Design and Construction of Modular Concrete Footings and Testing Assembly for Use in Cal Poly's High Bay Laboratory

ARCE 453 SENIOR DESIGN PROJECT

Winter/Spring 2021

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Modular Block Introduction

To improve the testing capabilities of the ARCE high bay lab, a pair of modular concrete blocks were designed and fabricated. These blocks were created with one hundred connection points each to allow for a variety of structural test setups and aid future student and faculty research.

Modular Block Design

The modular blocks were developed with adaptability as the primary goal. The top face of the concrete blocks consists of a regular pattern of 72 steel couplers to allow for a variety of attachment methods depending on the test setup. Additionally, there are steel couplers on the four vertical faces of the footing, which make up the remaining 28 connection points. The concrete itself is a high-strength mix design intended to maximize the capacity of each block to suit a variety of loading demands. Inside of each block is a tight cage of rebar to prevent tensile and flexural failure due to applied loading. The modular nature of these blocks is also shown in their ability to attach to each other as well as the existing lab environment. Through post-tensioning strands routed through PVC conduits, two blocks can be compressed together to provide a larger surface to connect to. The blocks are designed with a small gap that is filled with a sheetrock layer that allows the blocks to compress together without directly touching one another. The blocks can also be tied down to the existing strong floor present in the high bay lab through similar vertical conduit pipes. This provides a frictional capacity against the floor to prevent sliding and to avoid any impact on critical test data. The modular nature of these blocks will allow for more to be created in the future that can be tied to the existing blocks to further increase the testing opportunities within the lab. All calculations and drawings for the modular blocks can be found in the appendix.

Modular Block Construction

Before construction began, it became clear that pouring these blocks while inverted would be the best way to get a level surface along the face with the most connection points. In addition, the couplers could be secured to the plywood at the bottom, allowing them to be flush with the face of the concrete.

Throughout the construction process, multiple material suppliers were contacted to communicate project deadlines and the concrete pour date. Cal Portland donated all of the high-strength concrete for the project and was incredibly flexible with the project timeline as preparation for the pour date moved along.

The two modular blocks were fabricated with plywood formwork to contain the concrete while curing. A CNC machine was used to create the intricate pattern of connection points on all sides of the block to avoid error and increase the speed of fabrication. An AutoCAD file was prepared to guide the machine's path to match the structural drawings. This also allowed for a dado to be cut around the perimeter of the base piece, simplifying the connection process between the plywood.

Fig 1: CNC Machine Cutting Formwork

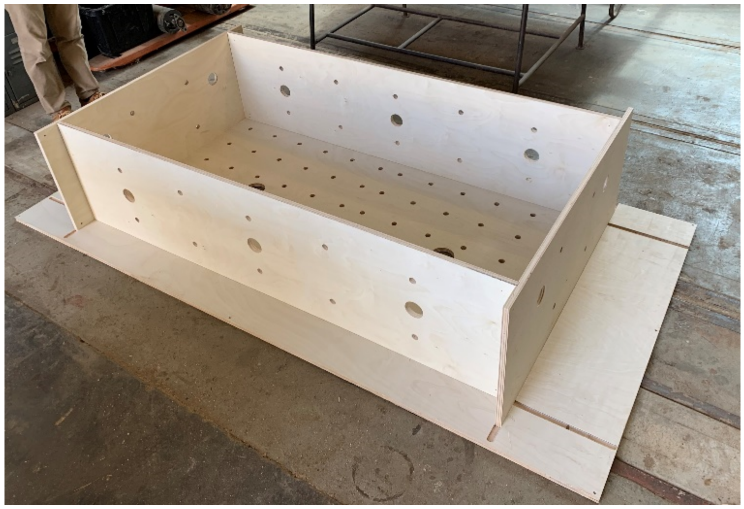

Fig 2: Final Formwork Assembly

Once the formwork was complete, the final assembly process began. The first task was to attach the couplers to the base of the formwork. The couplers were secured to the plywood using temporary 7/8” bolts and the permanent threaded rods were secured along with a plate washer at the far end to help prevent pullout once the concrete was poured.

Fig 3: Couplers and Threaded Rods in Formwork

After all of the couplers and anchor rods were secured in place, the formwork was laid down, restricting access to the bottom face of the assembly. This was done to make the placement of the remaining reinforcing steel more convenient, however, this caused additional issues. The formwork sides severely restricted access to much of the rebar cage, making it very difficult to complete the required rebar ties. Additionally, because the formwork sides were screwed on from the bottom to prevent uplift from the fluid pressure of the concrete, they could not be easily removed. To solve this issue, the formwork and partially assembled rebar cage were all lifted using a forklift and the screws were removed. For the construction of the second footing, this issue was proactively solved by waiting to attach the formwork sides to the formwork base. After construction, the formwork sides were connected to the base via toenails, which prevented uplift without requiring access to the bottom face of the assembly.

The horizontal rods spanning the block were secured to the plywood using a strongback system to increase the buckling capacity of the formwork. As concrete is poured into the block, the plywood wants to bow out due to the added load. By tying parallel ends of the plywood to one another, a tension force helps to resist the bowing and keep the vertical faces of the block level. The strongbacks helped to spread this force out to more surface area on the plywood face to avoid curvature over the entire surface. This step presented a challenge as the rods were placed at the same elevation in the drawings. This was done purposefully, as it was assumed that the rods would have enough tolerance to bend about one another without causing significant construction challenges. However, this did not end up being the case, instead, the longitudinal rods had to be forcibly bent around the transverse rods to lead both to the proper connection points, requiring significant forceful effort. Once these were in place, horizontal PVC pipes were installed as paths for the post-tensioning rods, and vertical pipes were installed to allow connection to the strong floor anchors. These pipes were capped with duct tape to avoid any concrete filling during the final pour.

Fig 4: Formwork Strongbacks

After all of the steel components were attached to the formwork, the blocks were ready to be poured. With the assistance of the concrete supplier, the concrete was poured from a truck and vibrated to remove any voids. Once both sets of formwork were filled, the excess concrete was used to prepare four cylinders to be tested later in the curing process to determine the concrete strength. These cylinders were kept next to the formwork to be sure the curing process would match the concrete blocks and provide useful test data.

Fig 5: Vibrating Concrete on Pour Day

The blocks were leveled using a piece of dimensional lumber and finished using hand trowels and jointers. The top surface was leveled through multiple passes to provide a steady base once the block was flipped to its correct orientation. Once finishing was completed, the blocks were covered to avoid additional moisture and debris and left to cure for the next few weeks.

Fig 6: Final Concrete Finish

After two weeks of curing, the plywood formwork was removed to view the results of the pour. The concrete was mostly void-free, and nearly all of the connection points were flush with the edge as expected. Some of the concrete leaked into the couplers due to voids between the bolts on the exterior and threaded rods on the interior. In addition, the connection points on the sides of the blocks were slightly tilted since the rods had to be bent on the inside. Although this was not an ideal result, the connections were still useable and gave a learning experience on the fabrication of future blocks. The blocks were then covered again to finish the curing process.

Fig 7: Two-Week Formwork Removal

Once the formwork sides had been removed and the concrete allowed to cure another two weeks, the footings were flipped using the overhead crane in the High Bay Laboratory. Each block weighed approximately 4000 pounds which neared the capacity of the overhead crane. To begin the process, each footing was lifted onto small wooden blocks using a forklift which allowed large straps to be wrapped around the concrete. These straps were then connected to the crane and lifted, slowly rotating the footing until it sat vertically on its side. The straps were then repositioned and the process was reversed, lowering the block into its final inverted position. Special care had to be taken not to let the block slip or drop suddenly, as this posed a serious risk of overload to the overhead crane and threatened the safety of those under it. After the blocks were properly oriented, they were craned into place within the steel frames to be utilized in the testing setup.

Modular Block Compressive Test Results

During the curing process, compressive tests were performed to analyze the concrete strength. The compressive strength discussed with the supplier was to be a minimum of 5000 psi. Each test was performed on four-inch diameter cylinders. The first test was completed at 16 days and resulted in compressive strength of 4883 psi. This was already nearing the 5000 psi design strength that was desired in the order. At 48 days, a second compressive test was performed which resulted in a concrete strength of 6322 psi. The final strength ended up over 25 percent greater than the minimum. As a result, many of the preliminary calculations performed on the concrete strength were conservative since they were based on the predicted strength.

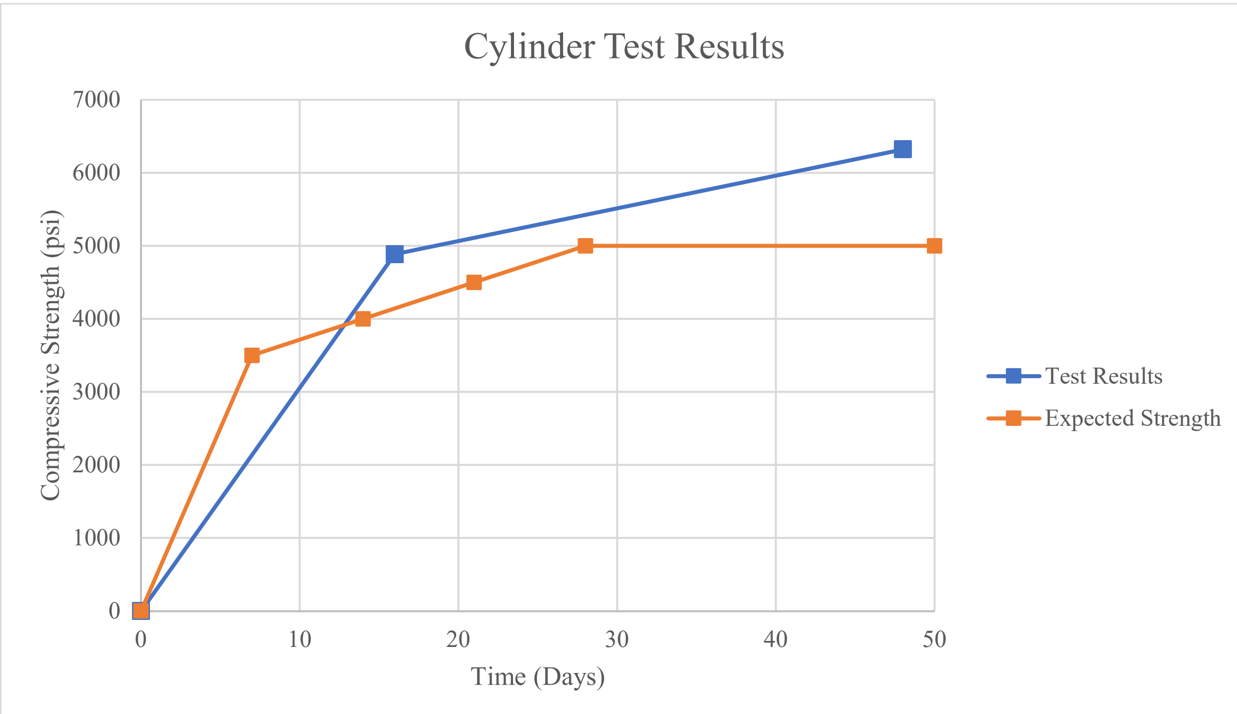

Fig 8: Theoretical Compressive Strength and Compression Test Results

Modular Block Hydraulic Anchorage

To ensure there would be no uplift or sliding during testing, each footing was anchored to the existing Strong Floor. To accomplish this, a hydraulic jack and specially made jig (Figure 9) were used to pretension the rods used to anchor the footing. Once the rods were pretensioned, bolts were tightened down by hand so that they were snug against the footing. The hydraulic jack pressure was then released, causing the anchor rod to elastically shorten and impart a downward force of 25 kips onto the block at each anchorage point (See Appendix A1.2 for calculation details). Additionally, this anchorage provided a significant frictional force which helped prevent the blocks from sliding. As the Strong Floor below these footings is not intentionally roughened, it is recommended per ACI 318-19 that the coefficient of friction used is 0.6 (ACI 318-19, Table 22.9.4.2). As there are 50 kips of downward load on each block once fully loaded, this would result in a lateral sliding resistance of 30 kips per footing.

