University of Michigan

College of Engineering Honors Program

The Honors Porch

Heather Brouwer

Advisor: Jason McCormick

Date: *April 25, 2022*

Table of Contents

1.0 Introduction

The goal of this project was to create a plan for a community space for fellow Honors students through structural design. This project is important to the Honors community in terms of potential for community engagement, professionally as an effort to use existing structures to accomplish new uses, and personally to develop applied knowledge of structural design. The purpose of this document is to provide the methods, results, and conclusions from the project.

The Honors community is a rich and vibrant community that lacks a dedicated space of sufficient size for all in the program to enjoy. The Engineering Honors Program is an academic program that enables students to pursue interests outside of engineering and tie them all together through a Capstone project. The program spans several semesters, and in that time, many students across different disciplines get to engage in semester-long seminars, professional networking, and social events. However, the Honors administrative office has limited space and capacity to engage the whole community as it grows. This engagement is crucial to maintaining the atmosphere of the program such that creating a dedicated space would benefit the community.

Due to sustainability concerns and the ever-shrinking availability of land for development, renovation, repurposing, and adaptation of existing structures is crucial to the construction industry. Steel and concrete, the primary materials used for construction, are among the worlds most produced material ranging in the billions to tens of billions of tons per year, respectively. These materials also have a significant carbon footprint. As a result, continued new construction without regard for materials impact on the environment will continue to fuel climate change. Further, the number of undeveloped plots in the world and more specifically the US is shrinking severely. New developments are encroaching upon wildlife habitats and farming lands that are required for food production, which affects the way our society continues to evolve. Our society is also sufficiently built up that many of our buildings are left vacant and fall into disrepair. As a result, structural engineering needs to be able to adapt old buildings through deconstruction, renovation, or adaptation of existing buildings into new uses. These professional concerns yielded an interest in creating a space through the adaptation of an existing space.

Lastly, as a future structural engineer, it is important to become familiar with the standard practices of the industry and how various elements (materials, members, etc.) interact. Some standard practices involved with being a structural engineer include being able to work with codes and reference materials. It is also standard practice to be able to design and analyze parts of an entire system including horizontal members, vertical members, and connections. Lastly, it is important to get a level of understanding of how different materials (wood, steel, concrete, etc.) can interact. This can be particularly influential in creating a structure that optimizes sustainability, structural efficiency, and economic resources.

2.0 Problem Statement

The primary problem addressed by this project is how and where to create a space for honors students to enjoy together. The where was decided in the premise of the project as adjacent to the existing Honors Program office. This office is in the Chrysler Building on Bonisteel Ave. The way the building is laid out, the second story where the Honors Program office is located does not occupy the entire footprint of the building; therefore, there is a roof just outside the office to the south that is the location selected for the community space, henceforth referred to as the Honors Porch.

To create a community space here, there are several issues that need to be addressed in terms of the design and construction. An outdoor space was selected as it is simpler to construct and enables students to enjoy time outside and with each other. To build this space, analysis of building materials used and loading requirement of the new structure is required. This will enable the most sustainable, structurally efficient, and economically viable structure possible. These categories are the most important for the theoretical project if it were ever to be built. Further, information about the existing building such as the geometry and existing capacity of the roof structure is required. This information will enable the most structurally efficient design possible because partial or full support from the existing building would reduce the cost and increase the structural efficiency of the Honors Porch.

3.0 Methodology

This section details the methods and assumptions used to design the Honors Porch.

3.1 Loading Patterns

Prior to any structural design, the factored loads carried by the structure need to be determined. ASCE 7-16 was used to determine the design live and dead loads acting on the structure. The load resistance factor design (LRFD) methodology was used to ensure the safety of the structure. For live loading, the area was assumed to be a gathering area. The dead loads assumed wood decking on wood filler beams. This assumption will be checked in the design phase to ensure the selected wood filler beams have sufficient flexural capacity. Lastly, no snow loads were accounted for in this calculation because the snow that would have rested on the original roof will now rest on the deck. This assumption would need to be addressed should the structure ever be built.

3.2 Survey of Existing Conditions

Due to the nature of the project, an analysis of as many elements as possible that will interact with the new structure is crucial to creating the structure. Regardless of any physical interaction with the existing building, the geometry, access points, and any mechanical or plumbing elements located on the roof need to be identified. This includes the height of the roof from finish grade, any ledges or steps, any existing doorways, any roof top HVAC units, any roof drains, and any other elements located on the roof where the deck is to be placed. This information is critical to maintaining the operations of the existing building and enabling routine maintenance of the roof and HVAC system with the porch structure in place. This analysis also allows for an initial estimate of the shape and area of the porch that can be refined throughout the design process.

If the structure is transmitting any load through the existing structure, an analysis of the existing structural capacity is required. This can be accomplished using one of two methods: determining the current capacity through testing or obtaining the structural plans of the original building and any modifications. In this project the latter method was used. The original plans from Swanson Associates Inc. (1968) were provided by the College of Engineering Facilities Management and Coordination team. These plans are attached to the end of the report in Appendix A. These plans detailed the member sizes and construction specifications for the Chrysler building as well as other information including the soil bearing capacity, design loads for the original building, and material strength of members. This information allowed for an analysis of existing conditions without requiring invasive testing such that full or partial support of the new structure by the existing building can be evaluated for any given design. It also can inform the revision of the porch shape throughout the design phase to ensure that the new structure is compatible with the old.

3.3 Ideate Solutions Based on Existing Conditions

With all the information gathered in the previous two stages, a variety of solutions can be preliminarily evaluated based on the known and unknown information to select options for further design and analysis. Due to the analysis of existing plans, a wide variety of options were available for consideration. However, during the design process, these plans were not available until a later point, so some analysis of independent structures was performed initially.

Without the information from the existing plans or further testing, design options are limited. The primary option in this case is a system that is fully supported by its own foundation and does not impart any significant loads on the building. This would take the form of a cantilever in which a beam is supported only on one end by a column and foundation. Prior to any design, it is hypothesized that this will yield an inefficient structure due to the high moment generated by the cantilever.

Once the plans of the existing building were obtained, an option of a structure that was fully supported by the existing structure was possible. The information on the existing building enables a full theoretical capacity calculation for the structure that can be used to compare if there is any extra capacity in the structure that could bear the full load of the new structure. This would involve a full analysis of the roof, roof support members, bearing wall, foundations, and soil to ensure all had sufficient capacity. The fully supported options could take the form of connection directly to the roof, connection to the bearing wall, or connection to the beams supporting the roof among others. If there was not sufficient capacity, retrofit of any given member could be explored.

From this stage, the cantilever and existing-structure-supported by direct connection to the roof were selected due to the findings of the analysis phase to be conceptually designed and analyzed for viability.

3.4 Create Conceptual Design Options

For both design options, each type of member (filler beam, girder, and column) was designed to bear the capacity of the loading conditions determined in Section 3.1. This process started with determining the bay size of the deck structure, the size of the wood filler beams, the size of steel girders, and finally the size of the steel columns. In the case of the fully supported by direct connection to the roof option, an analysis of the existing structure was also required once the structure was designed to ensure the true dead load could be supported.

The bay sizing of the deck structure is dependent on the support system chosen. For the independently supported cantilever, foundations will be required, so the spacing of girders and columns is primarily dependent on what the designer believes is optimal. In this case, 30 ft bays were selected as this is a standard size in industry. For the fully dependent structure, the spacing is dependent on the spacing of the concrete beams below the roof which are 9 ft on center.

The required wood filler beam size was determined by an iterative method that compared the stress in dimensional lumber of a specific size to the capacity of wood in flexure. The capacity of dimensional lumber is assumed to be 1600 psi (The Engineering ToolBox, 2009). The maximum moment (M) for the filler beam was calculated assuming a simply supported beam. The section modulus (S_x) for the dimensional lumber was calculated based on the dimensions of the dimensional lumber being checked. The stress due to flexure (f_t) in the beam was calculated with Equation 1 using the maximum moment and section modulus from the previous two equations. A large size timber beginning with a 4" x 12" was used in the calculation and reduced as needed such that the beam was efficiently carrying the required capacity. The timber design also assumed a spacing of 16" on center for the filler beams. A sample calculation is given in Appendix B.

$$f_t = \frac{M}{S_x} \tag{1}$$

3.4.1 Steel Design of Cantilever Option

The design of the steel girder in the cantilever design assumed that deflections would govern. The maximum allowable deflection was calculated. Then, using the cantilever deflection equation, the required moment of inertia was calculated. This moment of inertia was used to identify an appropriate I-shape section from the AISC Steel Construction Manual. A full calculation is provided in Appendix C.

The maximum deflection was determined by calculating natural frequency of various deflections and compared to the expected frequencies that the structure may undergo. Equation 2 (AISC, 2016) demonstrates the relationship between deflection (δ) and natural frequency (*f*). The most likely frequency that the structure would experience is a walking human which is assumed to be 2Hz (Heinemann and Kasperski, 2017). Frequencies were calculated for several different displacements beginning at 1" and decreasing to 0.25". These frequencies were compared to the natural frequency of human walking, and the deflection was selected that was sufficiently far from 2Hz such that resonance would not occur.

$$f = \frac{1}{2\pi}\left(\frac{g}{\delta}\right)^{\frac{1}{2}} \tag{2}$$

The beam section was selected based on the deflection equation for cantilever beams and the AISC Steel Construction Manual. The deflection equation for cantilever beams (AISC Table 23-2 case 19) was rearranged to isolate the moment of inertia (I) and is given in Equation 3. This represents the required moment of inertia for the selected section. The required moment was then compared to the moments of inertia given in Table 3-3 of the AISC steel construction manual such that the selected section has a sufficient moment of inertia and is the lightest section in the grouping as denoted by the bolded entries.

$$I \geq \frac{wL^4}{8E\Delta_{max}} \tag{3}$$

The column was selected assuming beam-column design was applicable. Beam-column design applies to members undergoing both axial load and bending. The maximum moment and axial load exerted by the cantilever beam were calculated. The maximum moment was calculated using the effective length method and assuming a fix-free connection for the effective length factor of 1.2. The design also assumes an unbraced length of 20 ft. Using Table 6-2 of the Steel Construction Manual, a section with sufficient moment and axial capacity was selected. Lastly, due to the combined axial and bending loading on the member, the interaction of the forces was checked using AISC Equations H1-1a and H1-1b to ensure the result is less than 1.0.

$$if\,\frac{P_r}{P_c} \geq 0.2,\ \frac{P_r}{P_c} + \left(\frac{8}{9}\frac{M_{rx}}{M_{cx}}\right) \leq 1.0 \tag{AISC H1-1a}$$

$$if\,\frac{P_r}{P_c} < 0.2,\ \frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}}\right) \leq 1.0 \tag{AISC H1-1b}$$

3.4.2 Steel Design of Existing-Structure-Supported Option

The design of a structure fully supported by the existing structure assumed that the existing capacity of the structure would govern the design. First, an analysis of elements in the existing structure that would bear the load of the new structure were considered for their capacity. The analysis of the existing structure included concrete beams beneath the roof slab, the basement wall laterally and in bearing, and the bearing capacity of the soil. A full sample calculation of the existing structural capacity is given in Appendix D. Then, the beam and supporting members were designed to accommodate the load of the new structure. The full member design calculation is given in Appendix E.

3.4.2.1 Existing Structural Capacity Analysis

The capacity of the concrete beam was calculated using the design information provided by the structural plans. The effective depth of steel from the compression edge were calculated by finding the centroid of the steel area within the section of the beam. Then, the design moment was found using the design loads on the original structure. The resistance factor of concrete was calculated and compared to the design guide provided by John McCarthy for concrete design to determine the required amount of steel for the design load. The required amount of steel was then compared to the provided amount to determine the flexural capacity available.

The shear capacity of the beam was also determined through analysis of the existing structure detailed in the structural plans. The shear capacity has two components: the capacity provided by the concrete and the capacity provided by steel stirrups. The concrete capacity was calculated using the equation from ACI Code Table 22.5.5. The shear capacity of the stirrups is calculated using ACI Code Equation 22.5.8.5.3. The sum of these capacities yields the total shear capacity where stirrups are present. The concrete only capacity is the capacity of the beam in all other regions.

$$\phi V_c = \phi\, 2 \frac{\sqrt{f_c'}}{1000}\, bd \qquad \text{(ACI Table 22.5.5.1)}$$

$$\phi V_N = \phi \frac{A_v f_{yt} d}{s} \qquad \text{(ACI 22.5.8.5.3)}$$

The beam axial loading needs to be checked due to the soil pressure on the basement wall that is translated to the beam to determine if this affects the beam capacity. The load translated to the beam from the wall can be calculated by doing a force balance of the soil pressure and surcharge load. In this calculation, typical soil density for Ann Arbor and at-rest soil pressure coefficient values were assumed to be 100 pcf and 0.5 respectively based on the recommendation of John McCarthy, PE. Additionally, a surcharge of 100 psf was assumed and converted into a soil pressure load by multiplying it by the at-rest soil coefficient. The force of the surcharge and soil pressure were then calculated and the resultant force at the top of the wall that would be transmitted to the beam was calculated. If this force causes tension, this will reduce the capacity of the beam, especially in shear. However, if the force cause compression, it will increase the capacity of the beam in shear and no additional checks are required.

In addition to the beam, the bearing capacity of the existing wall are calculated using the ACI code and geotechnical force balance. The bearing wall capacity was calculated with ACI Equation 11.5.3.1 where l_c is the vertical distance between supports and k is the effective length factor (= 0.8 for fully fixed), h is the thickness of the wall, and the safety factor ϕ is 0.65.

$$\phi P_n = 0.55\, \phi\, f_c' A_g \left[1 - \left(\frac{k l_c}{32h}\right)^2\right] \qquad \text{(ACI 11.5.3.1)}$$

Lastly, the bearing capacity of the soil must be checked to ensure that it will hold the additional structure without any safety concerns or modification to the existing foundations. The original structural drawings provided the bearing capacity of the soil and the size of the foundations, so the existing bearing capacity can be calculated.

The existing capacity of the structure must then be compared to the existing demand and new demand to determine if the structure requires any modification. The structural plans provided the assumed loading conditions, so the loading, including the original live loading assumptions, can be calculated for the existing and new structure for all elements. This was undertaken for the wall and bearing capacity prior to the design of the system. However, the beam capacity will be dependent on point loads at the connection to the new structure and must be checked later. If the capacity of the structure is exceeded, structural modifications could be considered as another possible solution.

3.4.2.2 Design of Existing-Structure-Supported Option Members

The beam bearing the new structure was designed as a simply supported beam with supports 3 ft in from either end. The beam was designed such that the supports would fall within the area of the existing, concrete beam that has stirrups because it has the most shear capacity and likelihood of a viable design. The beam was assumed to be fully laterally supported due to the filler beams at 16” on center across the length of the beam. The shear force and bending moment diagrams for the new structure loading were calculated as the support conditions are not typical. From the bending moment diagram, the maximum moment was used to calculate the required plastic section modulus (Z_x). The required plastic section modulus was then used with AISC Table 3-2 to select the lightest section as denoted by bold entries with sufficient capacity. The self-weight of the beam was then added to the moment calculation, and the plastic section modulus was rechecked to ensure that the selected section could support the updated maximum moment.

The shear generated by the new system and beam was then compared to the remaining shear capacity of the concrete beam of the roof system. The shear generated by the new system at the supports was indicated by shear force diagram. A new analysis was performed on the existing beam including the point loads from the supports to generate a shear force diagram of the existing beam. Lastly, this new shear force diagram was compared to the shear force capacity generated previously. If sufficient capacity exists, no modification to the existing structure needs to be considered.

The final element to be designed at the conceptual phase was the vertical members connecting the new and existing beam. A pipe section was selected for this vertical member as it is typical in construction practices. The pipe section was designed as a short column because it will be less than 5 ft high. The axial force transmitted by the member was divided by its yield capacity to determine the gross area required for the member. The required area was then used in AISC Table 4-6 to determine a pipe size. Lastly, the strength of the pipe was then compared to the required strength to ensure it had sufficient capacity for its length. By assuming 5 ft for the pipe length, any shorter pipes should be acceptable as decreasing length will lead to increases in strength.

3.5 Iterative Refinement of Design Options

Upon completion of the first two conceptual designs, an analysis of benefits and disadvantages of each option was performed and used to inform a revision of the designs. The designs were analyzed on the criteria of constructability, cost implications, reinforcement of existing structure, and efficiency of the structure. This analysis was used to select optimal elements, discard poor elements, and ideate new possible solutions.

Ideation, refinement, and idea generation were repeated until the generated solution was deemed sufficiently suitable based on the analysis criteria. Upon the ideation of a new solution, conceptual designs of the new solution were created roughly following the same process of member design and capacity analysis of the existing structure.

3.5.1 Hybrid Steel Structure Conceptual Design

In refining the two conceptual designs, a hybrid structure in which some external support and some support from the existing structure was used and conceptually designed. In this case, the beams were supported by a pipe on one end and an external column with a moment connection on the other, such that most load went to the external column, but some was taken into the existing building. The full design calculations are given in Appendix F.

The same beam design process as used to calculate the existing-structure-supported beam case was used except for the fixity case. The fixity case was assumed to be simply supported on one end and fixed on the other. The shear and moment generated by this case was found using the equations in AISC Table 3-23. This moment was then used in the beam selection process detailed previously for the structure fully supported by the existing structure.

The shear capacity of the existing structural beam was calculated in the same way as with the structure that was fully supported by the existing building.

The pipe vertical supports between the new structural beam and the existing structure were selected using the same process as that of the pipes used in the structure fully supported by the existing structure.

The column running between the beam and external foundations was designed following the same process as that of the cantilever design. The axial load and moment were determined by calculating the shear and moment at the end of the beam that connects with the columns. The beam was assumed to be 16 ft above the foundation level due to the reduced beam depth as compared to the cantilever case. This data was used to determine the column size by the same method as the previous beam-column in the cantilever case.

3.6 Final Design Selection and Detailing

Upon the selection of a final design option, the connections between members and any specialty loading effects were considered. Both simple and moment connections were detailed. The primary loading effect addressed in the section was that of concentrated forces. Note, further finishing details are required prior to a complete set of construction plans being produced.

