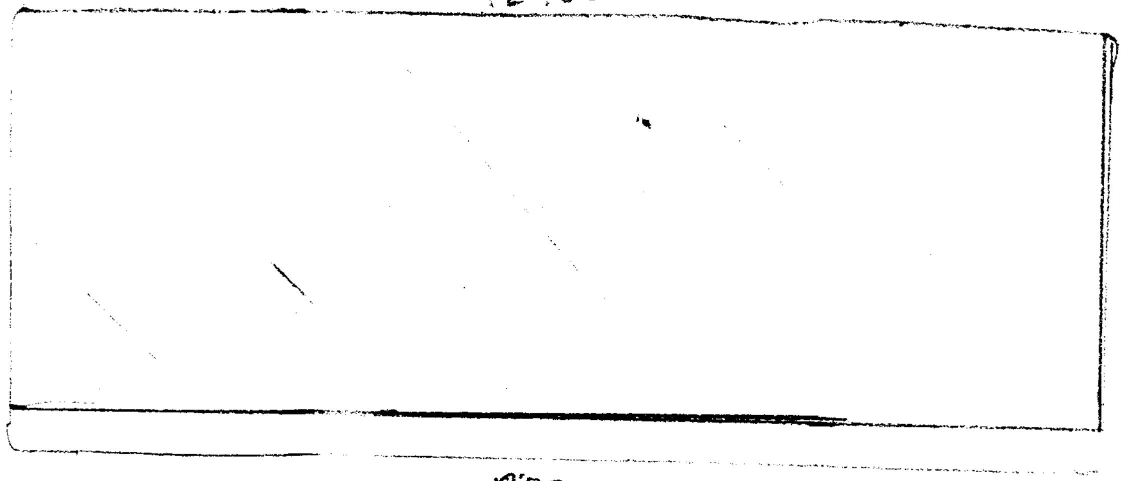
11:00



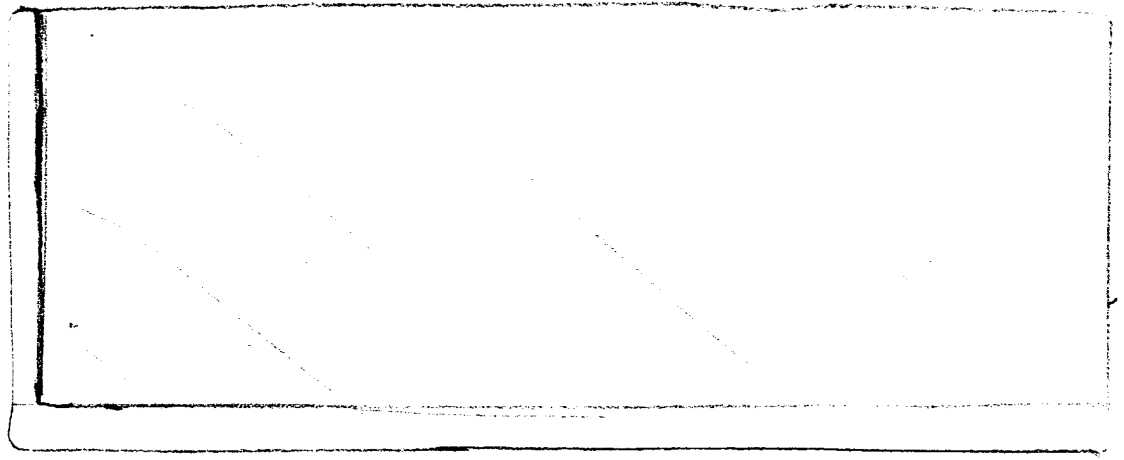
11:30



12:00



12:30



1300