Fig 9: Hydraulic Anchorage Rig

Modular Block Pullout Strength

In addition to addressing the pullout capacity of the Strong Floor, the pullout capacity of the couplers in the block itself was also addressed. To do this, Simpson Strong-Tie's Anchor Designer software was used. This software was unable to model the complex reinforcing present in the actual footing, so instead a lower bound pullout strength was determined assuming an unreinforced concrete block and a single applied point load. While a more detailed analysis will yield more accurate capacities, the pullout capacity of 41.6 kips as determined for a simple unreinforced section was significantly greater than required for any immediate tests and was therefore deemed acceptable for current use. It should be noted that this software was only able to provide calculations for a single applied point load. If several couplers are used close to one another, a pullout strength of 41.6 kips per coupler will likely not be valid and should be investigated further. Seen in the image below is a screenshot of the software used to check the pullout capacity of the concrete blocks. It shows the breakout strength, pullout strength, and steel strength of the footing for a single point load. As all of the hardware (apart from the rebar) used in the footings was donated by Simpson Strong-tie, the anchor rods used to attach the couplers were able to be modeled along with the concrete which showed the steel strength to be the failure mechanism for a single upwards point load.

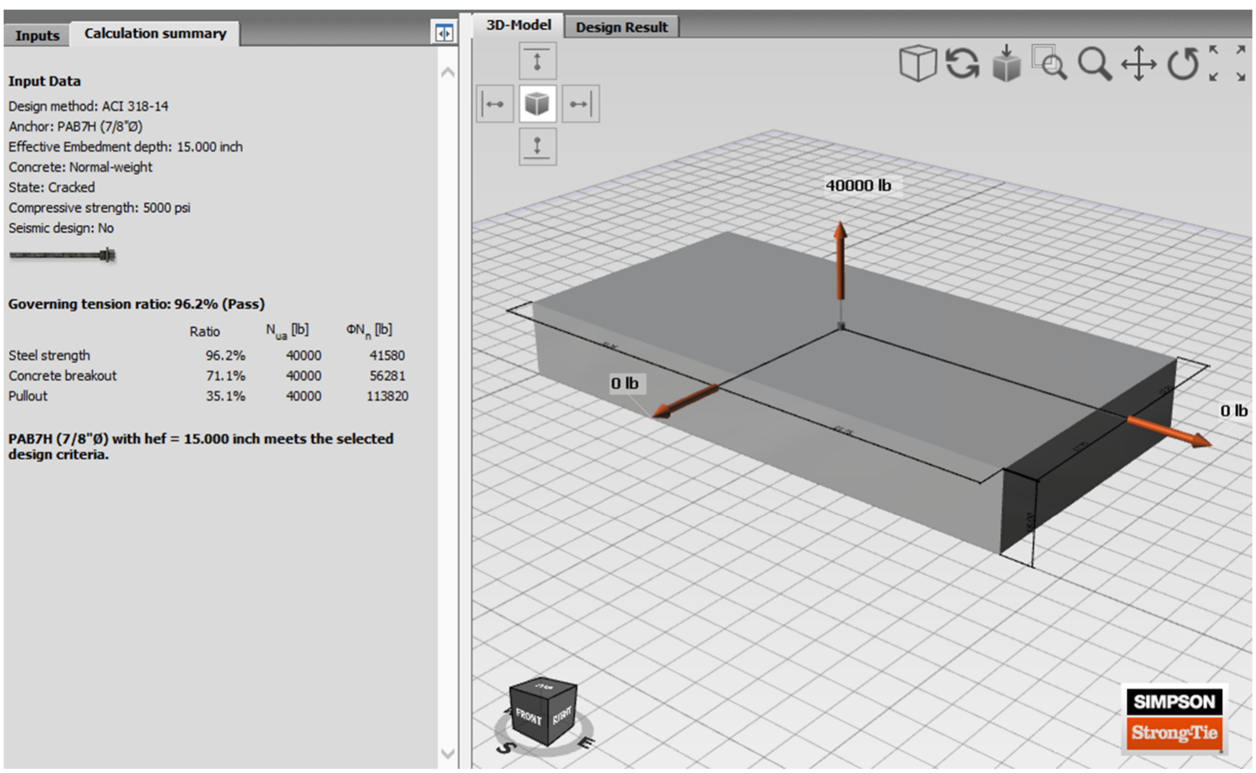

Fig 10: Simpson Anchorage Designer Sample Printout

Modular Block Conclusion

The construction of these modular blocks was a beneficial process in understanding the importance of coordination and careful detailing during the drawing phase. One of the most difficult parts of construction was tieing the rebar cage together inside of the plywood formwork. Although the process was improved for the second block, it was still a challenging process since the cage was so dense. This showed how much can vary in the construction phase because elements that fit tightly in the drawings are more difficult to place during construction. Efficiently laying out the rebar cage can make the construction process much smoother and thinking about the construction process while creating drawings improves their utility on-site. This lesson was emphasized by the longitudinal and transverse threaded rods which ended up occupying the same space in the drawings. This presented a challenge on-site that was solved but led to minor defects in the connections that could have been avoided. Both of these setbacks were dealt with, and the final product turned out well. In addition, these challenges led to a set of improvement goals for the next iteration of these blocks if more are fabricated in the future. The concrete was found to have few voids and all of the surfaces were level. Overall, the construction process was informative and will lead to improvements if more blocks are fabricated in the future.

Test Jig Design Goals

After the foundations were completed, the focus shifted towards the rest of the test setup. The goal was to provide a complete frame capable of testing steel beam to column connections. This setup would connect to the existing reaction frame in the high bay lab as well as the modular footings that are anchored to the Strong Floor. Shown below are the four main connections that had to be designed.

1. Column to reaction frame restraints to prevent column sway
2. Column base connection to modular footing
3. Load cell to test beam connection
4. Actuator to modular footing connection

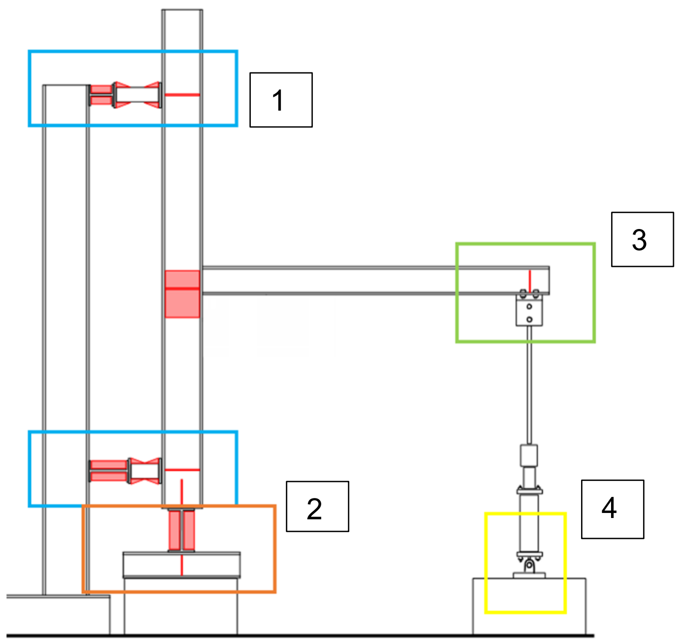

Fig 11: Test Jig Connection Layout

Along with these connections, the lateral-torsional bracing of the beam was considered as well as the panel zone shear at the beam-column connection. Once all of these elements were designed, they were fabricated and set in place to be used in future testing.

All of these elements were designed based on the capacity of the floor anchors. Each floor anchor has approximately 25 kips of pullout strength before yielding, so two anchors per block mean 50 kips is the maximum force before permanent damage to the Strong Floor will occur. As a result, all connections were designed to resist at least 40 kips of force to protect the permanent features in the lab. Any damage is intended to occur in the test connection or beam as intended by whoever is conducting the tests. To aid in this goal, all of the connections used bolts when connecting to the reaction frame or test jig to promote replaceability if anything was damaged or required changes in the future. All calculations for connection design are included in Appendix 1.

Connection Design

The first connections designed attached the existing reaction frame in the high bay lab to the new test column.

Fig 12: High Bay Reaction Frane

This was intended to brace the column from bending and isolate deflections to occur in the beam only. As a result, a restraint was placed near the top and base of the column to keep the system rigid throughout its height. The connections were applied at the work points of the reaction frame members to decrease bending and shear in these elements. Each connection utilized a beam spanning between the reaction frames to engage each of them for more resistance. Since the connections themselves would almost exclusively receive axial force demands, they were constructed using a pipe as the main element.

Fig 13: Column to Reaction Frame Connections

Next, the column base connection was designed. The original intent for this connection was to provide a pin to allow column rotation as needed for the test. During calculations, it was discovered that stiffener plates were required to avoid flange bending on the beam, so this goal had to be abandoned in the final design. Flange bending would have permanently damaged this member and allowed for greater uplift on the column which was undesirable. As a result, the rotational stiffness of the connection was increased to reduce the vertical deflection.

Fig 14: Column Base Connection

The third connection connects the top of the actuator to the beam's free end. This connection had to be detachable in case any damage occurred in the test beam warranting replacement. As a result, the connection to the actuator and the beam were both designed to use bolts so this piece could be removed and attached to another beam if needed. To avoid torsional eccentricity upon loading, the connection was symmetrically designed to transfer forces directly into the web of the beam. An assembly was created that would allow the actuator heim to attach with a single bolt that could be easily attached to the beam.

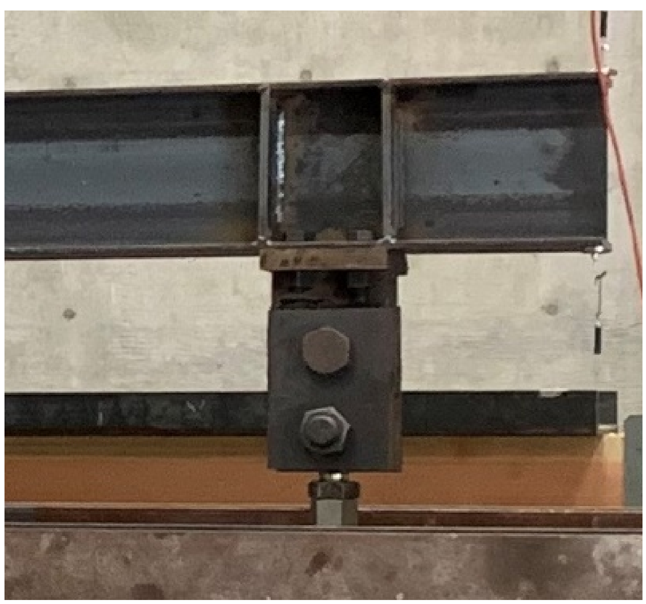

Fig 15: Actuator to Beam Connection

During construction, the design changed slightly since the original connection would have rotated due to the single bolt connection at the lower plates. The top face of the plate was welded as shown to avoid rotation in the connection.

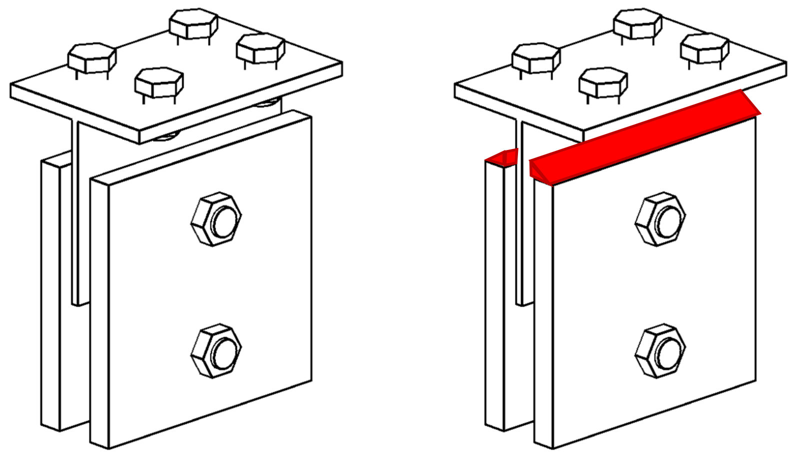

Fig 16: Original vs Final Actuator to Beam Connection Assembly

The fourth connection designed was the actuator base connection to the modular footings. Originally during design, this connection was planned to have the actuator close to the footing with a long rod spanning to the beam as pictured below.