3.6.1 Bearing Connection Design

For the final design selected, the simple connections were assumed to be bearing connections and designed according to AISC guidelines. A full calculation of the bearing connection is detailed in Appendix G. The simple connection design assumed 7/8" high strength bolts. First, the bearing plate was designed such that it could accommodate the size of the beam flange (AISC Table 1-1), the vertical member (AISC Table 1-14), and the required minimum edge distance (AISC Table J3.4).

Then, the base plate thickness was designed assuming a cantilever of length n and a thickness of 1". First, the pressure under the plate was calculated by dividing the transmitted force by the plate area. Second, the length of the cantilever of the base plate was determined using Equation 4 where k_{des} is given as a parameter in AISC Table 1-1 and B is the length of the rectangular plate parallel to the flange length. Lastly, the thickness of the plate is calculated using Equation 5 (Geschwindner, Liu, and Carter, 2012) where f_u is the stress under the plate and F_y is the material yield strength. The calculated plate thickness is rounded up to the nearest 1/8" for fabrication purposes.

$$n = \frac{B}{2} - k_{des} \quad (4)$$

$$t_p = 1.49\, n \sqrt{\frac{f_u}{F_y}} \quad (5)$$

The process for calculating bearing plate thickness was repeated for the pipe support side of the bearing plate. The only difference in this case is a longer cantilever length. The larger of the two plate thicknesses generated is the plate thickness that will be selected for design.

The connection between the pipe and the bearing plate is designed to be a welded connection. The required strength of the weld was calculated using Equation 10 from AISC Section J2 where F_{EXX} is the strength of the weld electrode, w is the leg of the weld, and l is the length of the weld. The value of w is considered in 1/16" increments such that it can be scaled to achieve the required strength. The strength of the weld is equal to the factored load transmitted transversely through the bearing connection. Due to the transverse nature of the load, the strength of the weld can be increased by a factor of 1.5. The required strength can be divided by the increased weld capacity to determine the strength of the weld.

$$\phi R_n = \phi(0.6F_{EXX}(0.707wl)) \quad \text{(AISC J2-46)}$$

Next, the bolted connection between the bearing plate and the beam are checked for various limit states to determine the controlling limit state and calculate capacity of the connection. The bolts were assumed to be Group A325 – N bolts with a 7/8" diameter. Based on AISC Table J3.3, the bolt hole diameter is 15/16". The three limit states assessed for this connection are shear, tearout, and tension. Shear and tension are calculated using AISC Eq. J3-1 where F_n is the strength of the bolt in shear and tension, respectively, and A_b is the area of the bolt without threads in plane. The tearout limit state was assessed due to the bolt's proximity to the edge of the plate. AISC Eq. J3-6c is used to calculate the bolt capacity in tearout where l_c is the clear distance from the edge of the hole to the edge of the plate, t is the thickness of the plate, and F_u is the ultimate strength of the connected material. The capacity of the connection is then determined by multiplying the lowest capacity limit state by the number of bolts. This value is compared to the required strength to determine the adequacy of the connection.

$$R_n = F_n A_b \quad \text{(AISC J3-1)}$$

$$R_n = 1.2 l_c t F_u \quad \text{(AISC J3-6c)}$$

3.6.2 Moment Connection Design

The moment connection was designed as a welded flange plate connection with a single plate web connection that is bolted to the beam and welded to the column according to AISC standards. The welded flange plates transfer the moment from the beam to the column. The single plate web connection transfers the shear in the web of the beam to the flange of the column.

The welded flange plate was designed following the process presented by Gershwinder, Liu, & Carter in Section 12.3.2. In this process, the top plate was designed to carry the force transmitted by the moment connection assuming yielding was the controlling limit state. The top flange of the beam is checked for tension rupture, weld rupture, rupture of base materials, and block shear rupture. The bottom flange was also designed assuming yielding was the controlling limit state and additional limit states were checked; these limit states included plate local buckling, compressive strength of the plate, and rupture strength of the base material. Lastly, the welds to the column flange were designed and checked for base material rupture. A sample calculation including all equations used is given in Appendix H.

The single plate web connection was designed following the process presented in Example II.A-17A of the AISC Design Examples. The bolts were assumed to be ¾" with standard size holes and 3" spacing. The length of the plate was assumed to be 11.5" such that 4 bolts were used with a vertical edge spacing of 1 ¼". AISC Table 10-10a was used to determine the capacity of the bolts. This process can be iterated to add (space allowing) or remove bolts until the capacity is sufficient and efficient. Once the number of bolts is selected, the web bearing and tearout capacity were determined using AISC Table 7-1 which yielded the available strength of a bolt per inch thickness of the web. This was then multiplied by the number of bolts and the thickness of the web to yield the tearout capacity. This capacity was compared to the required strength to determine if it was a controlling limit state and see if the capacity was sufficient. Lastly, block shear, shear yielding, and shear rupture were assumed not to govern because the beam is not coped.

3.6.3 Concentrated Force Effect Analysis

The web of the beam was checked to ensure it could withstand the concentrated force exerted on it due to the bearing plate connection. The beam in this design experiences a considerable concentrated load from the bearing plate which makes it susceptible to web local yielding and web local crippling. Both limit states were checked to ensure the bearing plate sufficiently diluted the load to avoid these types of failure under the concentrated load. A full calculation is detailed in Appendix I.

The web local buckling strength was found using AISC Equation J10-3 where F_y is the yield strength of the material, t_w is the web thickness, k is the k_{des} that is given in AISC Table 1-1 in the entry for the beam, and l_b is the length of bearing plate parallel to the length of the beam.

$$R_n = F_{yw} t_w (2.5k + l_b) \qquad \text{(AISC J10-3)}$$

The web local crippling was found using AISC Equation J10-5b which applies to beams where the concentrated load is applied within half the depth of the beam from the end of the beam and the ratio of the bearing plate length to beam depth is greater than 0.2. In this equation, the variables from J10-3 have the same meaning; d is the depth of the beam; t_f is the thickness of the beam flange; E is the modulus of elasticity of steel; and Q_f is a shape factor taken as 1.0 for wide flange sections.

$$R_n = 0.40\, t_w^2 \left[1 + \left(\frac{4l_b}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}\, Q_f \qquad \text{(AISC J10-5b)}$$

4.0 Results and Discussion

This section details the results of all analysis, design and detailing for the Honors Porch.

4.1 Loading Patterns

The factored area load for the steel structure was calculated to be 184 psf using LRFD load factors. The loading for live and dead load elements is given in Table 1 below. Note that snow loading would need to be considered prior to the publication of construction documents.

Table 1: Load elements on steel structure

Loading Type	Load (psf)
Live Load	100
Decking Load	5
Wood Filler Beams	15

4.2 Existing Conditions

The plans of the existing structure were obtained from the College of Engineering Facilities Management and Coordination department providing construction details of the existing structure. The plans showed that the existing structure was concrete on the first floor and steel on the second floor. The size and construction of members were shown and additional details such as material strength were provided in the general notes. The plans also included geometry and showed that there were no HVAC concerns in the area where the Honors Porch would be constructed.

From the plans, an initial rough geometry of 80 ft by 30 ft was selected. This area is centered between the two existing doors from the Honors Program and International Programs in Engineering offices. This avoids skylights present on the roof which would pose a challenge to construction.

4.3 Steel Design Cantilever Option and Analysis

Figure 1 below shows the design of the cantilever option including steel girders, steel columns, wood filler beams, decking, and a conceptual placement of the stairs. The girders are W40x199 wide flange sections at 20 ft on center, the columns are W27x178 wide flange sections, and the wood filler beams are 4x12 timber at 16" on center. The wood filler beams are staggered such that they cross the entire girder flange to optimize connection locations. The girders are connected to the column by a moment connection.

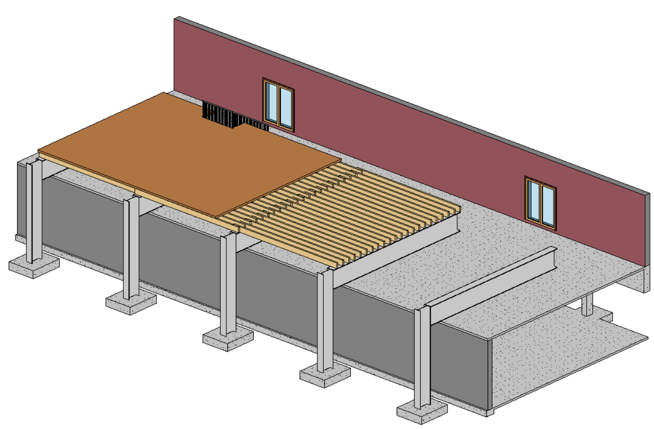


Figure 1: Cantilever design option 3D rendering

The 4x12 timber filler beams have sufficient structural capacity but are less common than dimensional lumber and leads to additional construction costs. Under the factored area load of 184 psf for a length of 20ft with beams at 16” on center, the stress experienced by the beam is 1244 psi which is within the capacity of 1600 psi. The capacity is sufficiently close to the required stress that the structure is considered efficient. However, a 4x12 is considered timber and will not be as common, easy to procure, or inexpensive as dimensional lumber is.

The steel girders were designed to be W40x199 sections using a maximum deflection of 1” and they do not interact with the existing structure, but they have a large structural depth and weight. The natural frequency of the beam for a 1” maximum deflection was 3.11 Hz which was deemed sufficiently far from the natural frequency of human walking at 2Hz. The required moment of inertia of the section at a 1” maximum deflection was 14,479 in^4, and the lightest section to satisfy this requirement was a W40x199 beam. This beam has a depth of approximately 40”. With the 4x12 filler beams and decking on top, the total structural depth is approximately 4’-5”. This depth will result in a longer stairway for access to the porch which reduces the useable area by limiting the depth of the structure or by cutting into useable area. These elements would also obstruct the existing view from the Honors Program Office windows. Further, the beams weigh 199 lbs. per foot of length. This large size and weight will make construction more challenging. The beam is also quite inefficient as the end furthest from the column bears minimal load relative to the capacity of the section.

The column design yielded a W27x178 section with an estimated length of 20 ft. The beam was designed to bear a maximum moment from the cantilevered beam of 1656 kip-ft and an axial load of 110 kips. The large size of the column was governed by the high moment relative to the axial load. The flange of the column is also larger than the flange of the beam, so a connection between the two is possible. The length of the column is assumed to be 20 ft by accounting for the height of the basement wall at 12 ft, the structural depth of 4’ – 5”, and a gap between the new structure and the existing roof such that the roof can be replaced throughout the life of the building.

The large structural members will likely require large foundations causing a longer cantilever length. Large foundations are likely required to transmit the load to the soil and would require the columns to move further from the building. As a result, if the design area is to be maintained, the cantilever would have to extend leading to an increase in the size of the beams and columns.

4.4 Steel Design of Existing-Structure-Supported Option and Analysis

Figure 2 below shows the design of the existing-structure-supported option including steel girders, steel vertical members, wood filler beams, decking, and a conceptual placement of the stairs. The girders are W14x22 wide flange sections at 9 ft on center, the vertical members are 3” standard pipes placed 3 ft in from the ends of each beam, and the wood filler beams are 2x10 dimensional lumber at 16” on center. The wood filler beams are staggered such that they cross the entire girder flange to optimize connection locations. The girders are connected to the column by a moment connection.

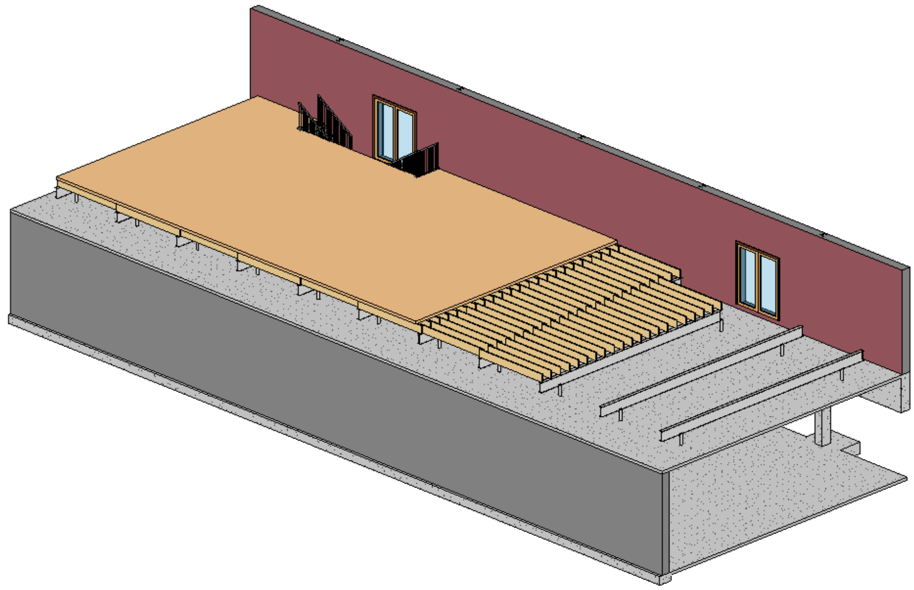


Figure 2: Existing-Structure-Supported design option 3D rendering

The wood filler beams were downsized from an initial estimate of 4x12 timber to 2x10 dimensional through an iterative process comparing the stress due to loading to the capacity of the wood. At 4x12 the stress in the wood was 252 psi which was approximately an eighth of its capacity. The size of the lumber was subsequently decreased until the capacity was at least half the capacity of lumber (800 psi). This occurred at a 2x10 section which had a capacity of 859 psi. The reduced size of the lumber makes this a more practical and cost-effective way to construct the decking.

The steel girder design yielded a W14x22 section and a much smaller structural depth than the cantilever design. The section was required to support a maximum moment of 115 kip-ft. This maximum moment yielded a required plastic section modulus of 31 in^3. The W14x22 satisfied this plastic section modulus and was verified to have sufficient capacity to bear the self-weight of the girder. The structural depth of this system is approximately 2' – 2" which leads to more useable area and minimal view obstruction for the offices adjacent to the deck. A drawback is that 6 more beams are required such that they align with the concrete beams of the roof that will transmit the load through the structure.

The vertical members were designed assuming a short column and yielded a 3 inch diameter standard pipe section, but the members are limited in their placement options. The vertical members were designed assuming a required load of 25.5 kips and a yield strength of 35 ksi. This yielded a required area of 0.73 in^2 which was satisfied by the 3" pipe assuming a length of 6ft or less. These vertical members will be significantly less expensive than the columns in the cantilever design due to their small size. The vertical members must be placed in an area with shear stirrups to have sufficient shear capacity, but not in such a way that the connection interferes with the stirrups.

The existing structural capacity was found to be sufficient in all elements except for the shear capacity of the concrete beam supporting the roof. The required steel in the roof beam designated

B102 in the structural drawings was determined to be 2.95 in^2, and the existing steel is 9.08 in^2. This indicates that there is a large amount of unused flexural capacity in the beam that load could be added to the beam. The shear capacity of the beam was determined to be 53 kips in the region that includes stirrups. The shear capacity generated by the existing roof design loads is a maximum of 29 kips, but the new structure increases that maximum shear in B102 to 54.3 kips. This indicates that retrofit of the beam would be required before this option would be viable for construction. The wall axial capacity was calculated to be 146 k/ft of wall. The lateral load on the wall is 9.7 kips and places the wall into tension which reduces its shear capacity. This further indicates that retrofit is required prior to construction. Lastly, the bearing capacity is 8 k/ft of length of foundation. Using the design loads, the capacity used by the current structure is 3.7 k/ft and the required bearing strength of the new foundation would add an additional 2.5 k/ft. This would be a total bearing load of 6.2 k/ft which is within the bearing capacity indicating that the foundations have sufficient bearing capacity for the additional load.

Lastly, no foundational work would be required for this system as it will bear on the existing foundations. This represents a potential for significant time and cost savings during construction as no excavation will be needed. This increases the range of times in which the structure can be completed and reduces the number of trades required to complete the job.

4.5 Steel Design of Hybrid Option and Analysis

Figure 3 below shows the design of the hybrid design option including steel girders, steel columns, wood filler beams, decking, and a conceptual placement of the stairs. The girders are W18x35 wide flange sections located 9 ft. on center, the vertical members are 3" standard pipes placed at the interior end of the beam and a W10x60 column at the exterior end, and the wood filler beams are 2x10 dimensional lumber at 16" on center. The girders extend 32 ft. rather than 30 ft over the deck from the column to maximize the useable area of the deck. The wood filler beams are staggered such that they cross the entire girder flange to optimize connection locations. The girders are connected to the column by a moment connection.

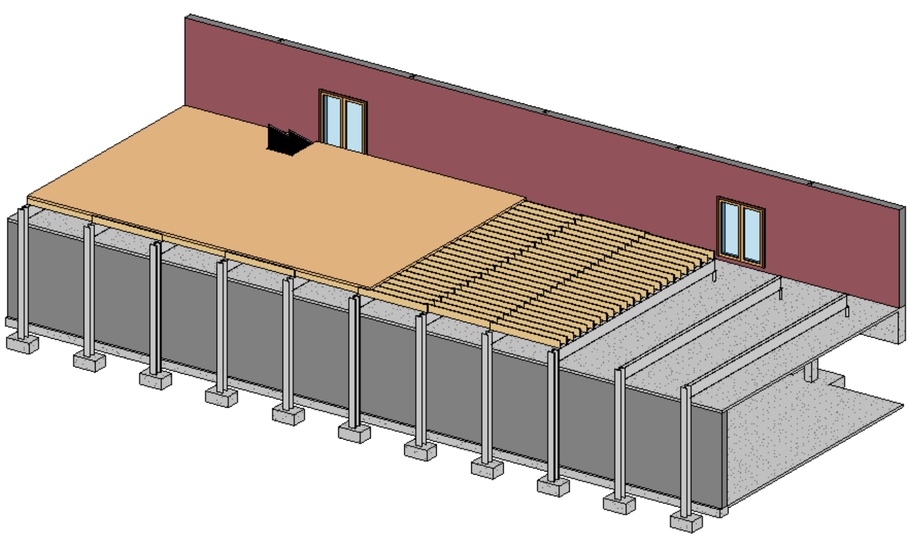


Figure 3: Hybrid design option 3D rendering

The 32 ft. W18x35 girder design uses a moment connection with the column to transmit most of the load through the column while dissipating some through the vertical pipe into the existing building to maximize the efficiency of the structure. The maximum moment in the girder is 212 kip-ft at the moment connection end. The W18x35 section has sufficient capacity to carry this moment and its self-weight. The additional length enables a useable area of 30 ft. and an extra two feet to extend from the edge of the existing roof to the column. The distribution of load in the beam reduces the size of the beam relative to the cantilever option and reduces the amount of load transmitted to the existing structure to be within the shear capacity of the concrete beam. Additionally, the structural depth is only increased from the existing-structure-supported option by 4”. This maintains the visibility from the office spaces and minimizes the number of stairs required. One drawback is that 11 beams will be required for the structure rather than the 5 required for the cantilever.

The vertical pipe member design yielded a 3” standard section. The required strength was 20.4 kips in axial loading, and the provided capacity was 53.7 kips for a pipe of 6 ft in length or less. As with the existing-structure-supported option, this represents a cost effective and structurally efficient solution.

The column was designed in the same way as the cantilever column yielding a W10x60 section with a height of 16 ft. The maximum moment carried by the column including the self-weight of the girder is 217 kips. The axial load in the column from the shear in the beam is 34 kips. The interaction of these forces is within the acceptable range for the W10x60 section. The length of the column accommodates the wall, structural depth, and a gap between the new and existing structure. It is reduced from the cantilever option because of the reduced structural depth making it a more economical option due to the reduced material.

Finally, new foundations will be required for this design; however, they are expected to have a smaller required size than the cantilever option. As some load is transmitted through the building, the load on the new foundation will be reduced and thus will minimize the size required for the new foundations. The foundations will impose additional primary and secondary costs to the design because excavation and earthwork will be required and the times at which construction can be performed are more limited. There is also a risk when excavating of encountering unforeseen circumstances that need to be addressed before construction can be completed.

4.6 Final Design Selection and Detailing

The hybrid design option was selected for final design and detailing due to its optimization of advantages and disadvantages of the previous two designs. The hybrid option requires no retrofit of the existing structure and is more structurally efficient than the cantilever option, which is the costliest options of the other alternatives. Further, it utilizes standard size lumber and minimizes the visual impact on the adjacent offices. However, it still has the disadvantage of requiring foundations and an increased number of beams from the cantilever option.

For the final design, the connection of the beam to the vertical elements were designed. The bearing connection design consists of a 6x8x3/4” inch bearing plate, 4 A325-N 7/8” bolts connecting the plate to the beam, and a 3/4” weld between the pipe and the bearing plate. The bearing plate is designed such that there is sufficient length parallel to the length of the beam for the minimum edge distance of the bolt holes and the 3” pipe to be welded to the plate. The bearing plate thickness is governed by the pipe side of the plate which has a longer cantilever distance than the beam side. The 7/8” bolts were checked for their shear capacity, tearout capacity and tension capacity. The shear capacity of 32.4 kips per bolt governed and resulted in a capacity of 97.2 kips for all four bolts which more than exceeded the required strength of 34 kip. The weld between the pipe and the plate was determined to have a capacity of 50.1 kip assuming loading in the transverse direction to the weld which also has sufficient capacity for the required load.

The moment connection between the beam and the column was designed such that the top flange plate is 5”x23”x 1 5/8”, the bottom flange plate is 7”x23”x 1 ¼”, and the single plate for web connection is an 11.5”x4.5”x ¼” plate. The bolts and welds for each element were also designed. The top plate is a ¼” fillet weld with an E70 electrode along the length of the plate on both sides. The bottom plate was assumed to have the same type of weld for consistency, and this assumption was verified to yield sufficient capacity. All other relevant limit states detailed in the methodology section were verified to have sufficient capacity. The web plate connection to the beam was designed based on bolt capacity to be 3 bolts at 3-inch center to center spacing with an edge distance of 1.25”. Web tearout was also checked and verified to have sufficient capacity.

The welds of the flange and web plates to the column were also designed. The flange plates were designed assuming transverse loading and a required capacity of 137 kips. This yielded a 5/16” weld for the top plate and a 7/16” weld for the bottom plate. This weld is on the top and bottom edge of the plate. The web plate is welded to the column with a 1/8” weld which is governed by the minimum weld size.

Lastly, due to the bearing connection on the beam, concentrated load effects were checked to ensure that the concentrated load was sufficiently diluted by the bearing plate. This analysis showed that the bearing plate has sufficient capacity in the web to prevent local yielding and local crippling.

Prior to the publication of construction drawings, further finishing steps are required. Further structural design includes the design of base plates at the connection with the existing structure and foundations, spandrel beam design and connections, and foundations. Architectural elements such as tables, chairs, and other aesthetic elements need to be selected to ensure any concentrated loads are accounted for. Lastly, consultation with a construction management firm is recommended to verify the cost assumptions made in this design, determine material availability, and evaluate the design for constructability.

5.0 Conclusion

In this project, the structure of a space, designated the Honors Porch, was designed to provide a community gathering space for students. It is important because it provides a space for community building between students in a large program, has sustainability implications for the structural engineering field, and provides structural engineering experience in a structure involving multiple materials.

The area selected to build the Honors Porch outside the existing Honors Program Office above a low roof of the Chrysler building was selected. This presented the challenges of both designing a new structure and analyzing an existing structure to determine its capacity to bear additional loads.

An iterative design approach utilizing the loading requirements of the new structure and the capacity of the existing structure was used to generate three conceptual designs: a cantilever option, an existing-structure-supported option, and a hybrid option.

Each design had their own constructability and cost considerations. The cantilever option was particularly inefficient at carrying load and would have high material costs due to the foundation requirements and size of the steel and timber members. The existing-structure-supported option was structurally efficient, but the existing structure would need to be retrofitted to bear the shear load of the new structure representing a significant cost and construction challenge. The hybrid option has the highest potential for a cost effective, structurally efficient, and constructible structure because the members are of a much smaller size than those of the cantilever and no retrofit is required for the existing structure.

The hybrid design was selected for the final design process and detailing. The beam bearing and moment connections were designed. The concentrated load effects of the bearing connection were also analyzed to ensure the load was sufficiently distributed by the bearing plate.

6.0 References

American Concrete Institute (2019). *Building Code Requirements for Structural Concrete (ACI 318-19).* ACI.

American Institute of Steel Construction (2016). *Design Guide 11: Vibrations of Steel-Framed Structural Systems Due to Human Activitly*. Appendix A. AISC

American Institute of Steel Construction (2017). *Steel Construction Manual* (15th edition).

American Institute of Steel Construction (2017). *Design Examples: Companion to the AISC Steel Construction Manual* (15th Edition). Example II.A-17A. AISC.

American Society of Civil Engineers (2016). *Minimum Design Loads and Associated Criteria for Buildings and Other Structures.* ASCE.

Engineering ToolBox (2009). *Wood Beams – Strength of Material*. https://www.engineeringtoolbox.com/wood-beams-strength-d_1480.html

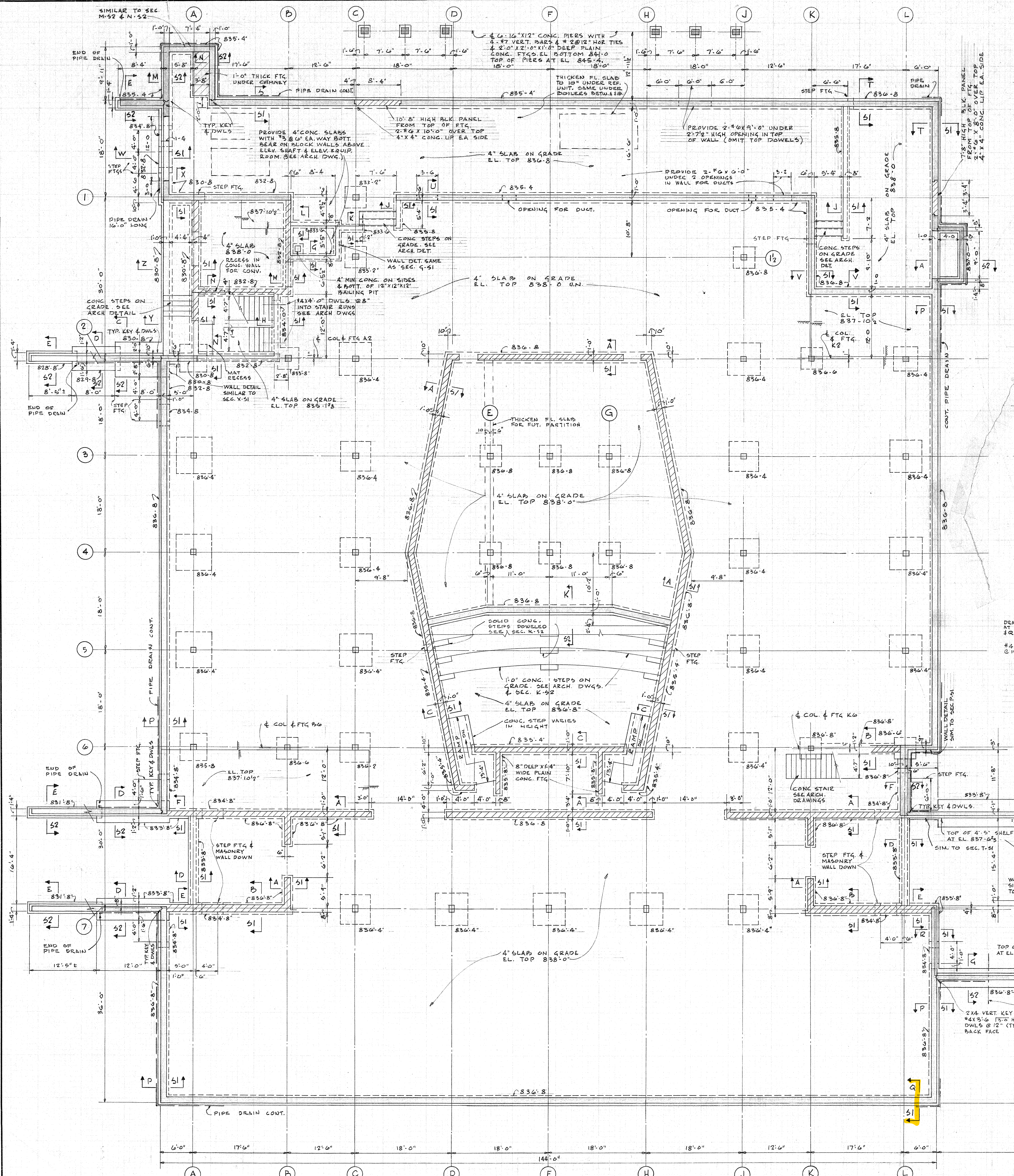
Gershwinder, L., Liu, J., & Carter, C (2012). *Unified Design of Steel Structures* (3rd Edition). Providence Engineering Corp.

Heinemann, P. and Kasperski, M. (2017). *Damping Induced by Walking and Running*. Procedia Engineering. https://www.sciencedirect.com/science/article/pii/S1877705817340481#:~:text=For%20both%20locomotion%20forms%20walking,for%20the%20preferred%20step%20frequency.

Wight, J. (2016). *Reinforced Concrete: Mechanics and Design* (7th edition). Pearson.

7.0 Appendices

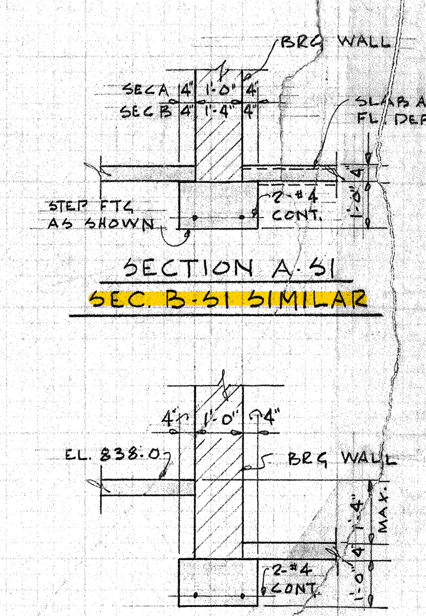
Appendix A: Original Structural Plans*
Appendix B: Wood Beam Capacity Calculation
Appendix C: Cantilever Design Calculations
Appendix D: Existing Structural Capacity Calculations
Appendix E: Existing Structure Supported Design
Calculations Appendix F: Hybrid Design Calculations
Appendix G: Bearing Connection Design Calculations
Appendix H: Moment Connection Design Calculations
Appendix I: Concentrated Load Effect Calculations



FOUNDATION PLAN

SCALE 1/8" = 1'-0"

NOTE- ELEVATIONS SHOWN ARE AT BOTTOM OF FOOTINGS UNLESS NOTED OTHERWISE
PROVIDE 6X6-#10 MESH NEAR TOP OF SLAB IN ALL SLABS & STEP SLABS ON GRADE
CROSS HATCHING INDICATES EITHER BRICK OR BLOCK MASONRY BEARING WALLS.
FOOTING STEPS ARE INDICATED AT TOP OF FOOTING.



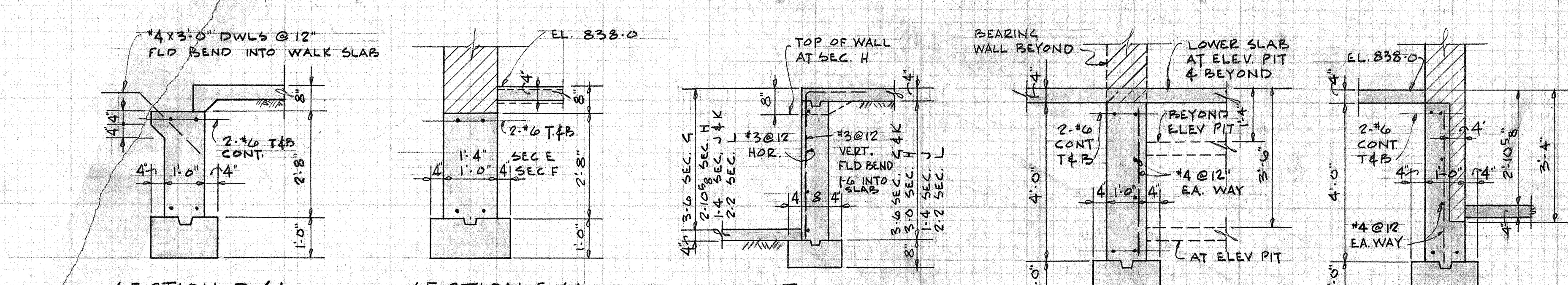
SECTION A-S1
SEC. B-S1 SIMILAR

SECTION C-S1

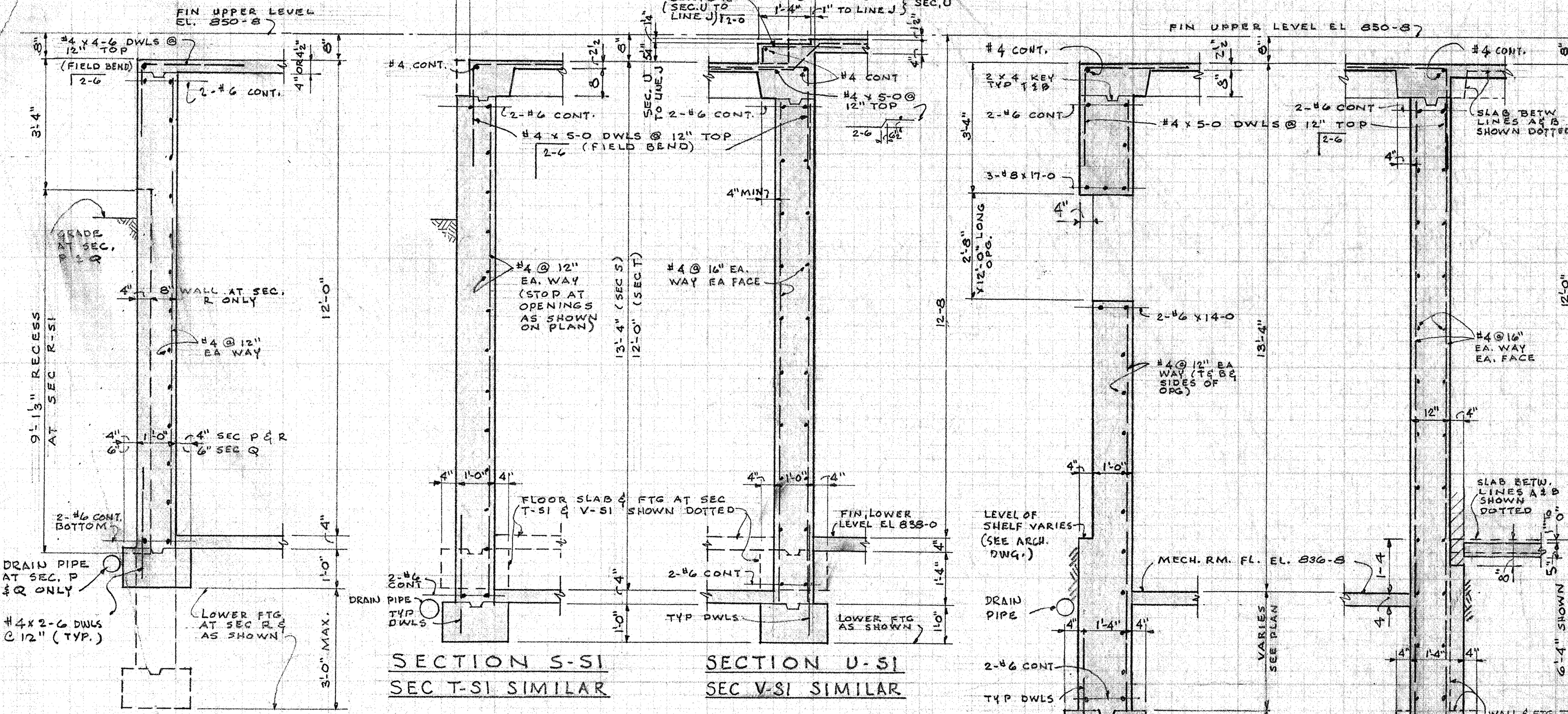
COLUMN & FOOTING SCHEDULE

COLUMN NUMBER		A2 A6	A3 A4 A5	B2 K2 B6 K6	C1	C1½ J1½ E3 E4 F3 F4 G3 G4	C2 J2	C3 C4 C6 J3 J4 J6	C5 J5	C7 J7	D1 J1 F1 H1	D7 F7 H7	L2 L6	L3 L4 L5
LOAD	ROOF	17	22	—	17	—	32	24 32	24	17	22 17	22	17	22
SUPPORTING	UPPER	55	120 110	48	51	55 44	134	112 104 97	121	108	54 31	125	69	120 110
	FOOTING	68	122 112	50	53	57 46	136	114 106 99	123	110	54 31	127	71	122 112
STEEL	SIZE	6WF15.5	6WF15.5	—	6WF15.5	—	6WF15.5	6WF15.5	6WF15.5	6WF15.5	6WF15.5	6WF15.5	6WF15.5	6WF15.5
COL.	B.P. SIZE	7x½x7	7x½x7	—	7x½x7	—	7x½x7	7x½x7	7x½x7	7x½x7	7x½x7	7x½x7	7x½x7	7x½x7
EL. BOTT.	B.P.	850-6¼	850-6¼	—	850-6¼	—	850-7½	850-7½	850-7½	850-6¼	850-6¼	850-6¼	850-6¼	850-6¼
CONC.	SIZE	12x12	12x12	12x10	12x12	12x12	12x12	12x12	12x12	12x16	12x12 IN WALL	12x16	12x12	12x12
COL.	VERT. REINF.	4-#6	4-#7	4-#5	4-#6	4-#6	4-#8	4-#6	4-#7	4-#7	4-#6	4-#7	4-#6	4-#6
	HOR. REINF.	#2@12	#2@12	#2@10	#2@12	#2@12	#2@12	#2@12	#2@12	#2@12	#2@12	#2@12	#2@12	#2@12
FOOTING	SIZE ◻	5-0	6-8	4-0	4-0	4-0	6-0	5-6	6-0	5-6	STRIP	6-0	4-6	6-0
	DEPTH	1-0	1-4	1-0	1-0	1-0	1-4	1-4	1-4	1-4	1-0	1-4	1-2	1-4
	REINF. E.W.	12-#4	14-#5	8-#4	8-#4	8-#4	12-#5	14-#4	12-#5	14-#4	—	12-#5	10-#4	12-#5
	EL. BOTT.	A2-830-8 A6-835-8	836-4	SEE PLAN	833-2	C1½ 833-2 REM. 836-8	836-4	C6 836-2 REM. 836-4	836-4	836-4	835-4	836-4	L2 836-4 L6 836-6	836-4