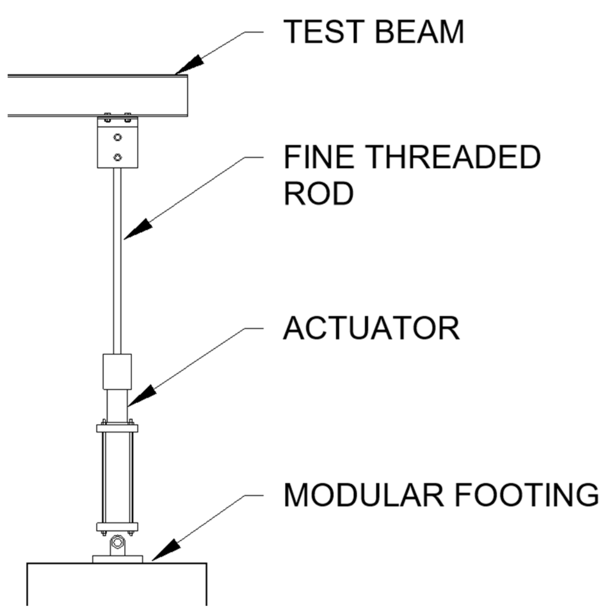

Fig 17: Original Actuator Base Connection

After challenges during the construction process and feedback from other sources, it was determined that the rod would undergo substantial bending moment demands which could

damage the load cell. The load cell was quite sensitive and damage to this element would be expensive and halt testing for the entire lab. As a result, the actuator base connection was redesigned to raise the bottom of the actuator higher and reduce the length of the threaded rod. This assembly was created similar to the reaction frame connections since it would also primarily experience axial loading.

Fig 18: Actuator Base Connection

Once these four connections were designed, the lateral-torsional buckling (LTB) restraints were considered. To prevent the beam from twisting during future tests, bracing columns were placed on either side of the beam at midspan to conform with AISC unbraced length requirements. These sat upon beams that spanned between the two modular footings already being used for other connections. The columns were then fitted with a sandwich-plate assembly that could be tightened or loosened to adjust for a variety of beam sizes.

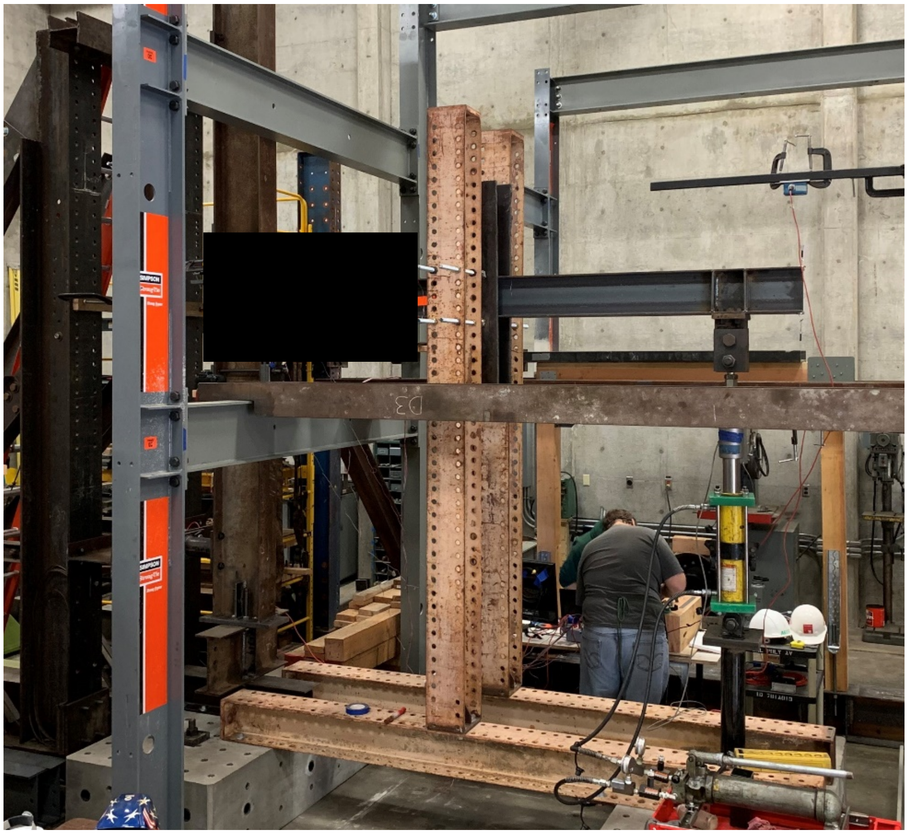

Fig 19: LTB Bracing Assembly

Finally, the panel zone shear was considered at the beam-column connection and it was determined that a doubler plate was required to be welded to the column web. Once all of the connections were designed and fabricated, the test jig members were raised into place using the overhead crane present in the High Bay and assembled as shown.

Test Jig Conclusion

The completed testing jig is capable of running full-scale steel connection tests, and with the modularity of the concrete footings, LTB bracing and sandwich plates, this assembly can provide effective and repeatable testing for future projects. As all components of the frame are designed to have higher capacities than the Strong Floor, there should be no concern of damaging the jig (given proper attention be paid to the Strong Floor capacity). All failures should be controlled by the test connection or beam itself to prevent impacting the permanent elements of the test jig. The reuse of this jig will provide financial savings to the ARCE department, as little additional material will have to be purchased to run future full-scale steel connection tests. This jig will also reduce the amount of material wasted on rerunning similar tests and reduce the amount of time spent by future students designing similar test jigs. By reusing this testing assembly, hopefully, the time, money, and material saved will be used by future students to better the ARCE department and the field of structural engineering.

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Appendix A1: Calculations

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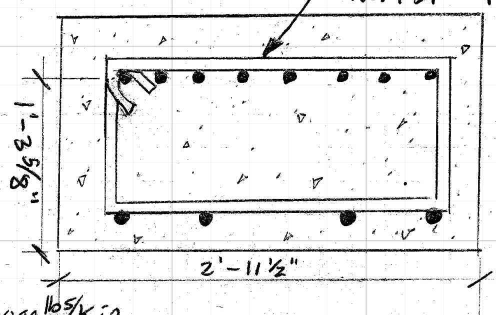
Appendix A1.1: Modular Footing Strength

Modular Footing Shear Capacity

Stirrup Spacing = No. 4 Closed Stirrups at 6" o.c. ($A_v = 2 \cdot 0.20\,in^2 = 0.40\,in^2$)

$V_c = 2\lambda\sqrt{f'_c}\,b_w d$ (ACI 318-19)

$f'_c = 5000\,psi$
$b_w = 2'\text{-}11\tfrac{1}{2}" = 35.5"$
$d = 1'\text{-}3\tfrac{5}{8}" = 15.625"$
$\lambda = 1.0$ (Normal Weight Concrete)



$V_c = 2(1.0)\sqrt{5000\,psi}\,(35.5")(15.625")/1000\,lbs/kip$

$V_c = \underline{78.45^k}$

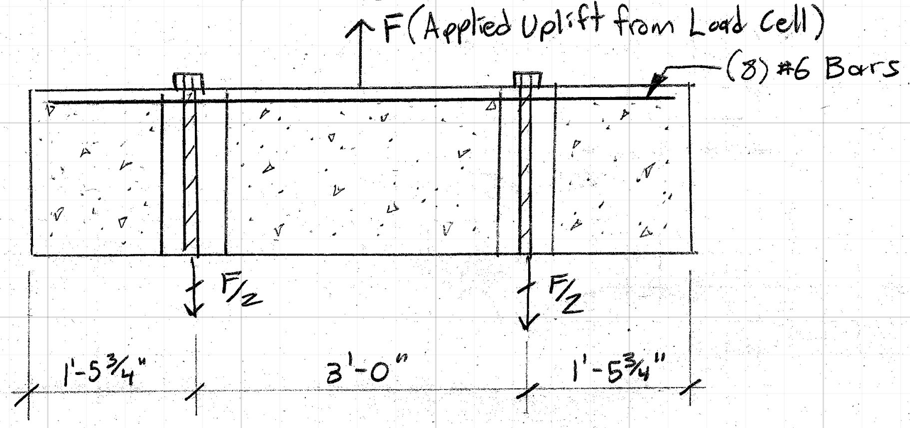
$V_s = \dfrac{A_v f_y d}{s}$ (ACI 318-19)

$A_v = 2 \cdot 0.20\,in^2 = 0.40\,in^2$
$f_y = 60\,ksi$
$d = 15.625"$
$s = 6"$

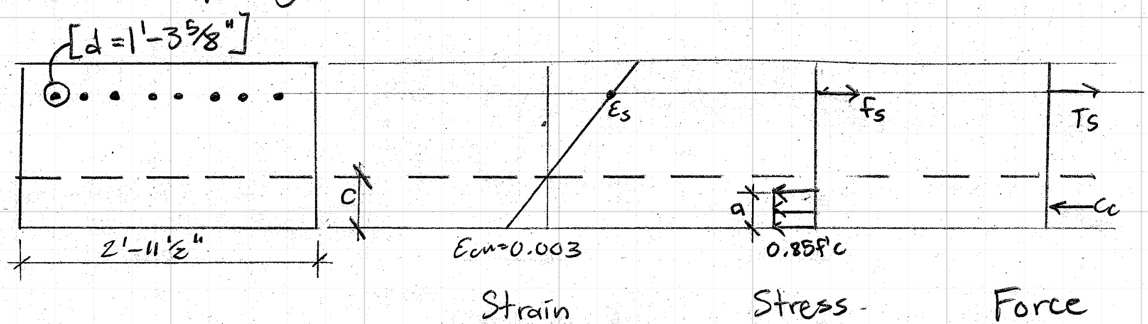
$V_s = \dfrac{0.40\,in^2(60\,ksi)(15.625")}{6"} = \underline{62.5^k}$

$V_n = \phi[V_c + V_s] = 0.75[78.45^k + 62.5^k] = \boxed{105.7^k}$

Modular Footing Bending Strength



Flexural Capacity



$T_s = A_s f_y = 8 \cdot 0.44 in^2 \cdot 60 ksi = 211.2^k$

$C_c = T_s \quad (\Sigma F_x = 0)$

$C_c = 0.85 f'_c a b \Rightarrow a = \dfrac{T_s}{0.85 f'_c b} = \dfrac{211.2^k}{0.85(5ksi)(35.5")} \Rightarrow a = 1.40"$

$c = \dfrac{a}{\beta_1} = \dfrac{1.40"}{0.80} = 1.75"$

Verify Top Steel is Yielded

Per Strain Diagram:

$\dfrac{\varepsilon_{cu}}{c} = \dfrac{\varepsilon_s}{d-c} \Rightarrow \varepsilon_s = \dfrac{0.003(15.625" - 1.75")}{1.75"} = 0.02379$

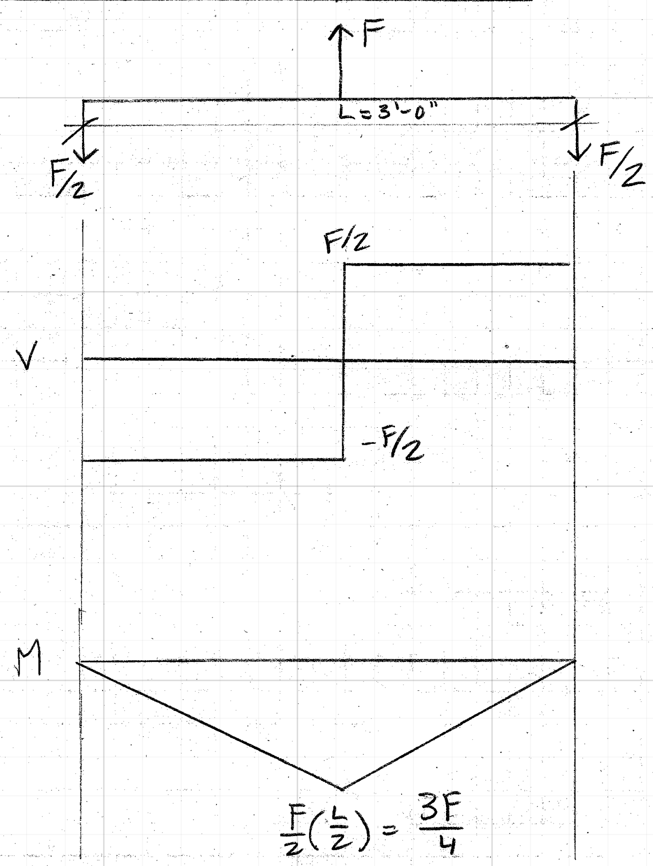
$\varepsilon_s = 0.02379 > 0.005 \rightarrow$ Steel is yielded and $\phi = 0.9$.