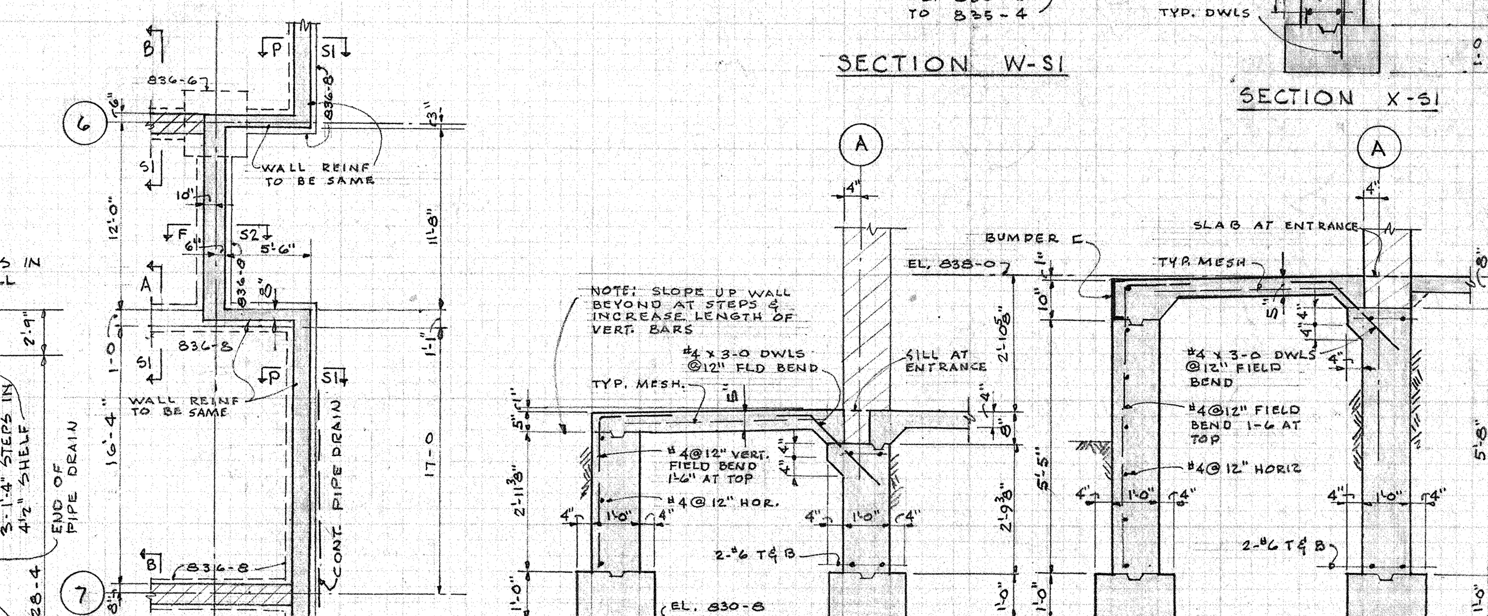
NOTE- FIRST DIMENSION OF CONC. COLUMN SIZE IS IN EAST & WEST DIRECTION IN PLAN.
PROVIDE DOWELS IN FOOTINGS TO MATCH VERTICAL COLUMN BARS WITH 24" DIA. LAP.
PROVIDE 2-¾ϕ X 1'-0" (+3" HOOK) ANCHOR BOLTS FOR EACH STEEL COLUMN WITH ¼" LEVELING PLATES UNDER BASE PLATES AND ¾" GROUT.



SECTION D-S1
SECTION E-S1
SEC. F-S1 SIMILAR
SECTION G-S1
SEC. H,J,K,L-S1 SIM.
SECTION M-S1
SECTION N-S1



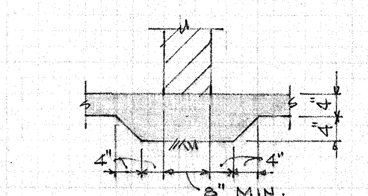
SECTION P-S1
SEC Q-S1 SIMILAR
SEC R-S1 SIMILAR
SECTION S-S1
SEC T-S1 SIMILAR
SECTION U-S1
SEC V-S1 SIMILAR
SECTION W-S1
SECTION X-S1



FOUNDATION PLAN OF ALTERNATE NO. 1
SCALE 1/8" = 1'-0"

SECTION Y-S1
SECTION Z-S1

SCALE - 3/8" = 1'-0" (TYPICAL FOR ALL SECTIONS)



DET. OF SLAB ON GRADE AT BLOCK PARTITIONS

GENERAL NOTES:

SOIL VALUE ASSUMED TO BE 4000 PSF EXCEPT 3000 PSF WEST OF COL LINE B.

LIVE LOADS ASSUMED AS FOLLOWS

ROOF	30 PSF
CLASSROOMS, LOUNGE, LOBBY, EXTERIOR PLAZA	80 PSF
STAIRS, LANDINGS	100 PSF

BOTTOM OF FOOTING ASSUMED TO BE IN SUITABLE BEARING SOIL. IF IN THE OPINION OF THE CONTRACTOR POOR SOIL IS ENCOUNTERED AT FOOTING LEVEL, NOTIFY ARCHITECT AT ONCE BEFORE PLACING FOOTINGS.

BRACE CONCRETE WALLS AGAINST BACKFILL UNTIL SUPPORTING SLABS ARE IN PLACE.

SPECIFIED STRENGTH OF CONCRETE TO BE 3000 PSI IN 28 DAYS, EXCEPT EXPOSED CONCRETE AT LOADING DOCK, CONC. CURBS, SIDEWALK SLABS AND COOLING TOWER BASIN TO BE 4000 PSI CONC. WITH ENTRAINED AIR. SEE MECH. DWG. FOR C. TOWER.

REINFORCING BARS #3 & LARGER TO BE DEFORMED PER A.S.T.M. A 305. REINF BARS TO BE INTERMEDIATE OR HARD GRADE NEW BILLET STEEL (A.S.T.M. A15) OR RAIL STEEL (A.S.T.M. A16)

LAP WALL REINF. 24 BAR DIAMETERS AND PROVIDE CORNER BARS ON OUTSIDE OF CORNERS TO MATCH HORIZ. REINF.

PROVIDE BAR PLACING DIAGRAMS SHOWING CHAIRS, BOLSTERS, ETC. ALSO SEE NOTE ON DOWELS UNDER COLUMN SCHEDULE.

PROVIDE ¾" CHAMFER ON EXPOSED CORNERS OF CONC. BEAMS & COLUMNS EXCEPT WHERE FLUSH WITH MASONRY PROVIDE SQUARE CORNERS.

STRUCTURAL STEEL (A.S.T.M. A-36) TO BE FABRICATED IN ACCORDANCE WITH AISC SPECS. PROVIDE ONE SHOP COAT OF PAINT AS SPECIFIED.

SHOP CONNECTIONS TO BE WELDED. FIELD CONNECTIONS TO BE WITH ¾"ϕ HIGH STRENGTH BOLTS EXCEPT AS OTHERWISE CALLED FOR OR SPECIFIED TO BE WELDED OR WITH ¾"ϕ UNFINISHED BOLTS.

SUBMIT SHOP DRAWINGS SHOWING AMOUNT & KIND OF WELDING.

STEEL JOISTS TO BE LA-SERIES (22000 PSI) AND TO BE FABRICATED IN ACCORDANCE WITH JOIST ASSOCIATION SPECS. PROVIDE ONE SHOP COAT OF PAINT. SUBMIT SHOP DRAWINGS.

WELD OR BOLT JOISTS TO SUPPORTING STEEL WITH 3¾" MIN. BEARING.

MINIMUM BEARING ON MASONRY TO BE 6" FOR JOISTS AND 8" FOR BEAMS U.N.

#443

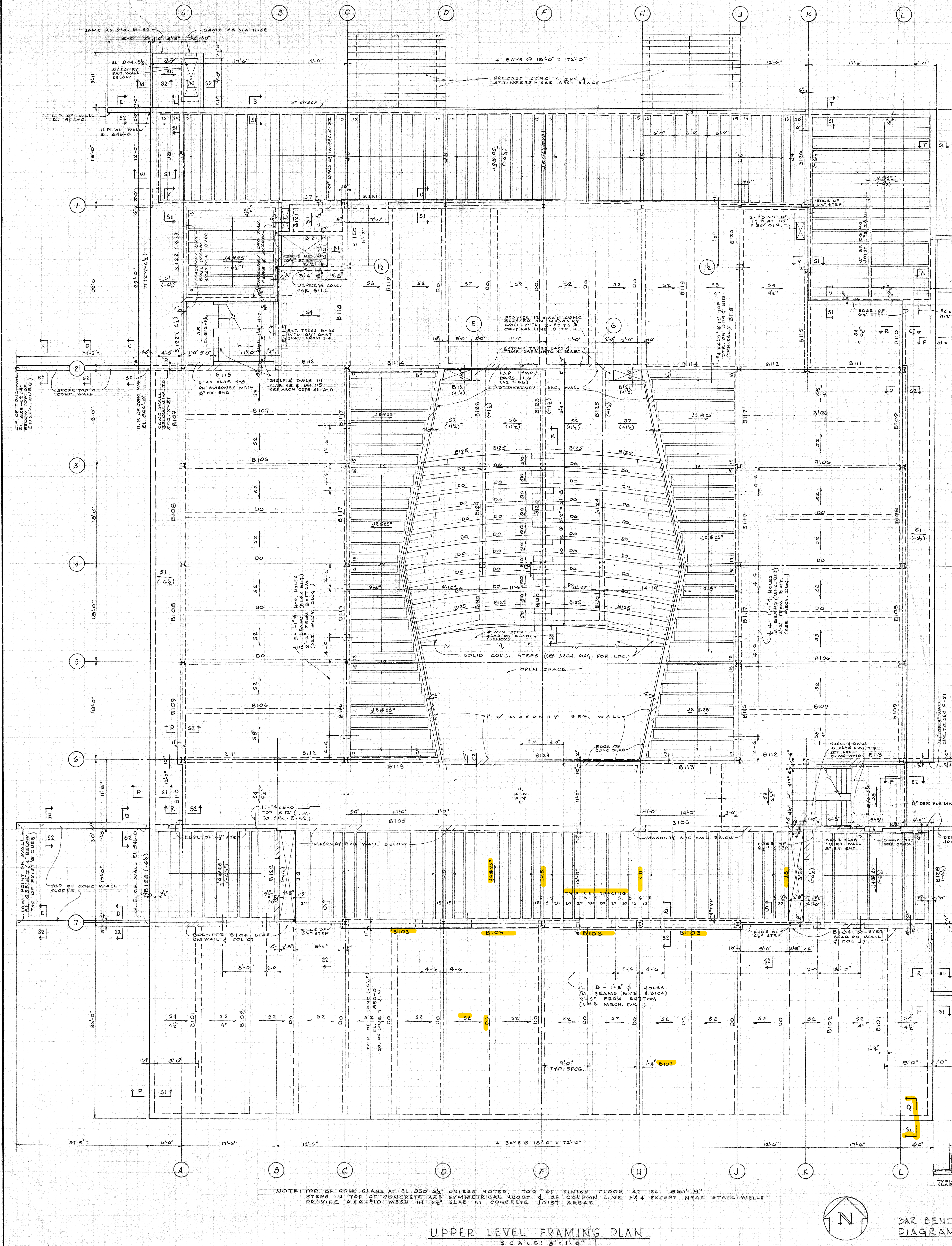
THE UNIVERSITY OF MICHIGAN
OFFICE OF THE UNIVERSITY ARCHITECT • ANN ARBOR, MICHIGAN

CHRYSLER CENTER FOR CONTINUING ENGINEERING EDUCATION

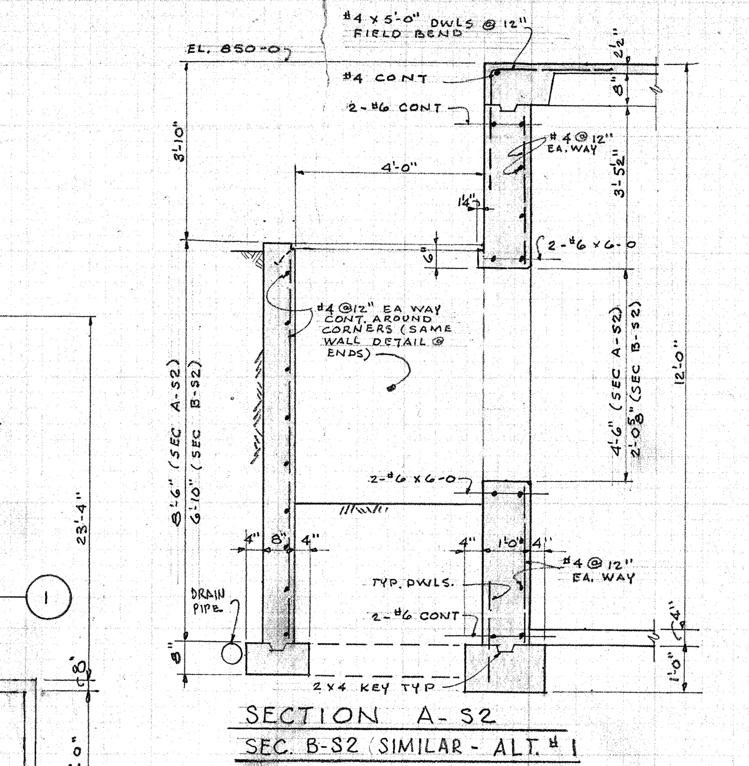
SWANSON ASSOCIATES, INC.
ARCHITECTS ENGINEERS
WEST LONG LAKE ROAD, BLOOMFIELD HILLS, MICHIGAN

SHEET TITLE: LOWER LEVEL FOUNDATION PLAN, COLUMN SCHEDULE & DETAILS

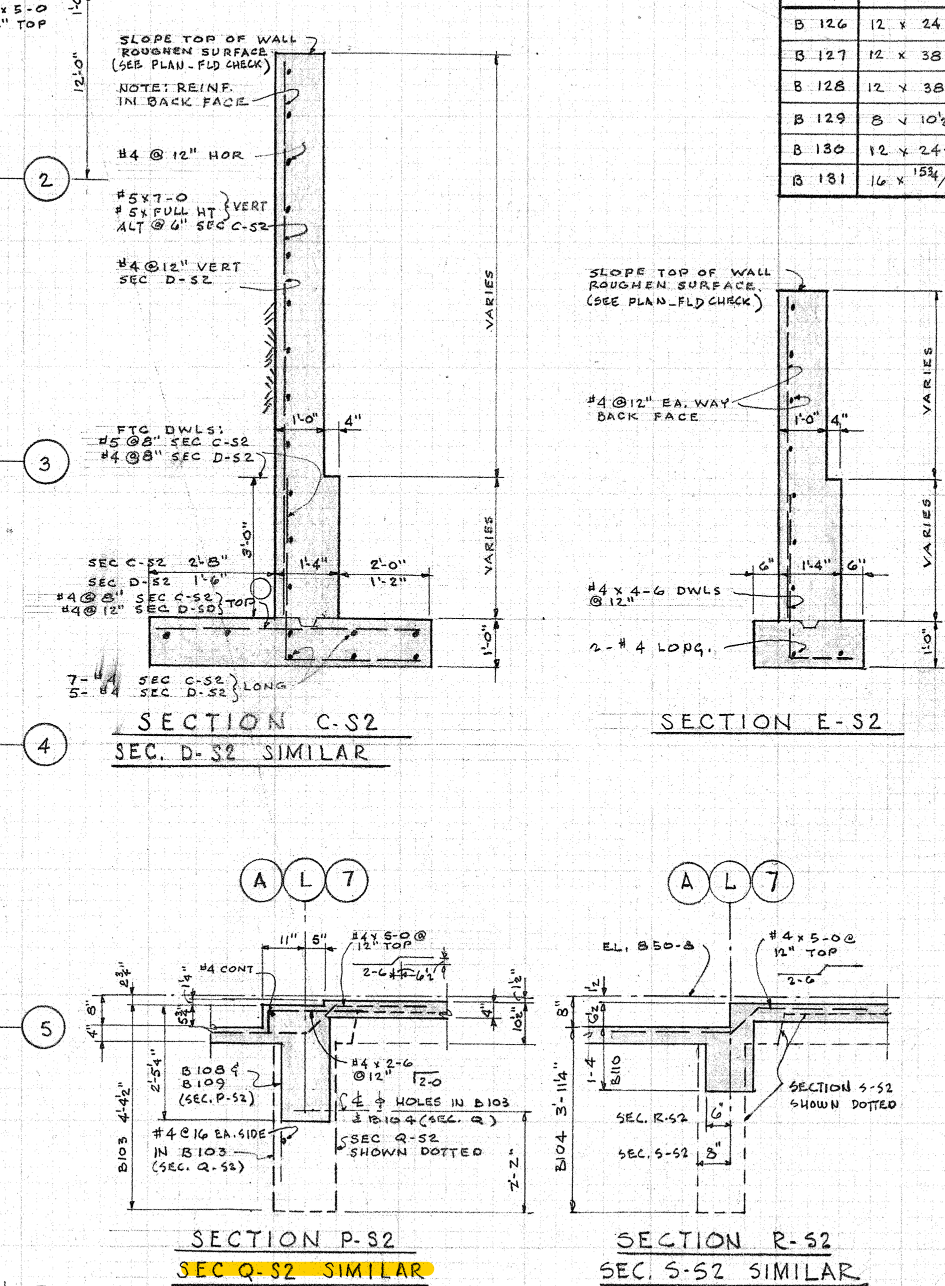
DES.	
DRWN.	RR & KP
CHKD.	RP
APPV'D	BE
DATE 5-10-66 PRELIMINARY	
DATE 6-15-66 CONSTRUCTION	
DATE 1-31-68 FINAL RECORD	
U. OF M. PROJECT NO.	264
ARCHITECT'S JOB NO.	6508
SHEET NO.	S-1