Nominal Flexural Capacity

$M_n = T_s[d - a/2] = 211.2^k[15.625" - 1.40"/2]/12^{in/ft} = 262.6^{k\text{-}ft}$

$\phi M_n = 0.9(262.6^{k\text{-}ft}) = \boxed{236.3^{k\text{-}ft}}$

Maximum Allowable Uplift Force



Shear Capacity: $\phi V_n = 105.7^k \Rightarrow 105.7^k = \frac{F}{2} \Rightarrow \underline{F_{max} = 52.9^k}$

Flexural Capicity: $\phi M_n = 236.3^{k\text{-}ft} \Rightarrow 236.3^{k\text{-}ft} = \frac{3F}{4} \Rightarrow \underline{F_{max} = 315.5^k}$

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Appendix A1.2: Hydraulic Anchorage

FOOTING ANCHORAGE

HYDRAULIC JACK CAPACITIES:

MAX FORCE = 120 kips

MAX PRESSURE = 10,000 psi

REQUIRED ANCHORAGE FORCE:

$F_{req'd}$ = 25 kips

* ASSUME PRESSURE $\propto$ FORCE IN HYDRAULIC RAM

$\therefore \; P_{req'd} = \frac{P_{MAX}}{F_{MAX}} \times F_{req'd} = \frac{10000}{120}(25) = 2083 \text{ psi}$

* NOTE: To maintain 25^k of force after nut engagement, pressurize to 2400 psi.

Since nuts are tightened only by hand

* ADDITIONALLY: Repeat this process at least twice to ensure pressure is at least 2083 psi.

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Appendix A1.3: Actuator Base Plate

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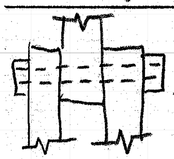
Assuming a 40^k maximum force on the actuator base connection:

Shear Strength (AISC 360 Table 7-1)



$\Rightarrow$ Double Shear, Group A

If a 1" ϕ Bolt is used, $\phi R_n = 63.6^k$ (Assuming Threads not continuous) $> 50^k$ ✓

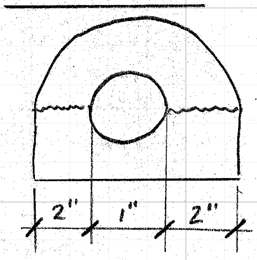
Strength Based on Edge Distance

2" of edge distance must be provided to ensure bolt shear capacity is reached.

For a 1" ϕ Bolt with edge distance ≥ 2", STD Hole:

$F_u = 58\,ksi \rightarrow \phi R_n = 75^k$
$F_u = 65\,ksi \rightarrow \phi R_n = 84.1^k$ $> 50^k$ ✓

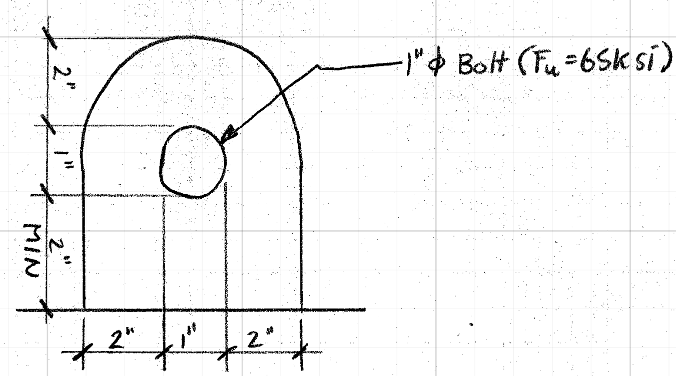
Block Shear



$L_{eh} = 2.5$"
$F_u = 65\,ksi$
$t_{plate} = 3/4$"
$\phi_{bolt} = 1$"

Tension Rupture $= 92.9^k/in$ (AISC Table 9-3a)
Strength $= 92.9^k/in\,(0.75") = 69.7^k > 40^k$ ✓

Final Connection



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Appendix A1.4: Actuator to Beam Connection

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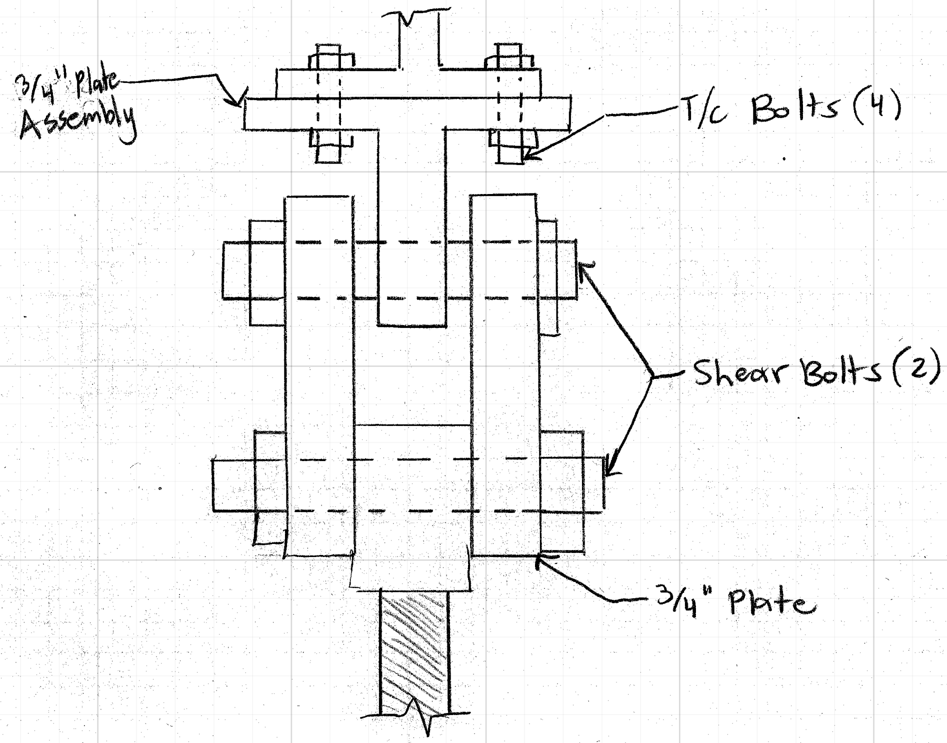
PROJECT Connection Test Jig

SUBJECT Actuator to Beam Connection



Shear Strength

Using A325 Bolts with threads included: Double Shear

AISC Table 7-1: Use 7/8" ϕ Bolts ($\phi r_n = 48.7^k$) $> 40^k$ ✓

Bolt Bearing Based on Spacing

Assuming a 3" Bolt spacing:

AISC Table 7-4: ($\phi R_n = 91.4 \frac{k}{in}$) for $F_u = 58$ksi (Conservative Case)

Using a 3/4" Plate: $\phi R_n = 91.4 \frac{k}{in} (0.75") = 68.6^k > 40^k$ ✓

Bolt Bearing Based on Edge Distance

Assuming a 2" Edge Distance:

AISC Table 7-5: ($\phi R_n = 79.9$ k/in) for $F_u = 68$ksi (Conservative)

Using a 3/4" Plate: $\phi R_n = 79.9 \frac{k}{in} (0.75") = 59.9^k > 40^k$ ✓

Required Plate Area

$\phi F_y A_g = 20^k \Rightarrow \frac{20}{0.9(36ksi)} = 0.617 in^2$

$t b = A_g \Rightarrow b = \frac{0.617 in^2}{0.75"} = 0.823" \Rightarrow$ Edge Distance will be more than enough

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PROJECT Connection Test Jig

SUBJECT Modular Block Strength

Use a 7" Long W8x21, Cut in half for the WT Piece

Yield Strength

$40^k \le \phi F_y A_g = 0.90(50\text{ ksi})(7")(0.25") = 78.8^k > 40^k$ ✓

Rupture Strength

$40^k \le \phi F_u A_e = 0.75(65\text{ksi})(7'' - \tfrac{7}{8}")(0.25") = 74.6^k > 40^k$ ✓

Revised to 3/4" Plates in final construction

T/C Bolts

4 Bolts between the two W8x21 pieces

Tension Capacity $\phi R_n = 40.6^k/\text{bolt}$ for 7/8" ϕ A352 Bolts

$4 \cdot 40.6^k = 162.4^k > 40^k$

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Appendix A1.5: Panel Zone Shear

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PANEL ZONE CHECK

DETERMINE SECTION MODULUS @ CONNECTION

$$S_{Gert} = \frac{I_{xx}}{y_{max}} = \frac{[75.3 + 6.16(8.28/2)^2] \times 2}{8.28} = 43.69 \text{ in.}^3$$

(I calc'd per connection specs)

DETERMINE MOMENT CAPACITY @ SUPPORT

$$M_n = S_{Gert} f_y = 43.26 \text{ in.}^3 \times 50 \text{ ksi} = 2185 \text{ k}'' = 182 \text{ k}'$$

$$F_f = \frac{M_n}{2d_b - t_f}$$

* Neglect Shear in Column for conservatism

$$= \frac{182 \text{ k}'}{2(8.28 - .4)/12}$$

$$= 139^k$$

DETERMINE PANEL ZONE STRENGTH PER AISC 360 § J10.6

$$\phi R_n = \phi 0.6 F_y d_c t_w \quad \text{(EQ. J10-9)}$$

← No AXIAL STRESS

$$= 0.9(0.6) \times 50 \times 8.28 \times 0.25$$

$$= 55.9^k < 139^k$$

X No Good

∴ STIFFNER PLATES REQ'D

↳ Add 3/8" Doubler PL. each side ($F_y = 36$ ksi)

W/ STIFFNER PLATES:

$$\phi R_n = 0.9(0.6) \times [50 \text{ ksi} \times 8.28'' \times 0.25'' + 36 \text{ ksi} \times 8.28'' \times \tfrac{3}{8}'$$

$$= 177^k > 139^k \checkmark$$

ARCE 453

WINTER 2021

ANDERS JOHNSON
THOMAS LITTLE

CALCULATED BY THOMAS LITTLE DATE 5/3/21

PROJECT TEST JIG DESIGN

SUBJECT PANEL ZONE SHEAR

PANEL ZONE SHEAR

DETERMINE MAX MOMENT ALLOWED W/O DOUBLER PLATES

$\phi R_n = 55.9^K$

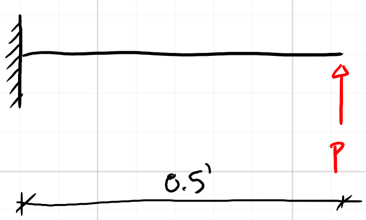
$F_f = \frac{M_n}{2d_b - t_f}$

Set $\phi R_n = F_f$

$55.9 = \frac{\phi M_n}{2(8.28 - .41)/12}$

$\phi M_n = 73.4^{K'}$

Assuming Beam Length = 8.5'



$\phi M_n = 8.5(\phi P)$

$\phi P = 73.4^{K'} / 8.5' = 8.64^K$

$\therefore$ 8.64^K is the maximum load that can be applied to ensure no panel zone shear

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Appendix A1.6: Lateral Torsional Buckling Requirements

ARCE 453

WINTER 2021

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THOMAS LITTLE

SHEET NO. 1 OF 1

CALCULATED BY AJ and TL DATE 6/10/2021

PROJECT Connection Test Jig

SUBJECT LTB Bracing

LTB Requirements

Beam Size = W8x21

Beam Length (Face of Column to Point of Load Application) ≈ 8'-6"

Based on AISC Table 3-2, (F_y = 50ksi),

L_p = 4.45' for a W8x21 Beam

Provide LTB restraint at midspan to prevent LTB.