UPPER LEVEL FRAMING PLAN
SCALE: ⅛" = 1'-0"



SECTION A-S2
SEC. B-S2 SIMILAR - ALT. #1



SECTION C-S2
SEC. D-S2 SIMILAR

SECTION E-S2

SECTION P-S2
SEC Q-S2 SIMILAR

SECTION R-S2
SEC. S-S2 SIMILAR

CONCRETE BEAM SCHEDULE

MARK	SIZE	REINFORCING BARS			STIRRUPS		REMARKS
		BOTTOM	BENT	TOP	SIZE	SPACING EA. END U.N.	
B 101	16 x 24	4-#10	4-#10	—	#4	5", 10 @ 10"	2 LAYER OF REINF.
B 102	16 x 24	4-#10	4-#9	—	#4	5", 9 @ 10"	DO.
B 103	16 x 53¾/52½	4-#9	—	2-#7 x 27'-0" CTRD	#3	8", 5 @ 16"	ADD 2-#4 @ 16 EACH SIDE CONT. SPREAD #4 @ 16 & STIRRUPS AROUND OPG
B 104	12 x 53¾/47¼	3-#9	—	2-#7 x 40'-0"	#3	12", REM. @ 24"	BOLSTER BM. DO.
B 105	12 x 18½	2-#9	2-#8	—	#3	3", 3 @ 6", 4 @ 8"	EXT TRUSS BARS 4'-0" PAST SUPPORT
B 106	12 x 30½	4-#8	2-#9	—	#3	4", 3 @ 8", 5 @ 10"	
B 107	12 x 30½	4-#9	2-#9	—	#3	3", 4 @ 6", 6 @ 10"	
B 108	12 x 29¼/30½	2-#8	2-#9	—	#4	6", 7 @ 12"	SEE SEC. P-S2
B 109	12 x 29¼/30½	2-#8	2-#10	—	#4	4", 5 @ 8", 4 @ 12"	EXT. TRUSS BARS INTO B 110
B 110	12 x 16/22½	3-#6	—	2-#6 x 16'-0"	—		
B 111	12 x 22½	2-#7	2-#8	—	#3	3", 5 @ 6", 5 @ 8"	
B 112	12 x 22½	3-#7	—	2-#7 x 21'-0" CTRD	#3	4", 4 @ 8"	
B 113	12 x 18½	2-#7	2-#7	—	#3	4", 4 @ 8"	
B 114	12 x 22½	2-#10	2-#8	—	#3	3", 4 @ 6", 4 @ 8", 4 @ 10"	
B 115	12 x 14¼"	2-#6	2-#6	—	#3	3", 4 @ 6"	
B 116	12 x 53¾"	3-#9	—	2-#8 x 23'-0"	#3	8", 8 @ 16"	ADD 2-#4 @ 16" EA SIDE CONT SPREAD #4 @ 16" & STIRRUPS AROUND OPGS
B 117	12 x 53¾	3-#9	—	2-#7 x 27'-0" CTRD.	#3	8", 3 @ 16"	DO.
B 118	16 x 22½	3-#6	2-#8	2-#7 x 10'-0" CTRD ON LINE 2	#3	3", 5 @ 6", 5 @ 8"	2 LAYERS TOP BARS AT LINE 2
B 119	16 x 22½	4-#10	2-#11	—	#3	4", 3 @ 8", 3 @ 10"	2 LAYERS REINF.
B 120	16 x 18½	3-#7	—	2-#8 x 16'-0"	#3	3", 3 @ 6", 3 @ 8"	
B 121	8 x 14½	2-#6	—	2-#5	#2	6", REM @ 12"	
B 122	12 x 16	2-#6	—	2-#6 CONT.	—		BOLSTER BM. BEAR ON MASONRY
B 123	12 x 24	3-#7	—	2-#6 x 15'-0" NO. END 2-#8 x 10'-6" CTR ON LINE 3	#3	3, 2 @ 6", 6 @ 8" AT LINE 3	SPLICE BOT. BARS. SEE SEC K-S2
B 124	12 x 24+	3-#7	—	2-#8 x 26'-0"	#3	3", 2 @ 6", 6 @ 8"	SLOPE TOP & BOT. BARS W/ BM. (SEC. K)
B 125	8 x 15	2-#6	—	2-#6 CONT.	—		SEE SEC. K-S2
B 126	12 x 24	2-#8	2-#7	—	#3	5", 4 @ 10"	
B 127	12 x 38	3-#9	—	2-#6	—		ADD 1-#4 HOR EA. SIDE CTRD AND #4 x 5'-0" @ 12" THUS O.F. 2-2
B 128	12 x 38	3-#8	—	2-#6	—		
B 129	8 x 10½	2-#7	—	2-#5 x 12'-0"	—		
B 130	12 x 24+	3-#7	—	2-#6 x 16'-0"	#3	4", 5 @ 8" AT LINE 4	SLOPE TOP & BOT BARS (SEC. K-S2)
B 131	16 x 15¾/17	4-#6	—	2-#7 x 13'-0"	—		

NOTE: PROVIDE 2-#4 TOP STIRRUP TIE BARS IN BEAMS WITHOUT TOP BARS.

CONCRETE JOIST SCHEDULE

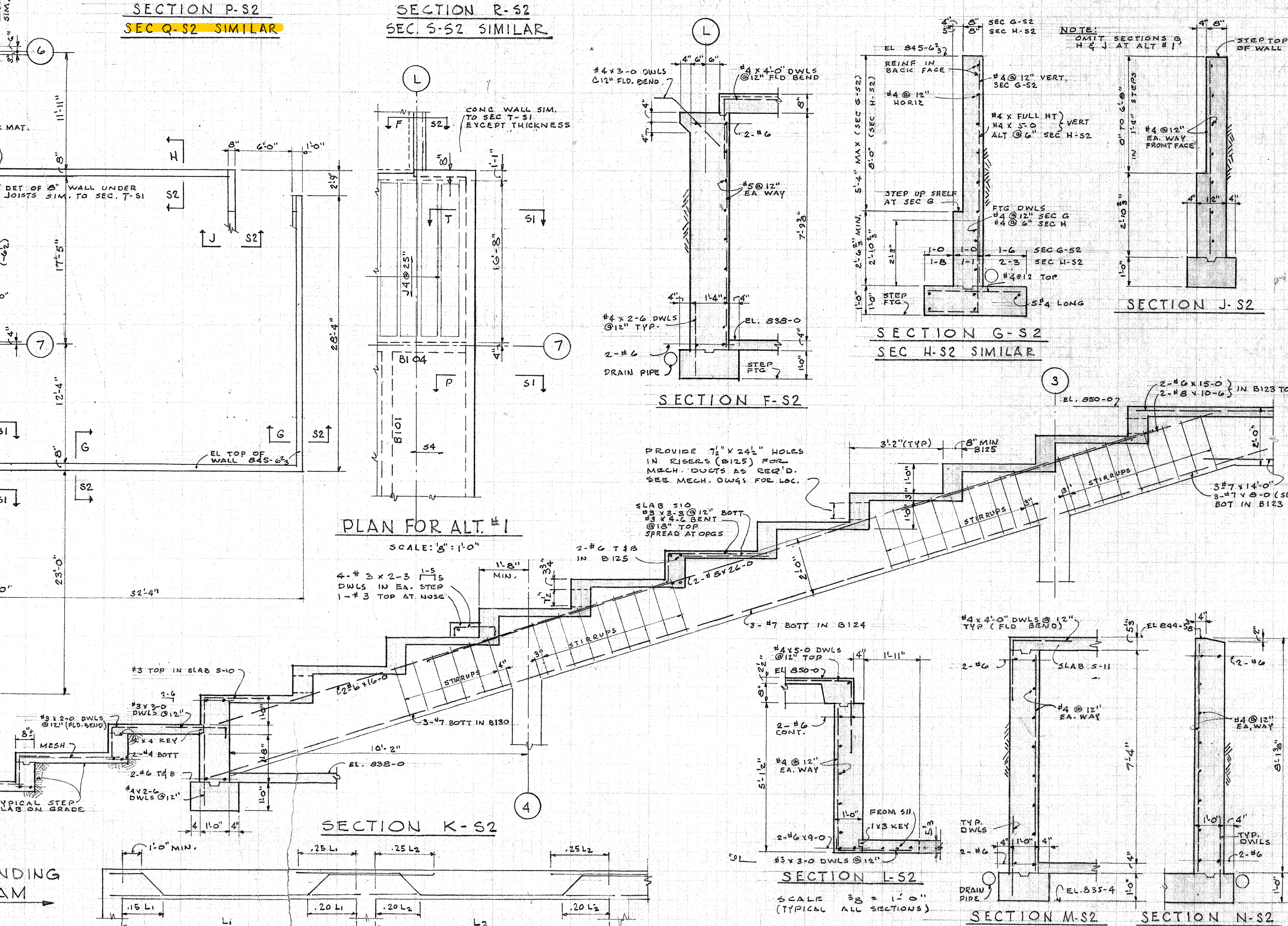
MARK	SIZE & SPACING	REINFORCING STEEL			REMARKS
		STRAIGHT	BENT	TOP	
J1	8 + 2½ @ 25"	2-#5	—	2-#4 x 4'-0" CTR ON LINE C & J	
J2	6 x 10½	2-#5	—	DO.	15" WIDE PANS EACH SIDE
J3	8 + 2½ @ 25"	2-#6	—	2-#4 x 6'-0" CTR ON LINE C & J	
J4	8 + 2½ @ 25"	1-#7	1-#6	—	EXT. TRUSS BAR 3-0 INTO ADJ. SLAB AT LINES 1, 7 & A EXCEPT AT STEPS
J5	6 x 10½	1-#8	1-#6	—	DO
J6	8 + 2½ @ 25"	1-#9	1-#7	—	
J7	18 x 10½"	4-#6	—	3-#6 x 16'-0" CTRD	
J8	9 x 10½	1-#9	2-#6	—	
J9	12 x 10½	3-#5	—	2-#6 x 16'-0" CTRD	

NOTE - TYPICAL WIDTH OF PAN FORMS TO BE 20" EXCEPT WHERE SHOWN OTHERWISE
" DEPTH " " " " " 8".

CONCRETE SLAB SCHEDULE

MARK	SIZE	REINFORCING STEEL			REMARKS
		BOTTOM	TOP	TEMP	
S1	4"	#3 @ 6" STRAIGHT	FROM WALLS OR BMS	#3 @ 12"	
S2	4"	#3 STRT, #4 BENT ALT @ 8"	—	DO.	
S3	4"	#3 STRT, #5 BENT ALT @ 8"	—	DO.	EXT. TRUSS BARS 2'-10" PAST B112 INTO S4
S4	4½"	#5 @ 8", ALT. BENT	—	DO.	EXT. TRUSS BARS 3'-0" INTO JOIST SLAB PAST BRG WALL BTWN COL LINE 6 & 7. FROM COL. LINES B TO K (SAME AT SLAB S9)
S5	4½"	#5 @ 6" ALT. BENT	—	DO.	
S6	4"	#3 STRT, #4 BENT ALT @ 6"	—	DO.	
S7	4"	#4 @ 8" STRAIGHT	#4 x 5'-0" @ 12" EA. END	DO.	OFFSET TOP BAR OVER WALL 1½"
S8	8"	#4 @ 6" ALT. BENT (SEE REM.)	1-#5 x 12-0 AT LDG. EDGE	#4 @ 12"	2-#6 x 11'-6" BOTT @ 3" AT EDGE OF LDG.
S9	6½"	#5 @ 8" ALT. BENT	DO	DO	EXT. TRUSS BARS AS IN SLAB S4. DO
S10	3"	#3 @ 12" STRAIGHT	#3 @ 18" x 4-6	#3 @ 12"	BEND DOWN TOP BARS 1'-1" (SEC. K-S2)
S11	5½"	#4 @ 6" STRAIGHT	FROM WALL	#4 @ 12"	

NOTE: PROVIDE AT LEAST TWO ROWS OF CONT. HIGH CHAIRS OR #4 TOP SUPPORT BARS IN EACH BAY TO SUPPORT TRUSS OR TOP BARS.



SECTION F-S2

SECTION G-S2
SEC H-S2 SIMILAR

SECTION J-S2

PLAN FOR ALT. #1
SCALE: ⅛" = 1'-0"

SECTION K-S2

BAR BENDING DIAGRAM

SECTION L-S2
SCALE ½" = 1'-0"
(TYPICAL ALL SECTIONS)

SECTION M-S2

SECTION N-S2

THE UNIVERSITY OF MICHIGAN
OFFICE OF THE UNIVERSITY ARCHITECT • ANN ARBOR, MICHIGAN
CHRYSLER CENTER
FOR CONTINUING ENGINEERING EDUCATION

SWANSON ASSOCIATES, INC.
ARCHITECTS ENGINEERS
WEST LONG LAKE ROAD, BLOOMFIELD HILLS, MICHIGAN

SHEET TITLE
UPPER LEVEL FRAMING PLAN, BEAM & SLAB SCHEDULES

BULL.	DATE	BULL.	DATE

DES.
DRWN. RR & KP
CHKD. KP
APPVD. PE
DATE

DATE 5-10-66 PRELIMINARY
DATE 6-15-66 CONSTRUCTION
DATE 1-5-68 FINAL RECORD

U. OF M. PROJECT NO. 264

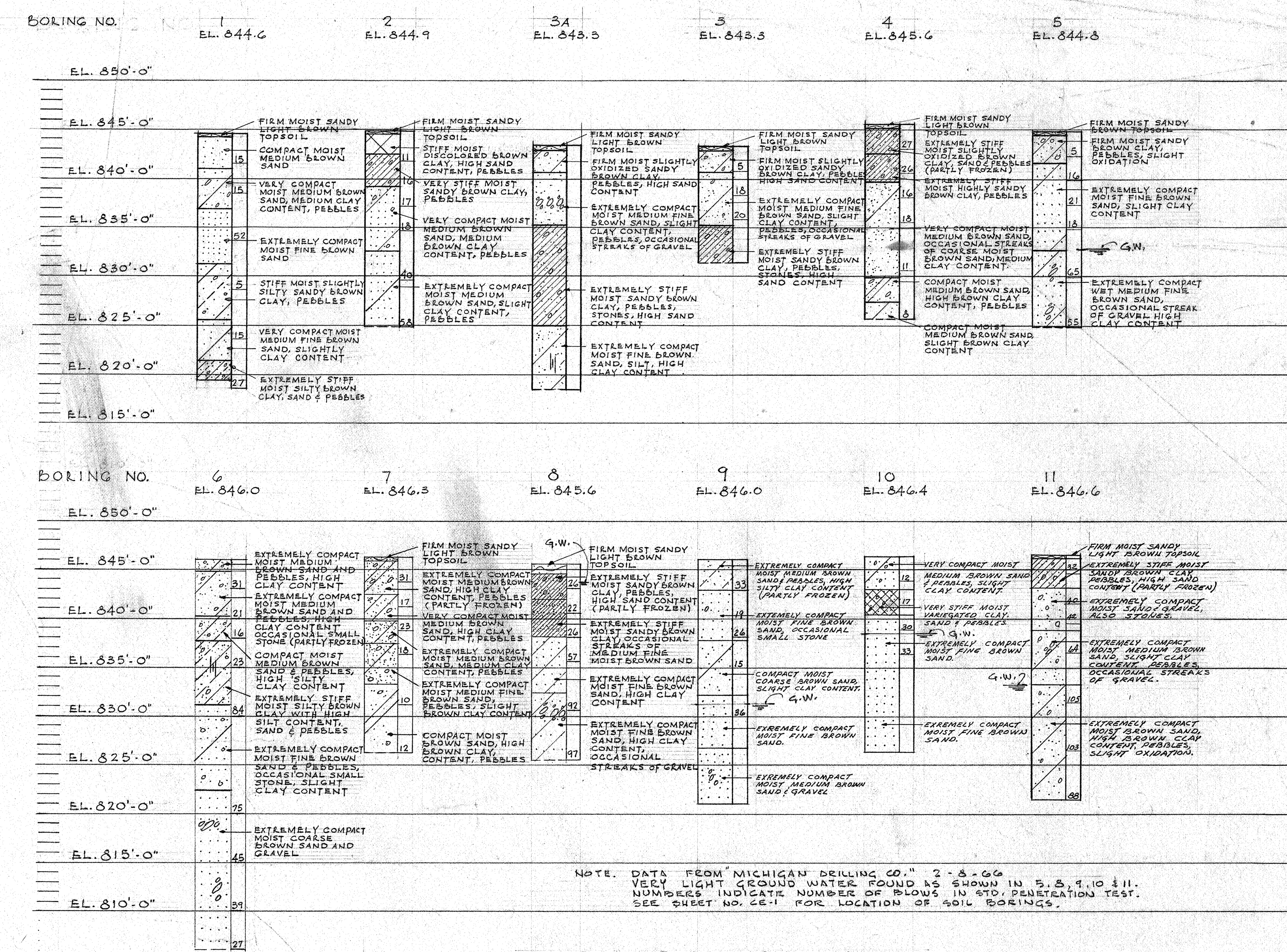
ARCHITECT'S JOB NO. 6508

SHEET NO. S-2

#443

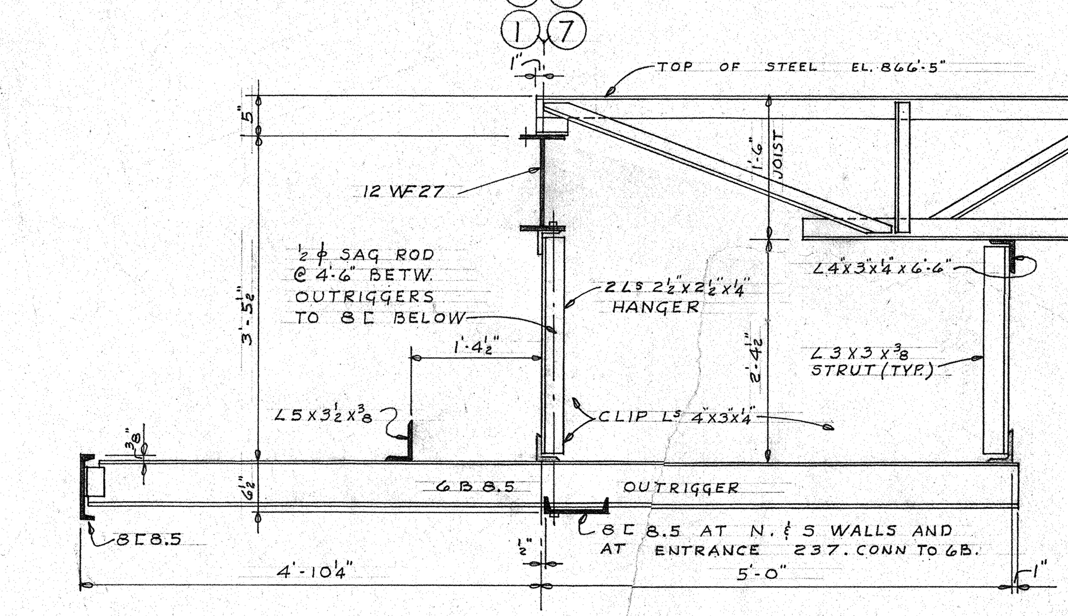
BLUE PRINT PAPER CO. 250 12-65

BORING NO.