Unbraced Length = 8.5'/2 = 4.25' < L_p ✓

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Appendix A1.7: Plate Bending

ARCE 453

WINTER 2021

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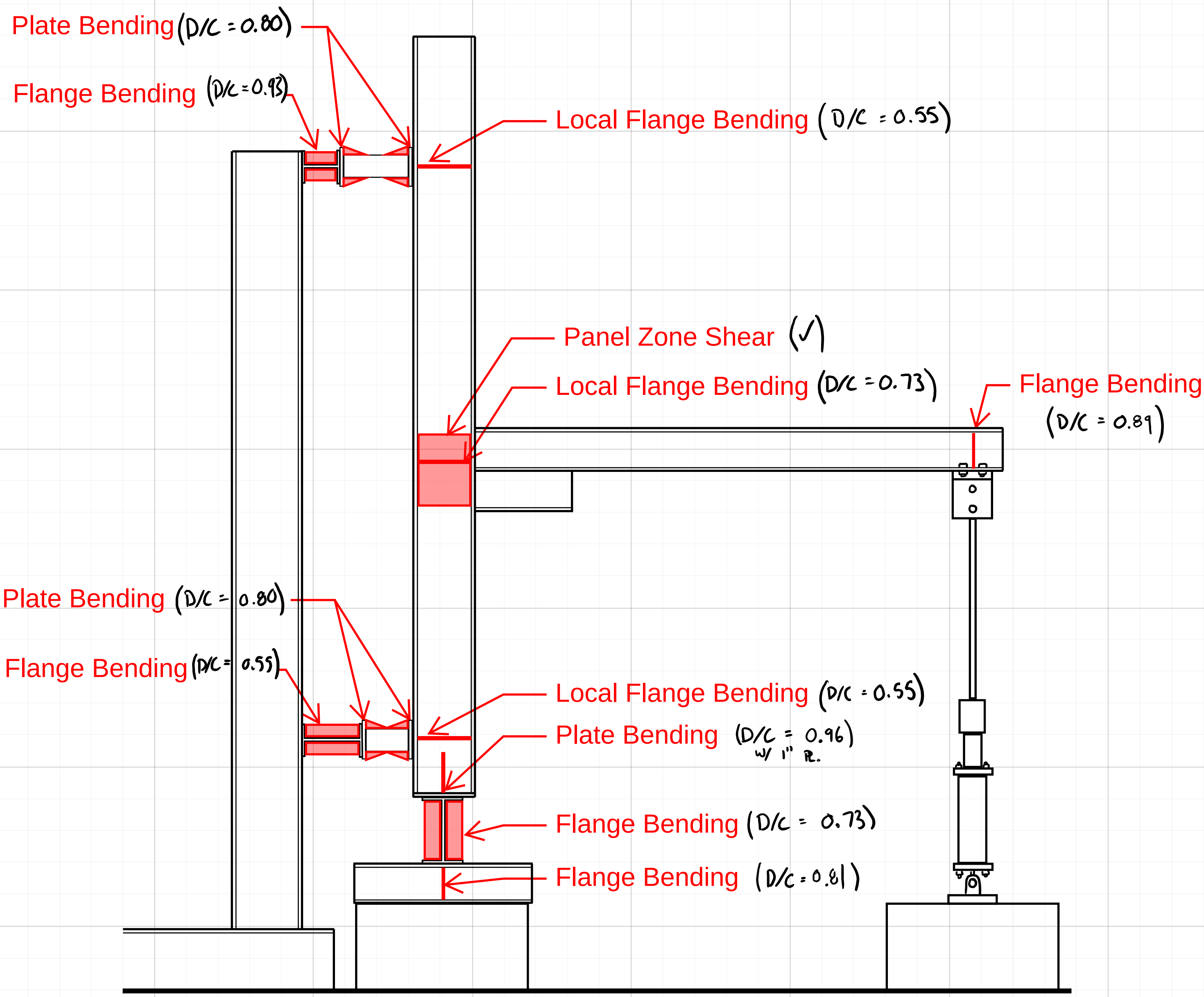
SHEET NO. 1 OF 9

CALCULATED BY AJ and TL DATE 6/10/2021

PROJECT Connection Test Jig

SUBJECT Plate Bending

STIFFENER PLATE SCHEMATIC DIAGRAM



★ D/C RATIOS ASSUMING NO STIFFENER PLATES

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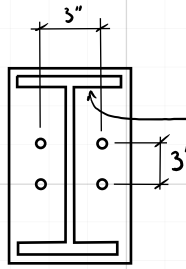
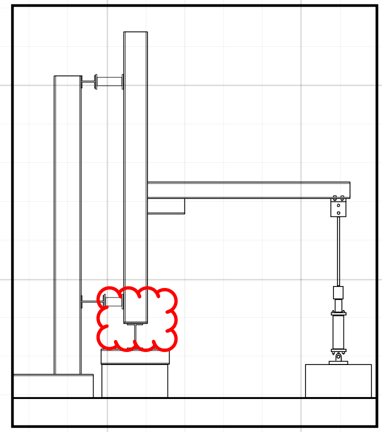
BASE PLATE BENDING CALCS

COLUMN BASE PLATE



Ignore fixity from flanges to simplify



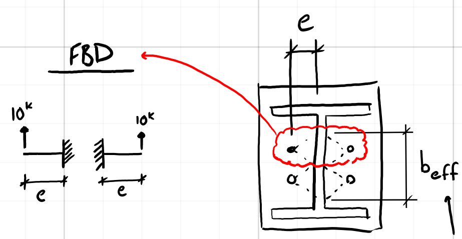
- Using AISC Base Plate & Anchor Rod Design Guide (2nd Edition)

DESIGN ASSUMPTIONS

↳ 40^k MAX LOAD
↳ BOLTS SHARE LOAD EQUALLY (10^k/bolt)
↳ ONE-WAY BENDING (45° bending projection lines)

$P = 10^k/\text{bolt}$

$$M_u = P \times e$$
$$= 10^k \times \left(\tfrac{3''}{2} - t_w/2\right)$$
$$= 10\left(\tfrac{3}{2} - \tfrac{3}{16}\right)$$
$$= 13.13 \text{ k}''$$



Determined using 45° projection lines

$$b_{eff} = \left(\tfrac{3}{2} - \tfrac{t_w}{2}\right) \times 2$$
$$= 2.63''$$

$$S = \frac{b_{eff} \times t^2}{6}$$

$$\therefore t_{req'd} = \sqrt{\frac{M_u\,(6)}{b_{eff}\,(\phi f_y)}} = \sqrt{\frac{13.13\,(6)}{2.63 \times 0.9 \times 36}} = 0.96''$$

$\boxed{\therefore \text{ USE STIFFENER PLATES}}$

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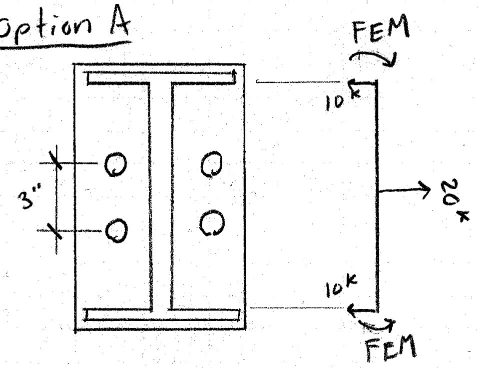
Using Simplified Model

Option A



MODEL

Section Properties: W12x35
$d = 12.5"$
$t_f = 0.520"$
$b_f = 6.56"$
$t_w = 0.300"$

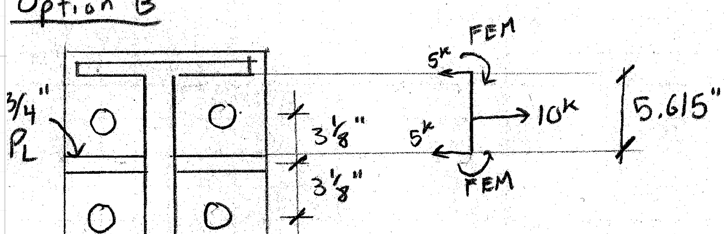
$$FEM = \frac{PL}{8} = \frac{20^k(12.5" - 0.520" \cdot 2)}{8} = 28.65^{k-in}$$ ⇒ Demand Moment

$$\phi M_n = \phi F_y Z_x$$

Where: $\phi = 0.9$
$F_y = 36 ksi$
$$Z_x = \frac{bh^3}{4} = \frac{(6.56"/2 - 0.300")(3/4")^3}{4} = 0.3143 in^3$$

$\phi M_n = 0.9\ (36ksi)(0.3143 in^3) = 10.18^{k-in}$ → Will not Work

Option B



MODEL

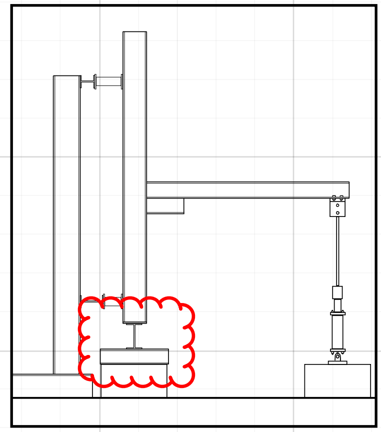
$$FEM = \frac{PL}{8} = \frac{10^k(12.5"/2 - 0.75"/2 - 0.52"/2)}{8} = 7.02^{k-in}$$

$$\phi M_n = \phi F_y Z_x$$

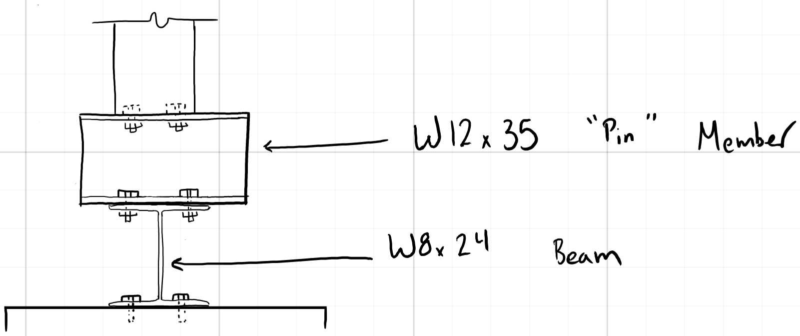
Where: $\phi = 0.9$
$F_y = 36 Ksi$
$$Z_x = \frac{bh^3}{4} = \frac{(6.56"/2 - 0.300")(0.75")^3}{4} = 0.3143 in^3$$

$\phi M_n = 0.9 (36Ksi)(0.3143 in^3) = 10.18^{k-in} > 7.02^{k-in}$ ✓

PLATE BENDING CALCS

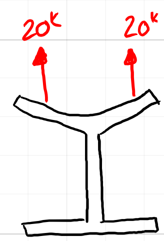


FOUNDATION CONNECTION ASSEMBLY



W8x24 BEAM

Per AISC § J10.1 - FLANGE LOCAL BENDING



$\phi R_n = 6.25 F_y t_f^2$ (Eq. J10-1)

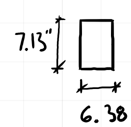
$= 0.9(6.25 \times 36ksi \times 0.400^2 in.^2)$

$= 32.4^k$

Assume A36 Member

Max T force = $40^k > \phi R_n = 32^k$ X

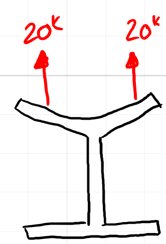
∴ Use STIFFENER PL. (D/C = 0.81)



STIF. PLATE

W12x35 "PIN"

Per AISC § J10.1 - FLANGE LOCAL BENDING



$\phi R_n = 6.25 F_y t_f^2$ (Eq. J10-1)

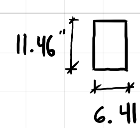
$= 0.9(6.25 \times 36ksi \times 0.520^2 in.^2)$

$= 54.8^k$

Assume A36 Member

Max T force = $40^k < 55^k$ ✓

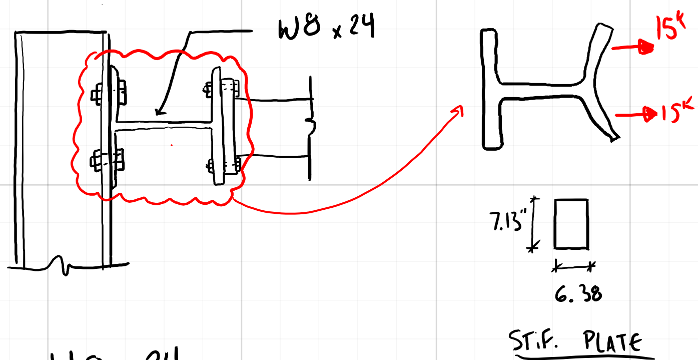
∴ Use STIFFENER PL. (D/C = 0.73)

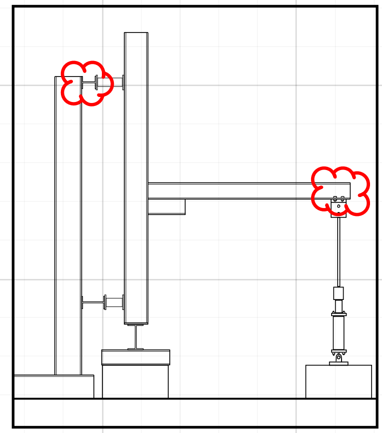


STIF. PLATE

PLATE BENDING CALCS

REACTION FRAME CONNECTION (TOP)





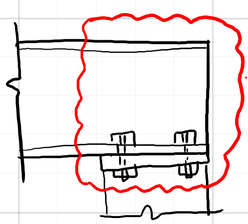
W8x24

Capacity Same as Previously Calculated

$\phi R_n = 32.4^k$

$P_u = 30^k$ (From Previous Frame Statics Calculation)

$D/C = 0.93$



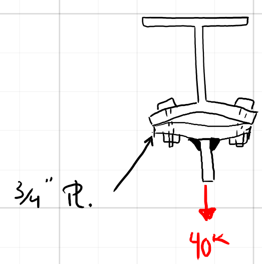
W8x21 BEAM-TO-ACTUATOR CONNECTION

Per AISC §J10.1 - FLANGE LOCAL BENDING

$$\phi R_n = 6.25 F_y t_f^2 \qquad \text{(Eq. J10-1)}$$

$$= 0.9(6.25 \times \underbrace{[50_{ksi} \times 0.400^2 \ in.^2]}_{\text{New W8x21}})$$

$$= 45^k$$

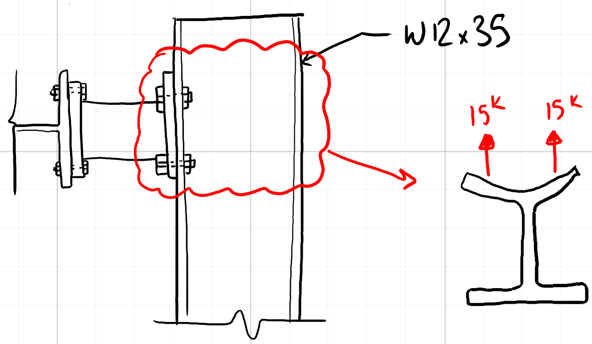


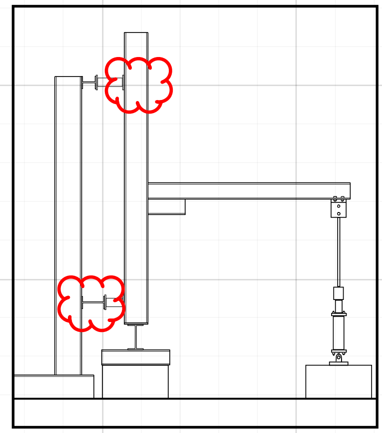
Max T force = $40^k < \phi R_n = 45^k$

$D/C = 0.89$ $\boxed{\therefore \text{Use STIFFENER PLATES}}$

PLATE BENDING CALCS

FOUNDATION CONNECTION ASSEMBLY





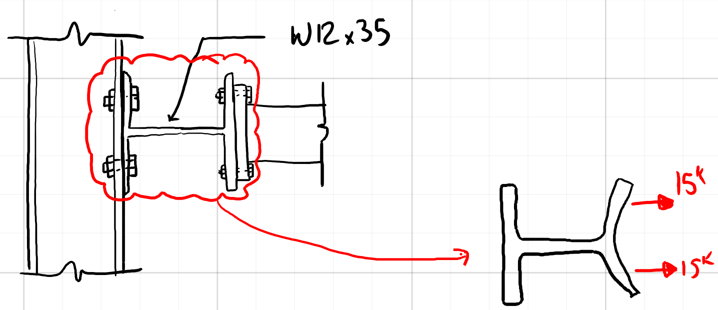
W12x35 COLUMN

Capacity Same as Previously Calculated

$\phi R_n = 54.8^k$

$P_u = 30^k$ (From Previous Frame Statics Calculation)

$D/C = 0.55$

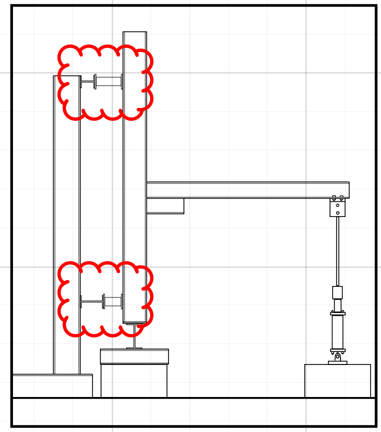


REACTION FRAME CONNECTION (BOTTOM)

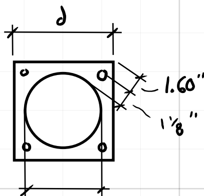
Same capacity & Demand as above

$D/C = 0.55$

BASE PLATE BENDING CALCS



PIPE PLATE BENDING



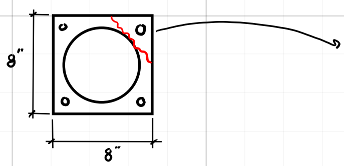
$d_{min} = (4.5" + 2(1.60 + 1.125)) \frac{1}{\sqrt{2}}$
$= 7.2"$

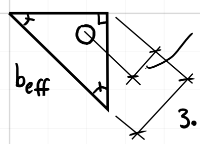
→ Use 8" x 8" PLATE

* Assume 1⅛" ED
(⅞" Bolts)

CHECK BENDING

USE 8" x 8" PL.

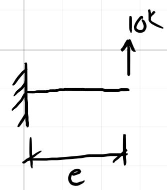




$1.60" = 1.125\sqrt{2}$

$3.41" = (8\sqrt{2} - 4.5)\frac{1}{2}$

by geometry
$b_{eff} = 2(3.41") = 6.8"$



* Assume 10k per bolt

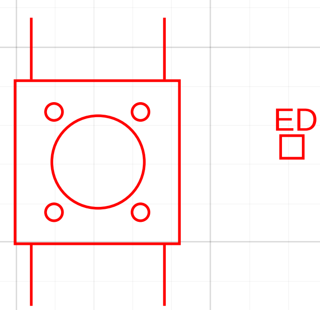
$S = \frac{b_{eff} \times t^2}{6}$

$M_u = P \times e$
$= 10^k \times 1.81"$
$= 18.1 k"$

$t_{req'd} = \sqrt{\frac{M_u(6)}{b_{eff}(\phi f_y)}} = \sqrt{\frac{18.1(6)}{6.8(0.9)36}} = 0.70"$

CHECK W/ NEW CONTROLLING DIMENSIONS
- SEE NEXT PAGE

Check w/ 6.5" max pl. width (gov'd by col.)



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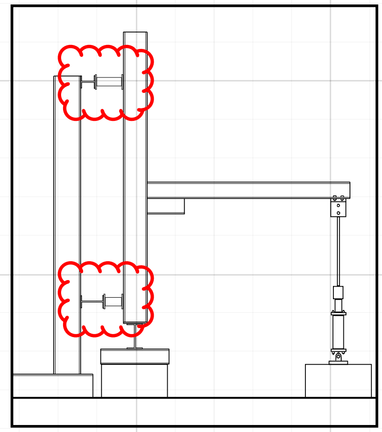
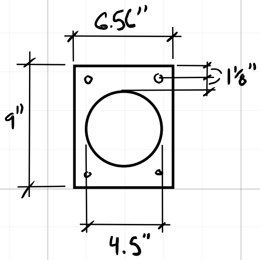
BASE PLATE BENDING CALCS



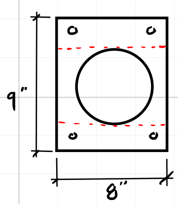
PIPE PLATE BENDING

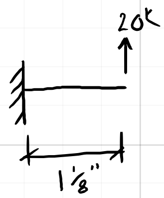


* Assume 1 1/8" ED
(7/8" Bolts)

CHECK BENDING

Use 8" × 9" PL.





* Assume 10k per bolt

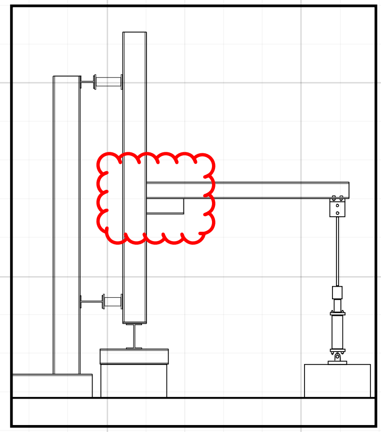
$S = \frac{b_{eff} \times t^2}{6}$

$M_u = P \times e$
$= 20^k \times 1.125''$
$= 22.5_{k}''$

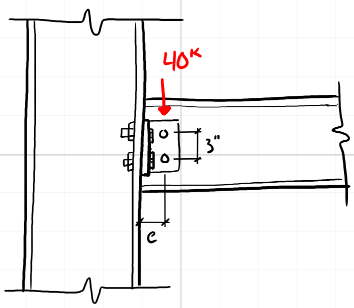
$t_{req'd} = \sqrt{\frac{M_u(6)}{b_{eff}(\phi f_y)}} = \sqrt{\frac{22.5(6)}{6.25(0.9)36}} = 0.80''$

$\therefore$ Add stiffener Plates

BASE PLATE BENDING CALCS



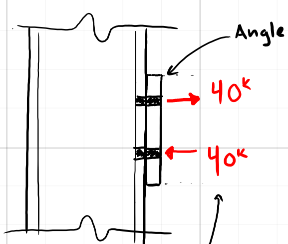
BEAM-TO-COLUMN PLATE BENDING



↳ Assume $e = 3"$

$M = 40^k \times 3" = 120\ k"$

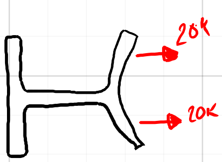
CHECK FLANGE LOCAL BENDING



Per AISC § J10.1 - FLANGE LOCAL BENDING

$$\phi R_n = 6.25 F_y t_f^2 \quad \text{(Eq. J10-1)}$$
$$= 0.9(6.25 \times 36\,ksi \times 0.520^2\ in.^2)$$
$$= 54.8^k$$

Assume A36 col.