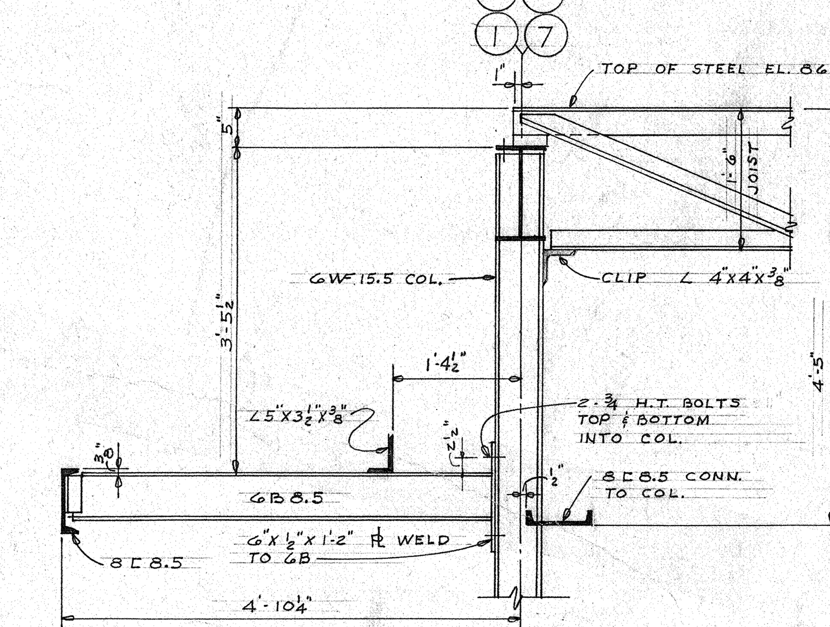


NOTE: DATA FROM MICHIGAN DRILLING CO. 2-8-66
VERY LIGHT GROUND WATER FOUND AS SHOWN IN 5, 8, 9, 10 & 11.
NUMBERS INDICATE NUMBER OF BLOWS IN STD. PENETRATION TEST.
SEE SHEET NO. CE-1 FOR LOCATION OF SOIL BORINGS.

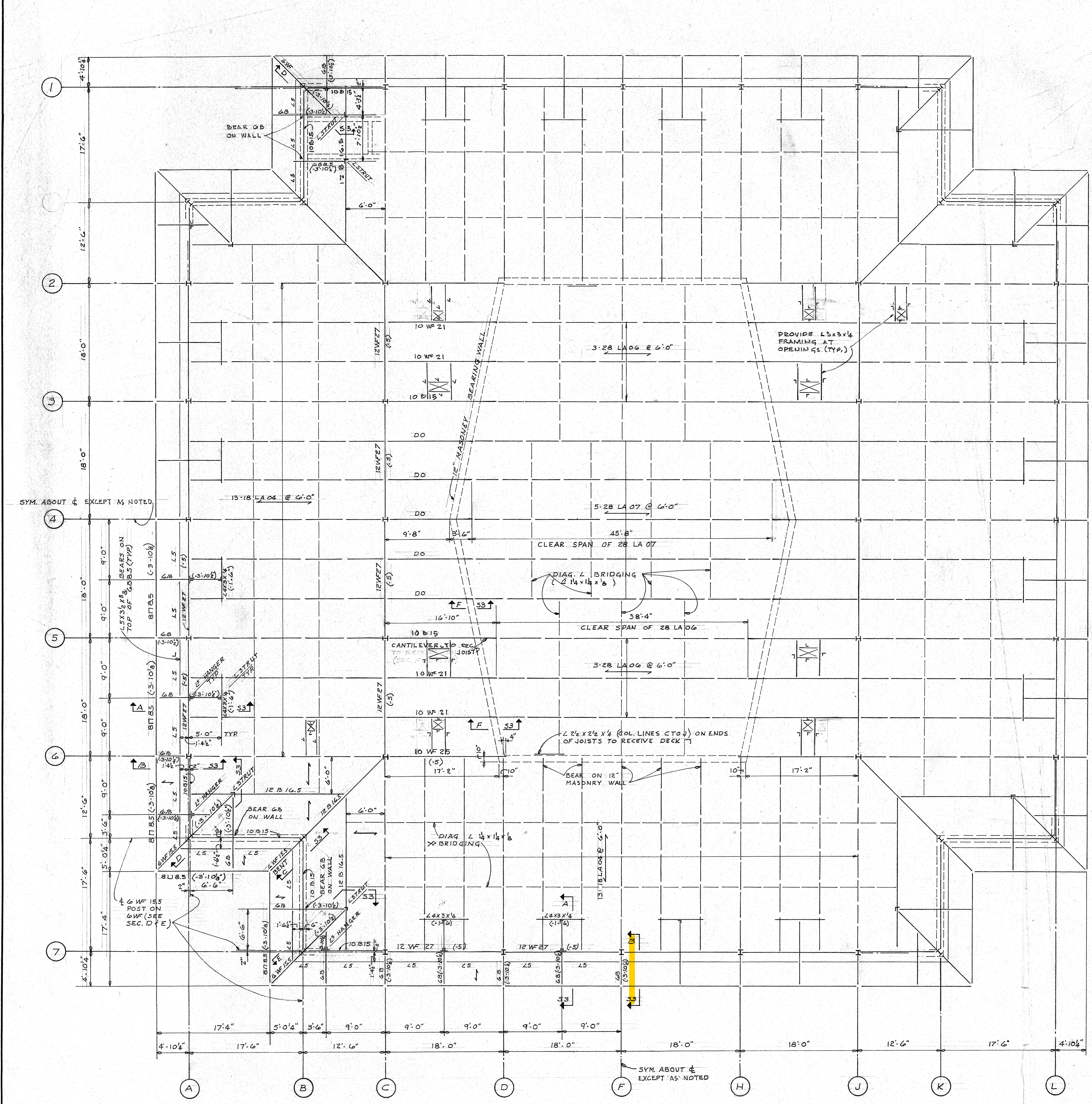
LOG OF SOIL BORINGS
VERT. SCALE: 1/8" = 1'-0"



SECTION A-S3 (TYP. BETW. COLS.)
SCALE: 3/4" = 1'-0"

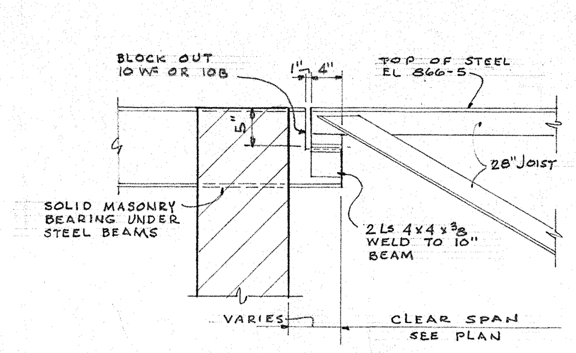


SECTION B-S3 (TYP. AT COLS.)
SCALE: 3/4" = 1'-0"

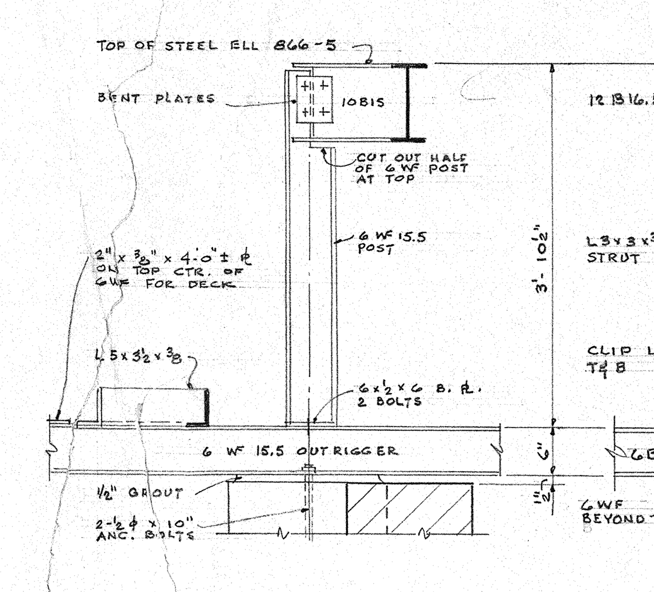


NOTE: TOP OF STEEL AT EL. 866'-5" UNLESS NOTED OTHERWISE MEASURED THUS (±) FROM SAME ELEV.
PROVIDE 1½" DEEP #20 GAGE STEEL DECK (EQUAL TO "MAHON QUAD-RIB") OVER ENTIRE ROOF.
PROVIDE CLIP L^s 4X4X3/8 ON COLUMNS, EXTEND BOTTOM CHORD OF JOISTS & WELD TO SAME.
FOR FURTHER DETAIL SEE ARCH. DWGS. (6B = 6B8.5 U.N.)

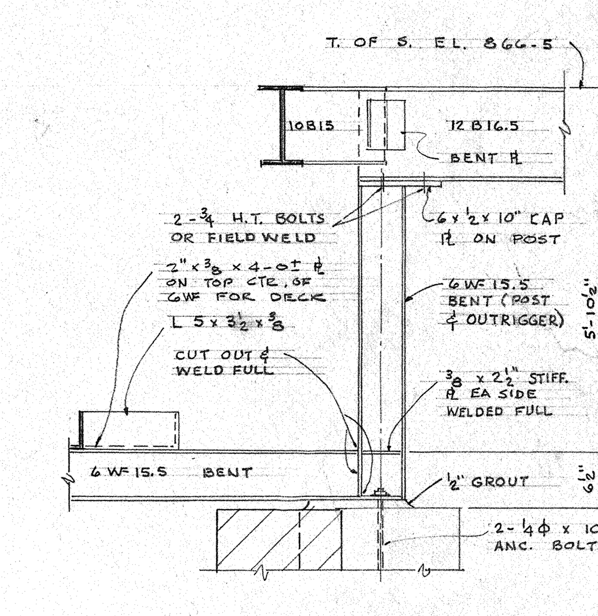
ROOF FRAMING PLAN
SCALE: 1/8" = 1'-0"



SECTION F-S3
SCALE: 3/4" = 1'-0"



SECTION D-S3
SEC. E-S3 OPP. HAND



SECTION C-S3

#443

THE UNIVERSITY OF MICHIGAN
OFFICE OF THE UNIVERSITY ARCHITECT • ANN ARBOR, MICHIGAN
CHRYSLER CENTER
FOR CONTINUING ENGINEERING EDUCATION

SWANSON ASSOCIATES, INC.
ARCHITECTS ENGINEERS
WEST LONG LAKE ROAD, BLOOMFIELD HILLS, MICHIGAN

SHEET TITLE: ROOF FRAMING PLAN & DETAILS
SOIL BORING DATA

DATE		DES.	
PRELIMINARY	5-10-66	DRWN.	JB & RR
CONSTRUCTION	6-15-66	CHKD.	GP
FINAL RECORD	1-31-68	APPVD.	DE

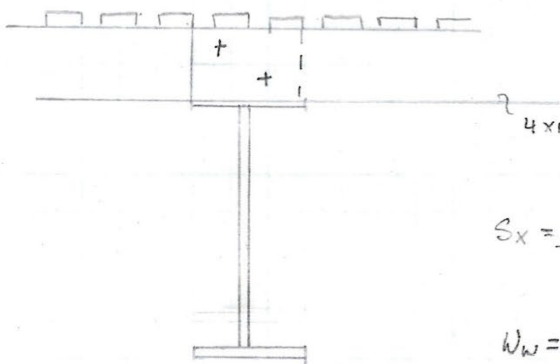
U. OF M. PROJECT NO. 264

ARCHITECT'S JOB NO. 6508

SHEET NO. S-3

Appendix B: Wood Beam Capacity Calculation

Wood beams capacity check (20' span)



4x12 @ 16" o.c.

$$S_x = \frac{bd^2}{6} = \frac{(3.5)(11.5)^2}{6} = 77 \text{ in}^3$$

$$W_w = \frac{q_v \cdot}{144 \text{ in}^2/\text{ft}^2} \times 16'' = 13.3 \text{ lb/in}$$

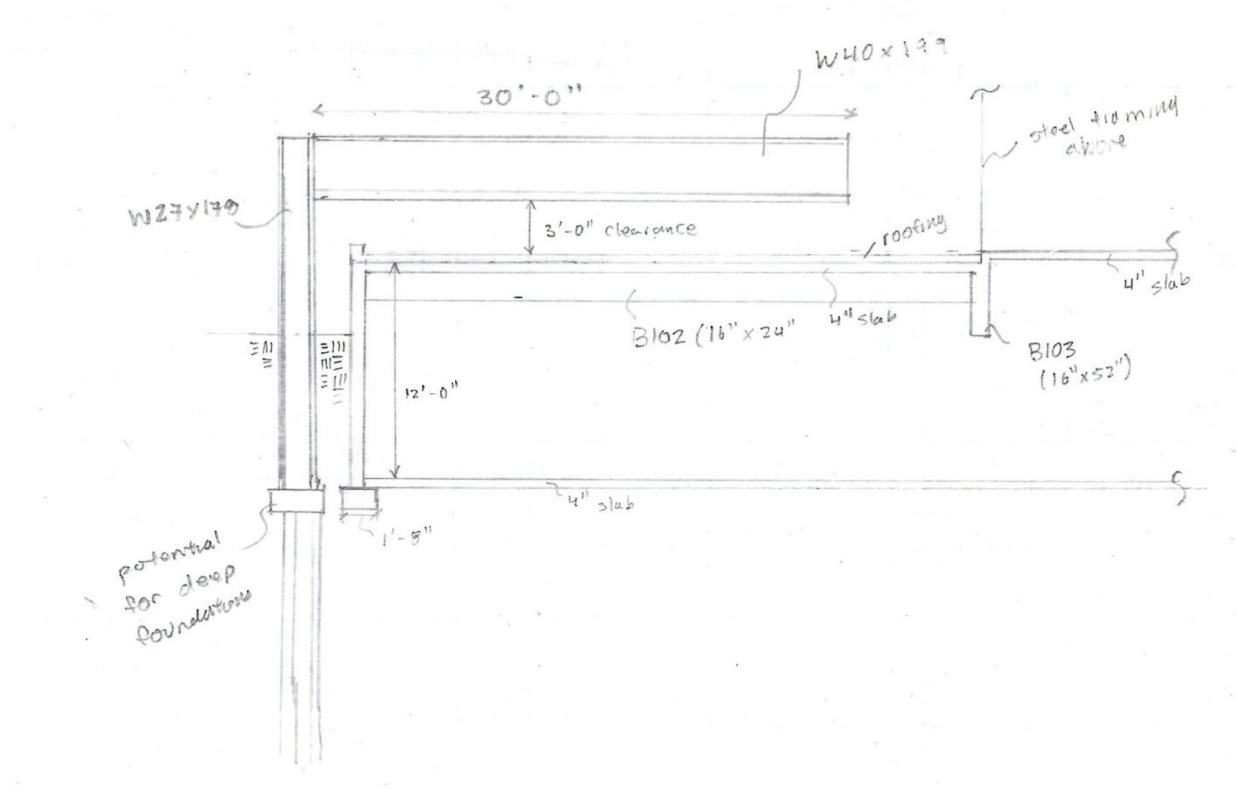
$$M = \frac{W_w L^2}{8} = \frac{13.3 \text{ lb/in } (20' \times 12'')^2}{8} = 96{,}000 \text{ lb-in}$$

$$f_t = \frac{M}{S_x} = \frac{96{,}000 \text{ lb-in}}{77 \text{ in}^3} = 1244 \text{ psi}$$

in range of
wood capacity
(1,000 - 2,000 psi) ✓

Appendix C: Cantilever Design Calculations

Figure C1: Cantilever Design Section View



C.1 Deflection Calculations

- assume deflections will govern

<u>Δ_{max} based on vibration frequency</u>

$f = \frac{1}{2\pi}\left(\frac{g}{\delta_o}\right)^{1/2}$ $g = 384$ in/s^2 natural frequency of walking human is 2 Hz