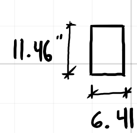


Max T force $= 40^k$

$\phi R_n > 40^k$ ✓

$(D/C = 0.73)$

$\boxed{\therefore \text{ stiffner Plate required}}$



STIF. PLATE

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Appendix A2: Drawings

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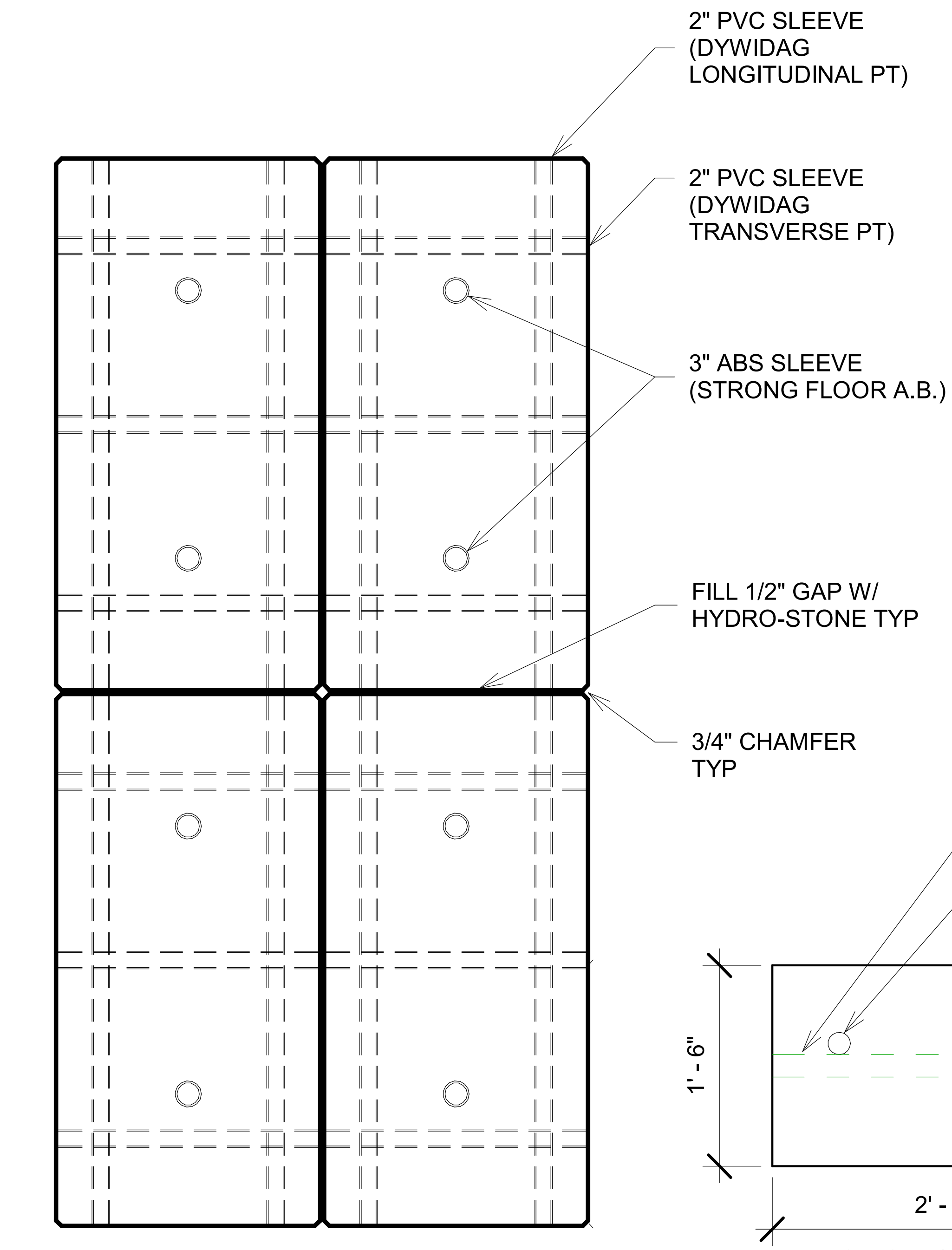
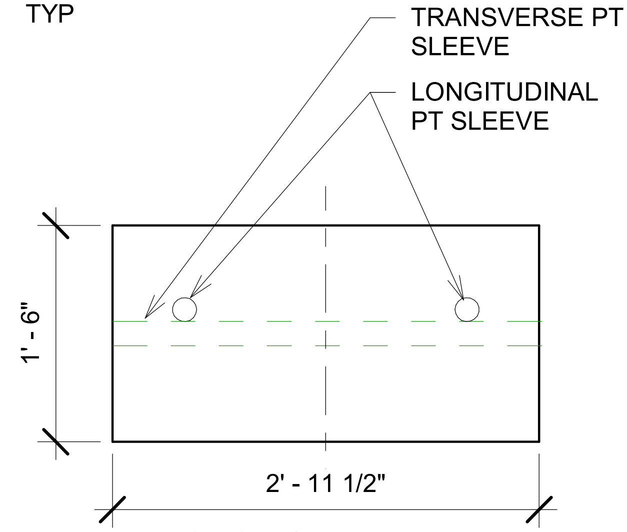
Appendix A2.1: Modular Block Drawings



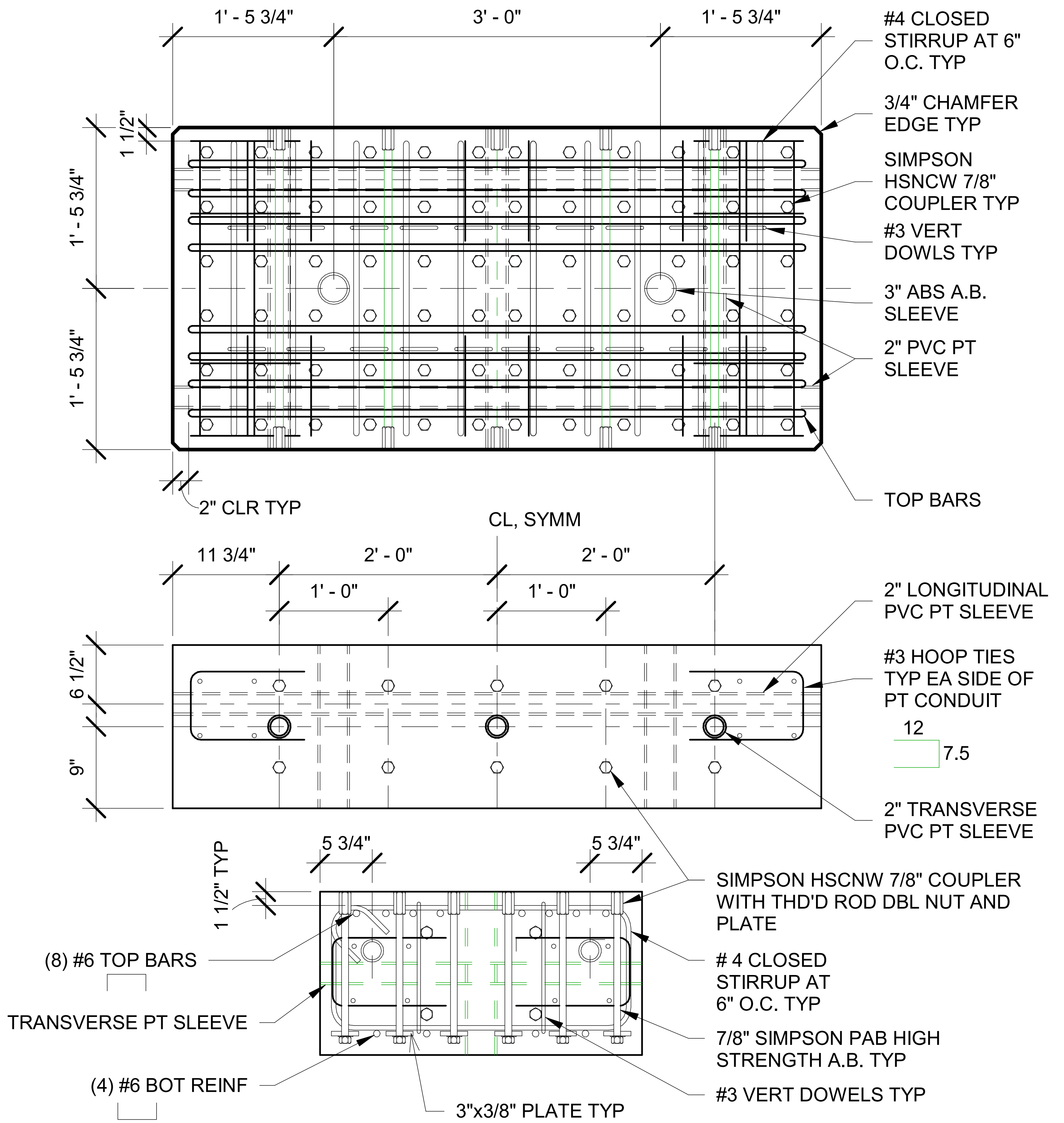
1 MB PLAN
1/2" = 1'-0"



2 MB SECTION
3/4" = 1'-0"

CAL POLY
SAN LUIS OBISPO

PROJECT: HIGHBAY LAB MODULAR TESTING BLOCKS		SKETCH NO.
ISSUE DATE: 6/8/2021 8:39:47 PM	SCALE: As indicated	
SKETCH TITLE: MODULAR BLOCK SYSTEM		DWG REF. S1



1 6x3 Modular Block-MJD
3/4" = 1'-0"

CAL POLY SAN LUIS OBISPO	PROJECT: HIGHBAY LAB MODULAR TESTING BLOCKS	SKETCH NO. Checker
	ISSUE DATE: 6/8/2021 8:39:47 PM — SCALE: 3/4" = 1'-0"	
	SKETCH TITLE: 6x3 MODULAR BLOCK REINFORCING	DWG REF. S2

MATERIAL TAKE OFF FOR ONE (1) 3'x6' CONCRETE FOOTING

MEMBER NAME	INDIVIDUAL LENGTH	NO. OF MEMBERS	TOTAL LENGTH	MEMBER DESCRIPTION
#6 TOP STL.	7' - 5"	8	59' - 4"	5' - 7 1/2" 1' - 0" NOTE: DIMENSIONS ARE MEASURED OUTSIDE TO OUSTIDE OF BAR
#6 BOTTOM STL.	7' - 5"	4	29' - 8"	5' - 7 1/2" 1' - 0" NOTE: DIMENSIONS ARE MEASURED OUTSIDE TO OUSTIDE OF BAR
#4 CLOSED STIRRUPS	8' - 3 1/2"	10	85'	2' - 8 1/2" 1' - 1 3/4" NOTE: DIMENSIONS ARE MEASURED OUTSIDE TO OUSTIDE OF BAR
#4 STRAIGHT BAR	5' - 7 1/2"	8	45'	
#4 STRAIGHT BAR	2' - 8 1/2"	8	21' - 8"	
#3 VERT DOWELS	1' - 10 1/4"	20	38' - 9"	1' - 2 1/2" NOTE: DIMENSIONS ARE MEASURED OUTSIDE TO OUSTIDE OF BAR
#3 HOOP TIE	2' - 7 3/4"	20	52' - 11"	12" 7 3/4" NOTE: DIMENSIONS ARE MEASURED OUTSIDE TO OUSTIDE OF BAR
2" PVC PT SLEEVE	3' - 1"	3	9' - 3"	
2" PVC PT SLEEVE	6' - 1"	2	12' - 2"	
3" ABS A.B. SLEEVE	1' - 6 3/4"	2	3' - 3"	
SIMPSON HSCNW 7/8" COUPLER WITH HS THD'D ROD DBL NUT AND PLATE	1' - 3 1/2" ROD	72	93'	
SIMPSON HSCNW 7/8" COUPLER WITH HS THD'D ROD DBL NUT AND PLATE	2' - 9" ROD	10	27' - 6"	
SIMPSON HSCNW 7/8" COUPLER WITH HS THD'D ROD DBL NUT AND PLATE	5' - 9" ROD	4	23'	
3/4" THK. 4' x 8' PLYWOOD SHEET	N/A	3	N/A	8' 4'
DF-L 2 x 4	5' - 11 1/2"	14	83' - 5"	
DF-L 2 x 4	3' - 1"	14	43' - 2"	
DF-L 2 x 4	1' - 6"	4	6'	

HS THD'D ROD - ASTM A449 or ASTM F1554
REBAR TO BE GRADE 60 - ASTM A615

1 Material Takeoff-Mod Ftg
1/2" = 1'-0"

CAL POLY
SAN LUIS OBISPO

PROJECT:
HIGHBAY LAB MODULAR TESTING BLOCKS

SKETCH NO.
MJD

ISSUE DATE:
6/8/2021 8:39:47 PM

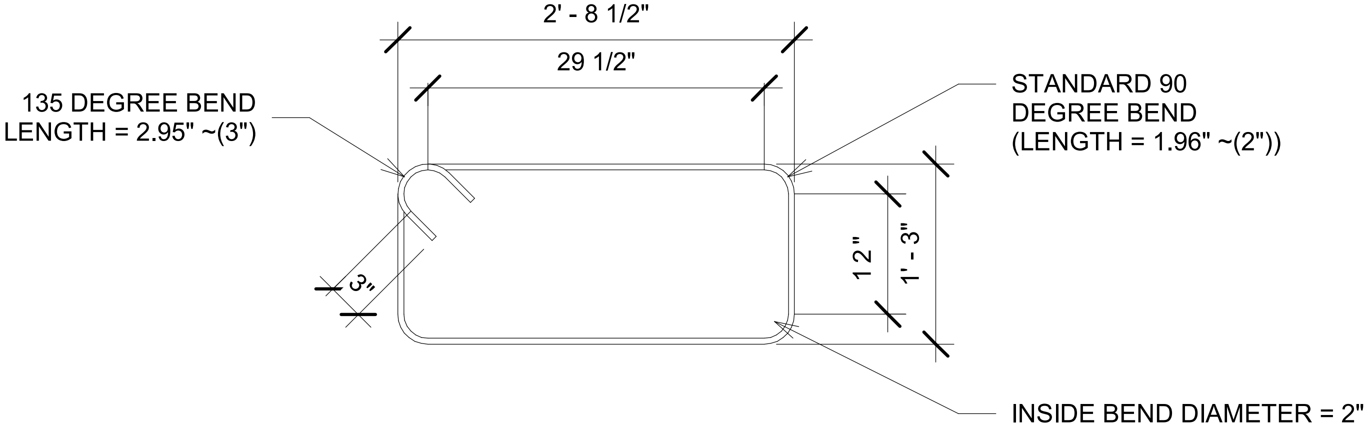
SCALE:
1/2" = 1'-0"