δ_o	f(Hz)	1/f (s)	
1"	3.11	0.32s	→ sufficiently for from 2Hz
0.75"	3.60	0.28s	
0.5"	4.41	0.23s	
0.25"	6.24	0.16s	∴ Δ_{max} = 1"

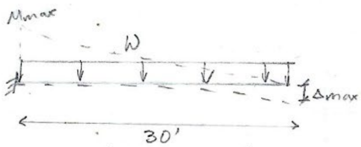
C.2 Girder Selection Calculation

Beam selection

$$\Delta max = \frac{\omega L^4}{8EI} \longrightarrow I \geq \frac{\omega L^4}{8E\Delta max}$$

$W = 120psf \times 20ft = 2.4\ klf = 0.2\ kli$ $\quad E = 29{,}000\ psi$

$L = 30' = 360''$



$$I(\Delta_{max}) \geq \frac{(0.2\,kli)(360\,in)^4}{8 \cdot 29000\,ksi\,\Delta_{max}} = \frac{14{,}479}{\Delta_{max}}$$

$I(1'') \geq 14{,}479\,in^4 \rightarrow (W40 \times 199)$

① large section

C.4 Column Selection Calculation

Column Selection

$$M max = \frac{\omega_f \times L^2}{2}$$

$\omega_f = q_f \times 20' = 3.68\ klf = 0.307\ kli$

$L = 30' = 360in$

$(M_r)\ M max = \frac{0.307\,kli\,(360\,in)^2}{2} = 19{,}872\ K\text{-}in\ \ (1656\ K\text{-}ft)$

$(P_r)\ P = 20' \times 30' \times 184\ psf = 110{,}400\,lb = 110\ K$

$k_y = 1.2$ (assumed from fixed - free translation)

$kL_y = 24'$ $\rightarrow$ table 6-2 $\rightarrow$ W27×178

Unbraced length $- L_b = 20'$

interaction check

$P_r = 110k \qquad P_c = 1590\ K$

$$\frac{P_r}{P_c} = 0.064 < 0.2$$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}}\right) \leq 1.0$$

$M_{rx} = 1656\ K\text{-}ft \qquad M_{cx} = 1730\ K\text{-}ft$

$$\frac{110K}{2 \cdot 1590K} + \left(\frac{1656\ K\text{-}ft}{1730\ K\text{-}ft}\right) = 0.99 \leq 1.0$$

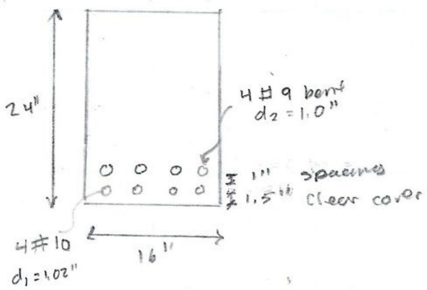
1

Appendix D: Existing Structural Capacity Calculations

D.1 Beam Flexural Capacity

Existing Strength

B102 @ center span



$A_s = 5.08 + 4.00 = 9.08\ in^2$

find centroid of steel

$$d = \frac{(24 - (1.5 + 0.51)) \times 5.08 + (24 - (1.5 + 1.02 + 1 + 0.5)) \times 4.00}{9.08}$$

$d \approx 21.1"$

$bd = 338\ in^2 \quad bd^2 = 7126\ in^3$

$$M_u = \frac{wL^2}{8}$$

$L = 36'$

$w = 1.2(60psf \cdot 9') + 1.6(30psf \cdot 9') = 1.08\ klf$

weight of concrete

tributary width

roof live load from gerlad notes

$$M_u = \frac{(1.08\ klf)(36')^2}{8} = 175\ k\text{-}ft$$

$$R_u = \frac{M_u}{bd^2} = \frac{175\ k\text{-}ft}{7126\ in^3} \approx 0.025$$ → John McCarthy design aid

for $f'_c = 3{,}000\ psi$, $f_y = 60\ ksi$

→ $\rho_{req} = 0.00896$

$A_{s,req} = \rho A_c = 0.00598 \cdot (24" \times 16") = 2.95\ in^2 \ll 9.08\ in^2$

→ beam has sufficient flexural capacity to add load

D.2 Existing Shear Capacity

Shear capacity

stirrups #4 5", 9@ 10"

$\phi V_n \geq V_u$

$\phi V_n = \phi V_c + \phi V_s$

$$\phi V_c = 0.75 \times 2 \frac{\sqrt{f'_c}}{1000} \times bd = 0.75 \cdot 2 \frac{\sqrt{3{,}000\ psi}}{1000\ K/lb} \times 16 \times 21.1 = 28K$$

$$\phi V_s = \frac{\phi A_v f_{yt} d}{s} = \frac{0.75(0.40 in^2)(40 ksi)(21.1")}{s} = \frac{253.2}{s}\ K\text{-}in$$

$$\phi V_s = \frac{253.2\ K\text{-}in}{10\ in} = 25\ K$$

$\phi V_n = 28 + 25 = 53K$

→ extra shear capacity in stirrups region.

D.3 Basement Wall Axial Capacity

Wall Axial capacity

- 12" x 12" column per foot

$h = 1'$ $h^w = 12'$

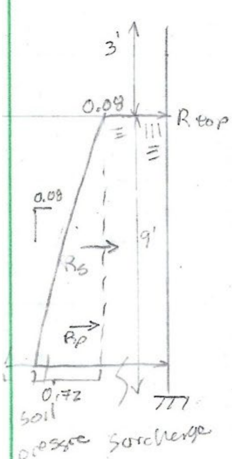
$$\phi P_n = 0.55\, \phi f_c'\, A_g \left[1 - \left(\frac{K l_c}{32 h} \right)^2 \right]$$

$$= 0.55\,(0.65)\,(3000\text{ psi})\,(12'' \times 12'') \left[1 - \left(\frac{0.8 \cdot 9' \cdot 12''}{32 \cdot 12''} \right)^2 \right]$$

= 146 K/ft length

D.4 Lateral Load on Beam from Soil Pressure

Lateral Wall capacity



typ $\gamma = 100$
$K_o = 0.5$

surcharge = 100 psf · 0.5 = 50 psf

$\rho = 100 \times 0.5 = 50$ psf/ft

$S = 0.05 \times 1.6 = 0.08$ Ksf
$\rho = 0.05 \times 1.6 = 0.08$ ksf

$R_s = 0.08 \times 9' = 0.72$ K
$R_p = \frac{1}{2} \cdot 9' \cdot 0.72 = 3.24$ K

$$R_{top} = \frac{R_p \cdot 9'/3 + R_s \cdot 9'/2}{12'} = 1.08 \text{ K/ft}$$

axial force on B102 = 1.08 K/ft × 9' = 9.7 K ok
↳ tension (shear reduction)

D.5 Bearing Capacity

load on wall concrete = 150 lb/cf (max)

floor system = slab + beam

$W_F = (50\text{ psf} + 30\text{ psf}) \times 18 = 1.4$ K/ft
slab beam (spread)

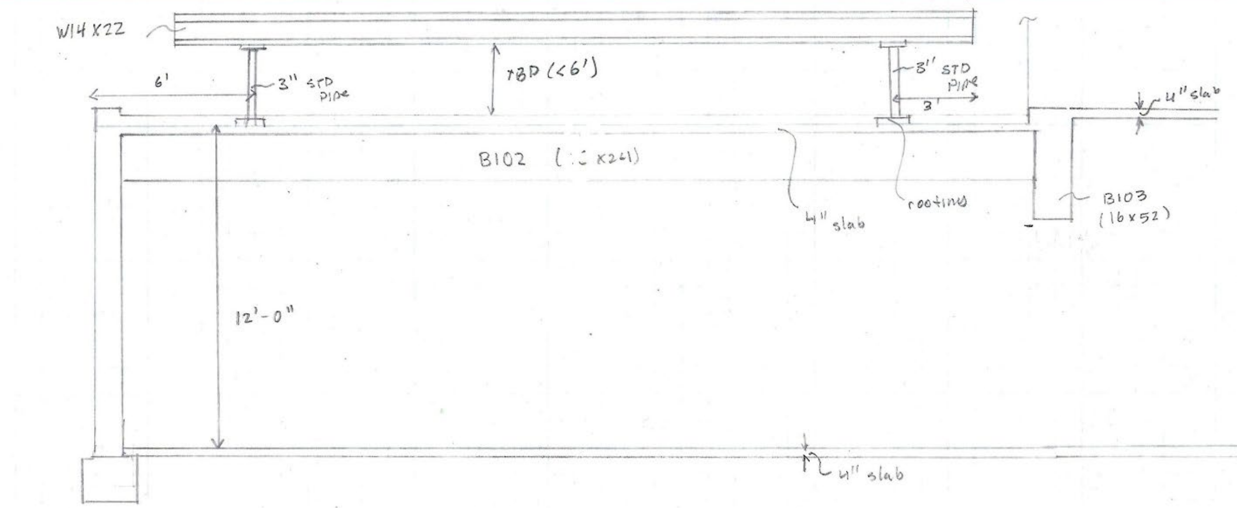
$W_{LL} = 30\text{psf} \times 18 = 0.540$ K/ft
$W_{wall} = 12.0' \times 1' \times 150$ lb/cf $= 1.8$ K/ft

$W = 3.7$ K/ft

Bearing Capacity
~ 2' × 4000 psf = 8000 lb/ft (8 K/ft)
3.7 K used, deck adds ~ 2.5 K ok

Appendix E: Existing-Structure-Supported Design Calculations

Figure E1: Existing-Structure-Supported Design Section View



E.1 Wood Filler Beam Design

Wood beams (9' span)

$$M = \frac{w_w L^2}{8} = \frac{13.3\ \text{lb/in}\ (9' \times 12''/1')^2}{8} = 19,391\ \text{lb in}$$

$$f_t = \frac{19,391\ \text{lb} \cdot \text{in}}{77\ \text{in}^3} = 252\ \text{psi} \quad (\sim 1/4\ \text{capacity of } 4 \times 12)$$

$$2 \times 12 \rightarrow S_x = \frac{(1.5)(11.5)^2}{6} = 33\ \text{in}^3$$

$f_t = 587$ psi ($\sim 1/2$ capacity) ✓

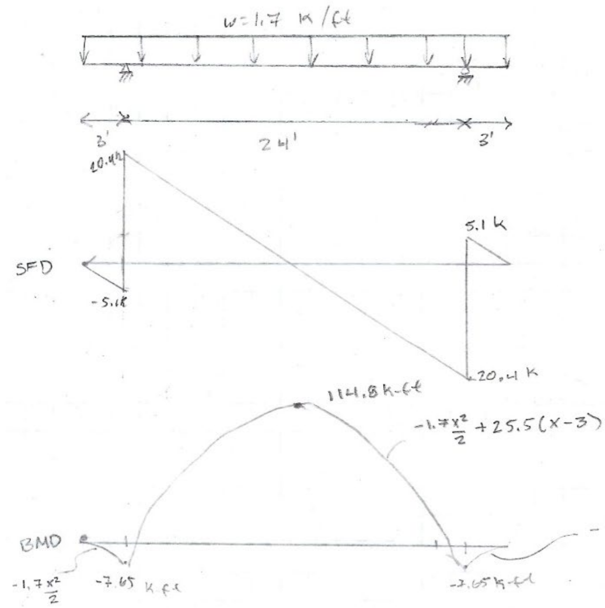
$2 \times 10 \rightarrow S_x = 22.6\ \text{in}^3 \rightarrow f_t = 859$

E.2 Steel Girder Design

beams @ 9' oc   length = 30'

$w = \frac{95 \cdot 9'}{1000^{lb}/k} = 1.7\ k/ft$

consider fully braced by wood ~~deck~~ beams on top flange



$-1.7\frac{x^2}{2} + 25.5(x-3) + 25.5(x-27)$

required capacity $M_U = M_{max} = 115$ k-ft

$Z_{req} = \frac{M_U}{\phi F_y} = \frac{(115\ k\text{-}ft)(12in/ft)}{0.9 \cdot 50\ ksi}$ 31 in^3 Table 3-2

W14×22 $Z = 33.2\ in^3$

$M_{U(self)} = 1.2\left(0.022 \cdot \frac{(15)^2}{2} + 0.33(15-3)\right) = 6.4$ k-ft

$M_U = 121.2$ (self weight included

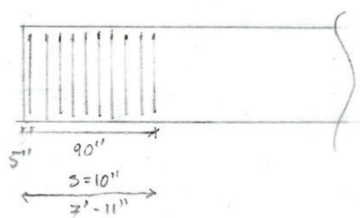
$Z_{req} = \frac{121.2\ k\text{-}ft \cdot 12in/ft}{0.9 \cdot 50\ ksi} = 32.3 in^3$ ok

E.3 Vertical Pipe Design and Placement

<u>Pipes</u> ~ 2-3'

$A_g = \frac{P_u}{F_y} = \frac{25.5\ K}{35\ ksi} = 0.73 in^2 \Rightarrow$ Pipe 3
$\phi_c P_n \geq 53.7$ (6ft or less)
$A_g = 2.07 in^2$

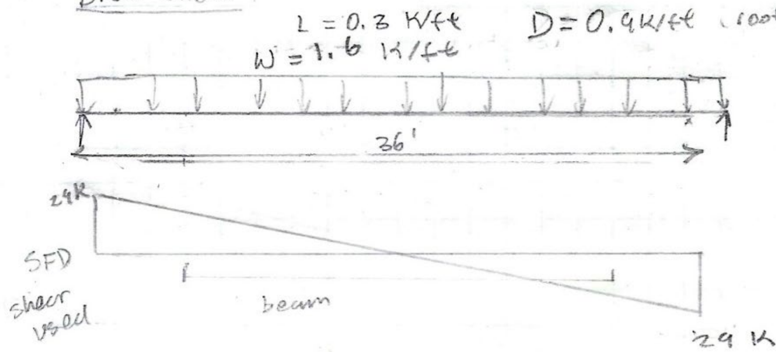
Pipe Placement



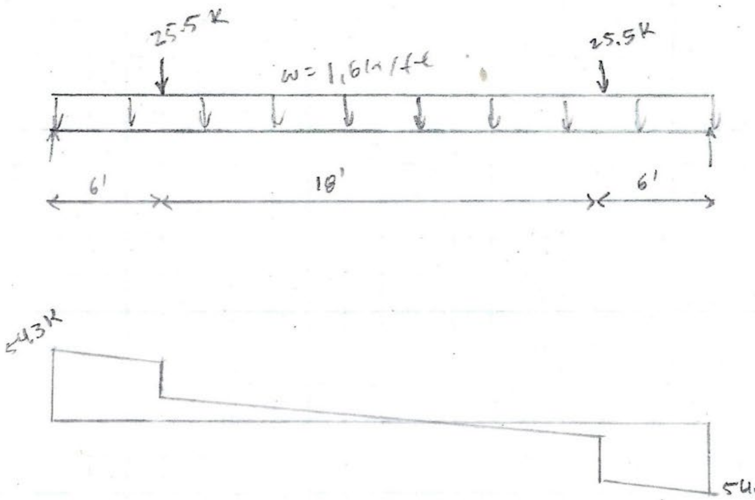
→ place pipe @ 6' from end of B102 (sufficient shear capacity) ✓

E.4 Shear Loading Envelope of Existing Building

B102 Shear



E.5 Shear Loading Envelope with New Structure Loading



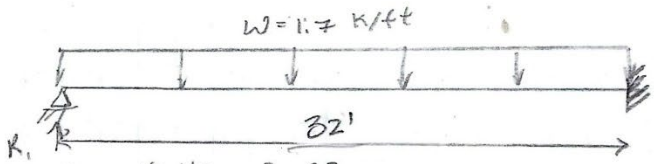
54.3K → slightly over capacity
↓
20' spans will definitely be over

Appendix F: Hybrid Design Calculations

F.1 Steel Girder Design

beams @ 9' oc

assume fully laterally braced



Per table 3-23

$$M_{max} = \frac{wl^2}{8} = \frac{1.7\ K/ft\ (32ft)^2}{8} = 212\ K\text{-}ft$$

$$Z_{req} = \frac{M_u}{\phi F_y} = \frac{(212\ k\text{-}ft)(12\ in/ft)}{0.9 \cdot 50\ ksi} = 57\ in^3$$

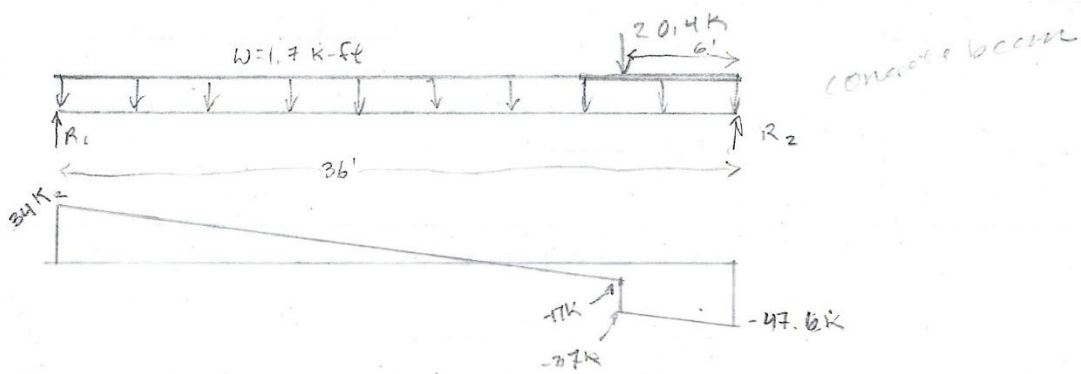
W18X35

$Z = 64\ in^3$

$$M_{self} = 1.2(0.035)\frac{32^2}{8} = 5.4\ k\text{-}ft$$

$Z_{req} = 58\ in^3$ ok

$$V_1 = \frac{3wl}{8} = \frac{3(1.7\ k/ft)(32')}{8} = 20.4k$$



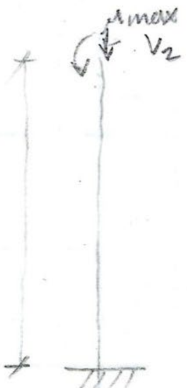
F.2 Steel Pipe Design

Pipe

$$A_g = \frac{P_u}{F_y} = \frac{20.4K}{35} < 1\ in^2 \rightarrow \text{Pipe 3 std } 53k \geq (6'<)$$

$A_g = 2.07$

F.3 Steel Column Design



$V_2 = \frac{5wl}{8} = \frac{5(1.7\ K/ft)(32')}{8} = 34\ K$

$M_{max} = 212\ k\text{-}ft + 5.4 = 217.4\ k\text{-}ft$

$K = 1.2$ (fixed free)

$L = 12' + 3/2' + 1/2' = 14' \rightarrow 16'$

crawl space (crawl only)

$KL_y = 19' \rightarrow$ Table 6-2

W10 × 60

$\Phi_b M_{ny} = 242$
$\Phi_c P_n = 448$

$\frac{P_r}{P_c} = \frac{34\ k}{448\ k} = 0.08 < 0.2$

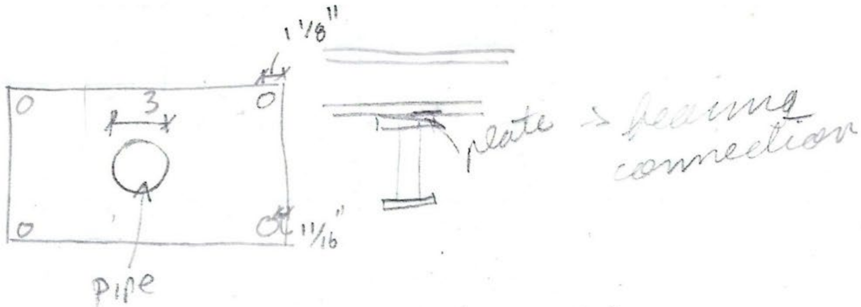
$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}}\right) \leq 1.0$

$\frac{34\ k}{2(448\ k)} + \frac{(222.5\ k\text{-}ft)}{242\ k\text{-}ft} = 0.96 < 1$ OK

Appendix G: Bearing Connection Design Calculations

G.1 Bearing Plate Design (Iteration 1)

- Pipe connections



$$M_n = F_y Z = F_y \left(\frac{1.0\, t_p^2}{4} \right)$$

W16x30 A992 (50 ksi) pipe - 3" diameter

$b_f = 5.53$, $k_{des} = 0.842$ in

plate 6" x 6" plate? A 36 - 36 ksi

reaction $R_u = 34$ K

① pressure under plate

$$f_u = \frac{34\text{ k}}{(6\text{ in})(6\text{ in})} = 0.94\text{ ksi}$$

② cantilever dimension

$$l = n = \frac{b}{2} - k_{des} = 3 - 0.842 = 2.158''$$

③ t_p min

$$t_p = 1.49\, l \sqrt{\frac{f_u}{F_y}} = 1.49\,(2.158'')\sqrt{\frac{0.94\text{ ksi}}{36\text{ ksi}}} = 0.521''$$

④ $t_p = 5/8''$

pipe 3" std (35ksi) punching?

$A_g = 2.07$ $d_{outer} = 3.50''$

6x6 plate $R_U = 34k$

$$f_U = \frac{34\ K}{(6in)(6in)} \Rightarrow 0.94\ ksi$$

$l = n = >$ options? (edge = min?, corner = max)

$$n = \frac{6}{2} - \frac{d_{outer}}{2} = 3 - 1.75 = 1.25''$$

$$n = \frac{\sqrt{2\,6^2}}{2} - \frac{d_{outer}}{2} = \frac{\sqrt{2}}{2}\,6 - 1.75 = 2.49''$$

$$t_p = 1.49\,(2.49'')\sqrt{\frac{0.94ksi}{36ksi}} = 0.601\ in \rightarrow 5/8''\ \text{plate ok}$$

G.2 Bearing Plate Bolt Design (Iteration 1)

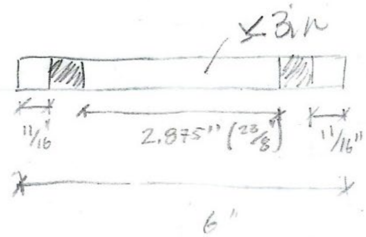
$\boxed{\text{bolts}}$ A − 325 N or X? 7/8"?

· tension as primary force

min edge distance = 1 1/8" (c to e)

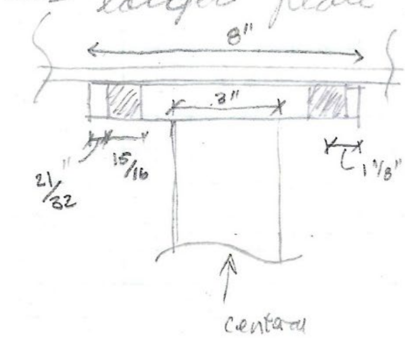
edge distance = 11/16" ?

shear



G.3 Bearing Plate Design (Iteration 2)

→ larger plate required ~~6x6"~~ 6" x 8"



$$f_U = \frac{34K}{(6in)(8in)} = 0.708ksi$$

$$l = n = \frac{6}{2} - 0.842 = 2.158''$$

$$t_p = 1.49\,(2.158'')\sqrt{\frac{0.708ksi}{36\ ksi}} = 0.451''$$

$\rightarrow t_p = 0.5'' = 1/2''$

A_{pe}

$f_v = 0.708$ ksi

$l_{max}? = \frac{\sqrt{6^2+8^2}}{2} - \frac{3.5}{2} = 3.25''$

$t_p = 1.49\,(3.25'')\sqrt{\frac{0.708\text{ ksi}}{36\text{ ksi}}} = 0.679'' \Rightarrow t_p = 3/4''$

G.4 Bearing Plate Bolt Design (Iteration 2)

bolts   325 - N/X   7/8" diameter

Shear $R_n = F_n A_b\ 4$

$F_{nv} = 54$ ksi $A_b = 0.601\ in^2$

$r_n = 32.4$ K

bearing / tear out $l_c = \frac{21}{32}''$

$R_n = 0.6\,(120\text{ ksi})\left(2 \cdot \frac{21}{32}''\right) \cdot \left(\frac{3}{4}''\right) = 70.9$ K

tension

$R_n = F_n A_b = 90 \cdot 0.601 = 54.1$ K

$\Rightarrow R_n = 32.4$ K

$R_n = 4 r_n = 129.6$ K

$\phi R_n = 97.2$ K   (shear controlled)   ok?

G.5 Bearing Plate Weld Design

- loading transverse to load direction

$\phi R_n = F_{nw} A_{we}$ $F_{exx} = 70$ksi   1/16" weld

$\phi R_n = 0.75\,(0.6 \cdot 70\text{ ksi})\,(0.707 \cdot 1/16''\,(2\pi\,(3.5''/2)))$

$= 2.78$ K / 1/16" weld

$\rightarrow$ 3/16" weld $\rightarrow$ 33 K

transverse

$1.5\,\phi R_n = 50.1$ K

Appendix H: Moment Connection Design Calculations

H.1 Bolted Flange Plate Design (Iteration 1)

Moment Connection

W 18 × 35 beam ~~column~~ $f_y = 50ksi$, $f_u = 65ksi$ ~~$V_u = -47.6K$~~ ~~$M_u =$~~ Plates A36 $f_y = 36ksi$ $f_u = 58ksi$

W10 × 60 column

$V_u = V_2 = \frac{5}{8} w\ell = \frac{5}{8}(1.7 \text{ K/ft})(32') = 34 \text{ K}$

$M_{max} = \frac{w\ell^2}{8} = \frac{(1.7 \text{ K/ft})(32')^2}{8} = 218 \text{ K-ft}$

beam flexural strength

$A_{fg} = b_f t_f = (6.00'')(0.425 \text{ in}) = 2.55 \text{ in}^2$

net area of beam flange ($d_h = 15/16''$ → $7/8''$ bolt)

$A_{fn} = A_{fg} - (2 \text{ bolts})(d_h + 1/16) t_f$

$A_{fn} = 2.55 \text{ in}^2 - (2)(1'')(0.425'') = 1.7 \text{ in}^2$

$\frac{F_y}{F_u} = \frac{50 \text{ Ksi}}{65 \text{ Ksi}} = 0.769 < 0.8 \Rightarrow Y_t = 1.0$

$F_u A_{fn} = (65 \text{ Ksi})(1.7 \text{ in}^2) = 111 \text{ K}$

$Y_t F_y A_{fg} = 1.0 (50 \text{ ksi})(2.55 \text{ in}^2) = 128 \text{ K} > 111 \text{ K}$

$\therefore M_n$ @ holes in tension flange $\leq \frac{F_u A_{fn}}{A_{fg}} S_x$

$\phi M_n = 0.9 \left(\frac{111 \text{ k}}{2.55 \text{ in}^2}\right)(57.6 \text{ in}^3) = \left(\frac{1 \text{ ft}}{12 \text{ in}}\right) = 188 \text{ K-ft} < 218 \text{ K-ft}$

H.2 Welded Flange Plate Design (Iteration 2)

Moment Connection

Beam : W18 × 35

$d = 17.7\,in \quad b_f = 6.00\,in \quad t_w = 0.300\,in \quad t_f = 0.425\,in$

Column : W10 × 60

$d = 10.2\,in \quad b_f = 10.1\,in \quad t_w = 0.420\,in \quad t_f = 0.680\,in$

① Force carried by flange

* assume moment arm = d

$$P_u = \frac{M_u}{d} = \frac{217.4\ K\text{-}ft\ (12in/ft)}{10.2\ in} = 255\ K$$

H.2.1 Tension Plate Design

② minimum plate area based on yielding of tension flange

$$A_p = \frac{P_u}{\phi F_y} = \frac{255\ K}{0.9\ (36\ ksi)} = 7.88\ in^2$$

top flange $b < b_f - 1$ (beam)

$b_f - 1 = 6.00 - 1 = 5.00$ in

$A_p = 5'' \times 1\ 5/8'' = 8.13\ in^2$

③ check for tension rupture (Table D3.1)

$$U = \frac{3\ell^2}{3\ell^2 + w^2}\left(1 - \frac{\bar{x}}{\ell}\right) \qquad \bar{x} = 0 + \frac{t_{plate}}{2} = 13/16''$$

* assume $\ell = 5.00$ in to start $\qquad \bar{x} = 2.39$

$$U = \frac{3\ (5.00\ in)^2}{3(5.00\ in)^2 + (5.00\ in)^2}\left(1 - \frac{13/16''}{5.00''}\right) = 0.628$$

$A_e = A_n U \quad (A_n = A_p) \qquad$ (D3-1)

$A_e = 8.13\ in^2\ (0.628) \Rightarrow 5.11\ in^2$

$P_n = F_u A_e \quad$ (J4-2)

$P_n = 58\ ksi \cdot 5.11\ in^2 = 296.2\ K$

$\phi P_n = 0.75 P_n > 222\ K \not> 255\ K$

⚠ weld length is insufficient

try $\ell = 6.00$ in

$U = 0.702 \qquad A_e = 5.71\ in^2 \qquad \phi P_n = 248\ K < 255\ K$

try $\ell = 7.00$ in

$U = 0.755 \qquad A_e = 6.14\ in^2 \qquad \phi P_n = 267\ K > 255\ K$ ok

$\Rightarrow$ use $\ell = 7.00$ in. welds = minimum tension rupture length

⑤ weld size based on weld rupture

min fillet weld for 1 5/8" plate and $t_f = 0.425$

is 3/16" (Table J2.4)

$$l = \frac{P_u}{2(1.392(3))}$$

welds

strength of 1/16" weld / in length

# of 1/16" in weld thickness

$$l = \frac{255}{2(1.392(3))} = 30.5\,in \rightarrow \text{large}$$

⇒ increase weld size 1/4"

$$l = \frac{255}{2(1.392(4))} = 22.8'' \rightarrow 23''$$

$$\boxed{\text{top plate} = 1\,5/8'' \times 5.00'' \times 23''}$$

⚠ can reduce plate length w/ increased weld size (cost trade off)

(6) rupture strength of base @ tension flange

plate:

$$t_{min} = \frac{3.09D}{F_u}$$

D → # 1/16" of weld

F_u → material ultimate strength

(Manual 9-2)

$$t_{min} = \frac{3.09(4)}{58} = 0.213in < t_p = 1.5/8'' \quad ok$$

beam

$$t_{min} = \frac{3.09(4)}{65} = 0.190in < t_f = 0.425in \quad ok$$

H.2.2 Compression Plate Design

(7) block shear rupture of top flange of beam

→ critical limit state = shear yielding

$$A_{gv} = (1.5'' + 23'')(0.425'') = 10.4\,in^2$$

$$A_{nt} = (0.5'')(0.425'') = 0.213\,in^2$$

nominal strength for tension rupture

$$F_uA_{nt} = 65(0.213) = 13.8\,K$$

nominal strength for shear yield

$0.6 F_y A_{gv} = 0.6(50\,ksi)(10.4\,in^2) = 312\,K$

block shear

$\phi R_n = 2(0.75(312 + 13.8)) = 488.7\,K > 255\,K$ ok

(8) required compression flange plate

- assume yielding controls

- b plate > $b_f + 1$ (maintain downward welds

$b_{plate} = 6.00 + 1.00 = 7.00\,in = l$ (maintain plate area of tension flange)

$t_p = \frac{A_p}{l} = \frac{8.13\,in^2}{7.00\,in} = 1.16 \rightarrow 1''\ 1/4''$

(9) compression plate local buckling (Table B4.1a)

stiffened plate (where plate and beam flange overlaps)

$\frac{b_f}{t} = \frac{6.00\,in}{1.25\,in} = 4.8 < 1.49\sqrt{\frac{E}{F_y}} = 1.49\sqrt{\frac{29{,}000\,ksi}{36\,ksi}} = 42.3$ ok

unstiffed plate (only plate)

$\frac{b}{t} = \frac{0.5\,in}{1.25\,in} = 0.4 < 0.56\sqrt{\frac{E}{F_y}} = 0.56\sqrt{\frac{29{,}000}{36}} = 15.9$ OK

local buckling is not a controlling limit state

(10) compressive strength of plate

- same load → use same weld 1/4'' 28''

slenderness ratio of plate $K = 0.65$ (fixed - fixed)

$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{h^2}{12}} = \sqrt{\frac{(1.25\,in)^2}{12}} = 0.361$

$\frac{l_c}{r} = \frac{KL}{r} = \frac{0.65(2.0)}{0.361} = 3.603$ * 2.0 in between plate and column susseptible to local buckling

$\frac{l_c}{r} \leq 25$ $P_n = F_y A_g \Rightarrow F_y = F_{cr}$ (J4-6)

=> plate limiting state is plate yielding → original assumption holds

(11) rupture strength of base material @ compression flange

plate $t_{min} = \frac{3.09(4)}{58} = 0.213 < t_p = 1\ 1/4''$

beam $t_{min} = \frac{3.09(4)}{65} = 0.190 < t_f = 0.425''$ ok

H.2.3 Column Weld Design

(12) welds to column

$$P_u = \frac{M_u}{(d + t_{pt}/2 + t_{pc}/2)} = \frac{219\ k\text{-}ft\ (12)}{(17.7in + (1+3/8)/2 + (1+1/4)/2)} = 137\ K$$

$t_{pt} < t_{pc} < b_f = 10.1in$ → transverse loading

$$D = \frac{P_u}{1.5(1.392)(2b_p)} = \frac{137k}{1.5(1.392)(2b_p)} = \frac{32.8}{b_p}$$

increase for transverse loading

strength of 1'' 1/16'' weld thickness

plate width

bottom plate

$$D = \frac{32.8}{7.00} = 4.69 \text{ sixteenths} => 5/16''$$

top plate

$$D = \frac{32.8}{5.00} = 6.56 \text{ sixteenths} => 7/16''$$

required thickness of column flange

$$t_{min} = \frac{3.09D}{F_U} = \frac{3.09(7)}{65} = 0.333 < t_f = 0.660in \text{ ok}$$

H.3 Single Welded Web Plate Design

Single Plate web connection → bolted to beam, welded to column

(1) required strength · $R_U = 34K$

(2) Table 10-10a (assume)

$d_b = 3/4''$ w/ standard holes Group A-N

$t_p = 1/4''$; $w = 3/16''$ $n = 4 \Rightarrow l = 11.5''$

spacing = $3''$ c-t-c $1\ 1/4'' = l_{ev}$ $l_{eh} = 1\ 1/2''$

$\phi R_n = 52.2\ K > 34K$ ok (extra capacity)

→ try $n = 3$ $l = 8.5''$ (fits on web)

$\phi R_n = 38.3 > 34\ K$ ok (more efficient) → use 3 bolts

(3) web tear out & bearing (Table 7-4)

$S = 3in$ $F_u = 65ksi$

available strength = 87.8 k/in/bolt

$\phi R_n = 3\ (87.8\ K/in/bolt)\ (0.300) = 79\ K > 34K$ ok

⚠ uncoped section, so block shear, shear yielding, and shear rupture wont govern

Appendix I: Concentrated Load Effect Calculations

local effects

① Beam properties

W18x35 $d = 17.7in$ $t_f = 0.425in$ $t_w = 0.300in$ $k_{des} = 0.827in$

② Required Load $R_u = 34k$

③ Web local yielding

$R_n = F_{yw}\, t_w\, (2.5k + l_b)$ (Eq. J10-3)

$F_{yw} = 50\ ksi$ $l_b = 8\ in.$

$R_n = 50\ ksi\,(0.300in)(2.5\,(0.827in) + 8in) = 151\ K$

$\phi = 1.00$ => $\phi R_n = 151k > R_u = 34\ k$ ok

④ web local crippling

end reaction => $d/2 >$ distance from end

$l_b/d = 8in/17.7 = 0.452 > 0.2$

$$R_n = 0.40\, t_w^2\left[1 + \left(\frac{4l_b}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E F_{yw} t_f}{t_w}}\, Q_f \quad (J10\text{-}5b)$$

$$R_n = 0.40(0.3in)^2\left[1 + \left(\frac{4(8in)}{17.7in} - 0.2\right)\left(\frac{0.3in}{0.425in}\right)^{1.5}\right]\sqrt{\frac{29000\,ksi \cdot 50ksi \cdot 0.425in}{0.3\ in}} \cdot 1.0$$

$R_n = 100.8K$

$\phi = 0.75$ => $\phi R_n = 75.6k > 34\ K$ ok