SKETCH TITLE:
MATERIAL TAKE OFF

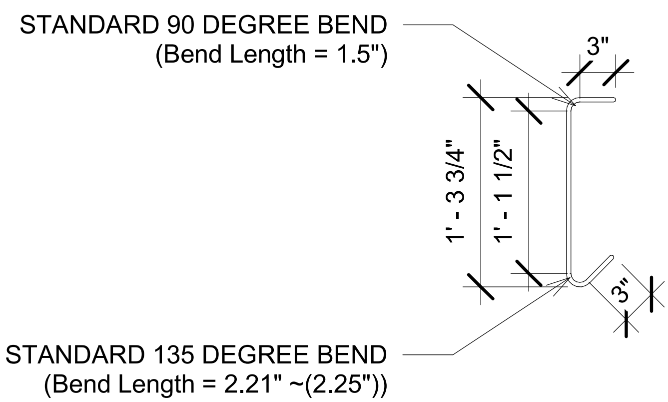
DWG REF.
S3

BAR BEND LENGTHS

NO. 4 STEEL



NO. 3 STEEL



NOTE: ALL BEND DIMENSIONS ARE BASED ON STANDARD BAR BEND LENGTHS AS PER ACI 318-19 TABLES 25.3.1 AND 25.3.2

1 Rebar Specifications
3/4" = 1'-0"

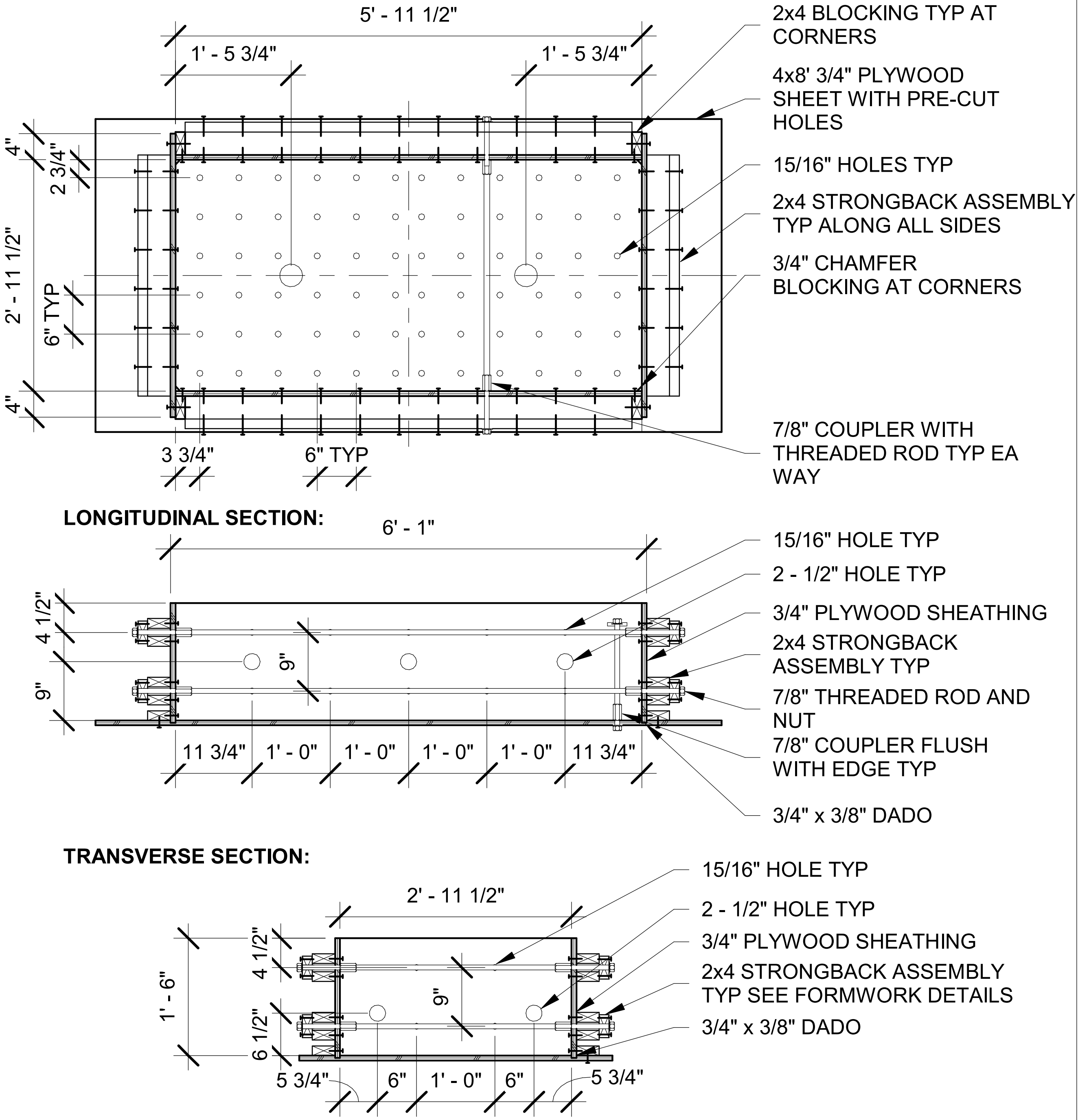
CAL POLY
SAN LUIS OBISPO

PROJECT: HIGHBAY LAB MODULAR TESTING BLOCKS		SKETCH NO. Checker
ISSUE DATE: 6/8/2021 8:39:47 PM	SCALE: 3/4" = 1'-0"	
SKETCH TITLE: REBAR SPECIFICATIONS		DWG REF. S3b

FORMWORK PLAN VIEW:

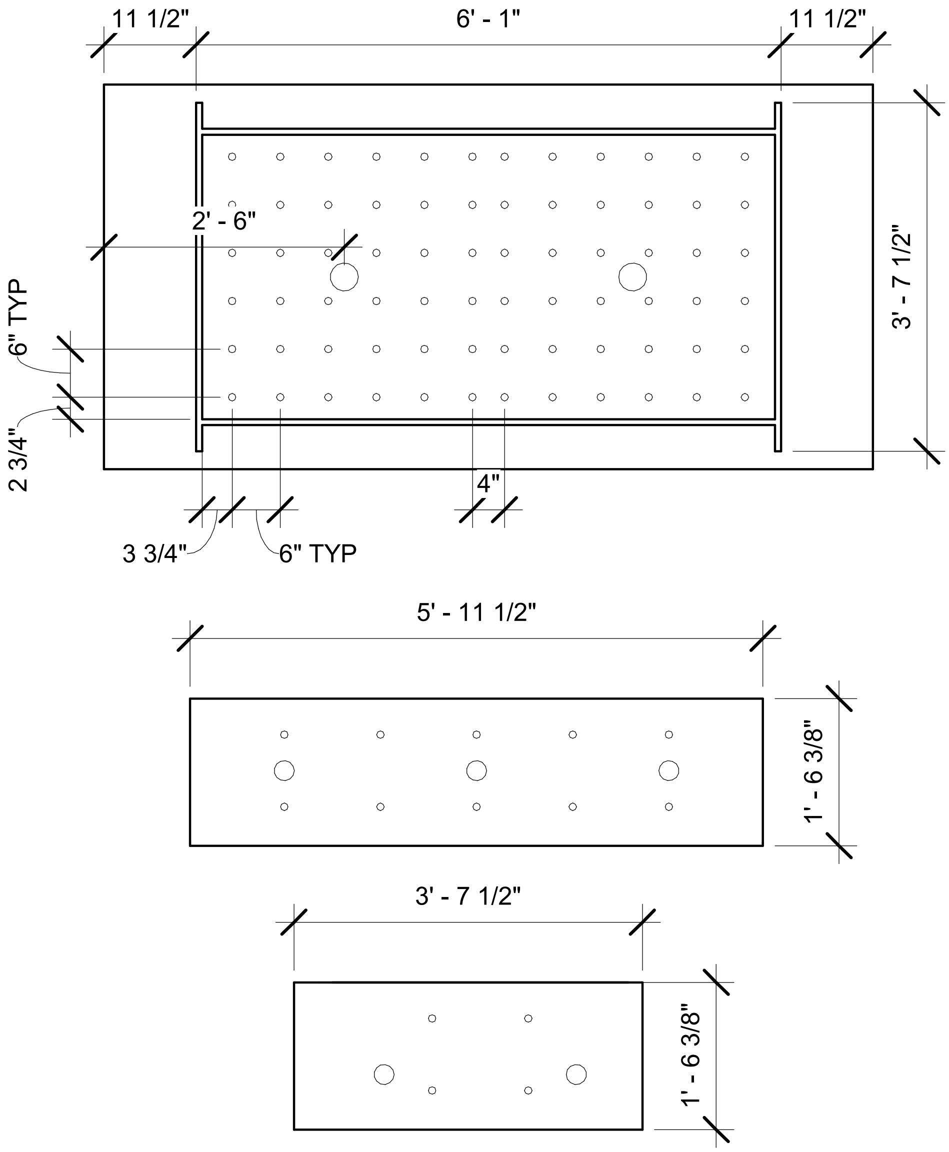
LONGITUDINAL SECTION:

TRANSVERSE SECTION:



1 Modular Block Formwork Strongbacks
1/2" = 1'-0"

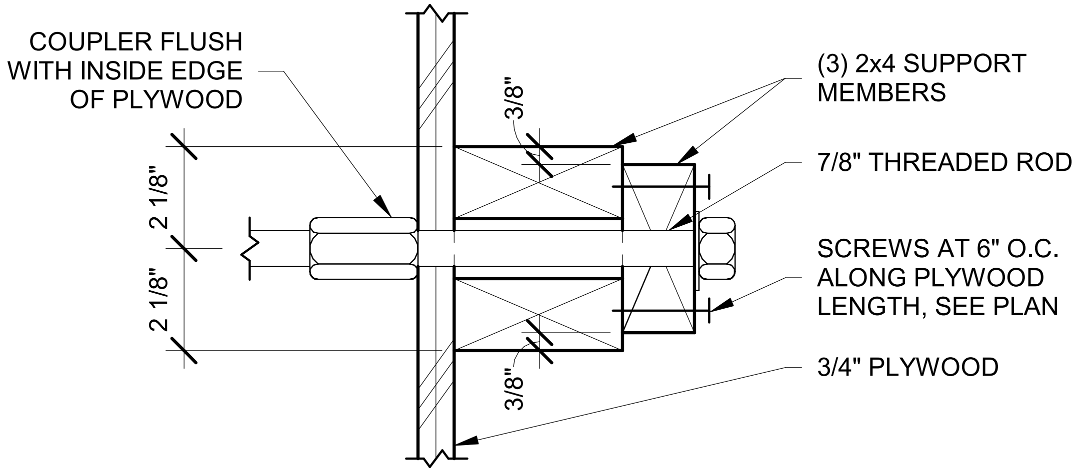
CAL POLY SAN LUIS OBISPO	PROJECT: HIGHBAY LAB MODULAR TESTING BLOCKS	SKETCH NO. Checker
	ISSUE DATE: 6/8/2021 8:39:48 PM — SCALE: 1/2" = 1'-0"	
	SKETCH TITLE: BLOCK FORMWORK STRONGBACKS	DWG REF. S4



1 Formwork CNC
1/2" = 1'-0"

CAL POLY
SAN LUIS OBISPO

PROJECT: HIGHBAY LAB MODULAR TESTING BLOCKS		SKETCH NO. Checker
ISSUE DATE: 6/8/2021 8:39:48 PM	SCALE: 1/2" = 1'-0"	
SKETCH TITLE: CNC FORMWORK DIMENSIONS		DWG REF. S4c



1 STRONGBACK ASSEMBLY
3" = 1'-0"

CAL POLY
SAN LUIS OBISPO

PROJECT: HIGHBAY LAB MODULAR TESTING BLOCKS

SKETCH NO. Checker

ISSUE DATE: 6/8/2021 8:39:48 PM

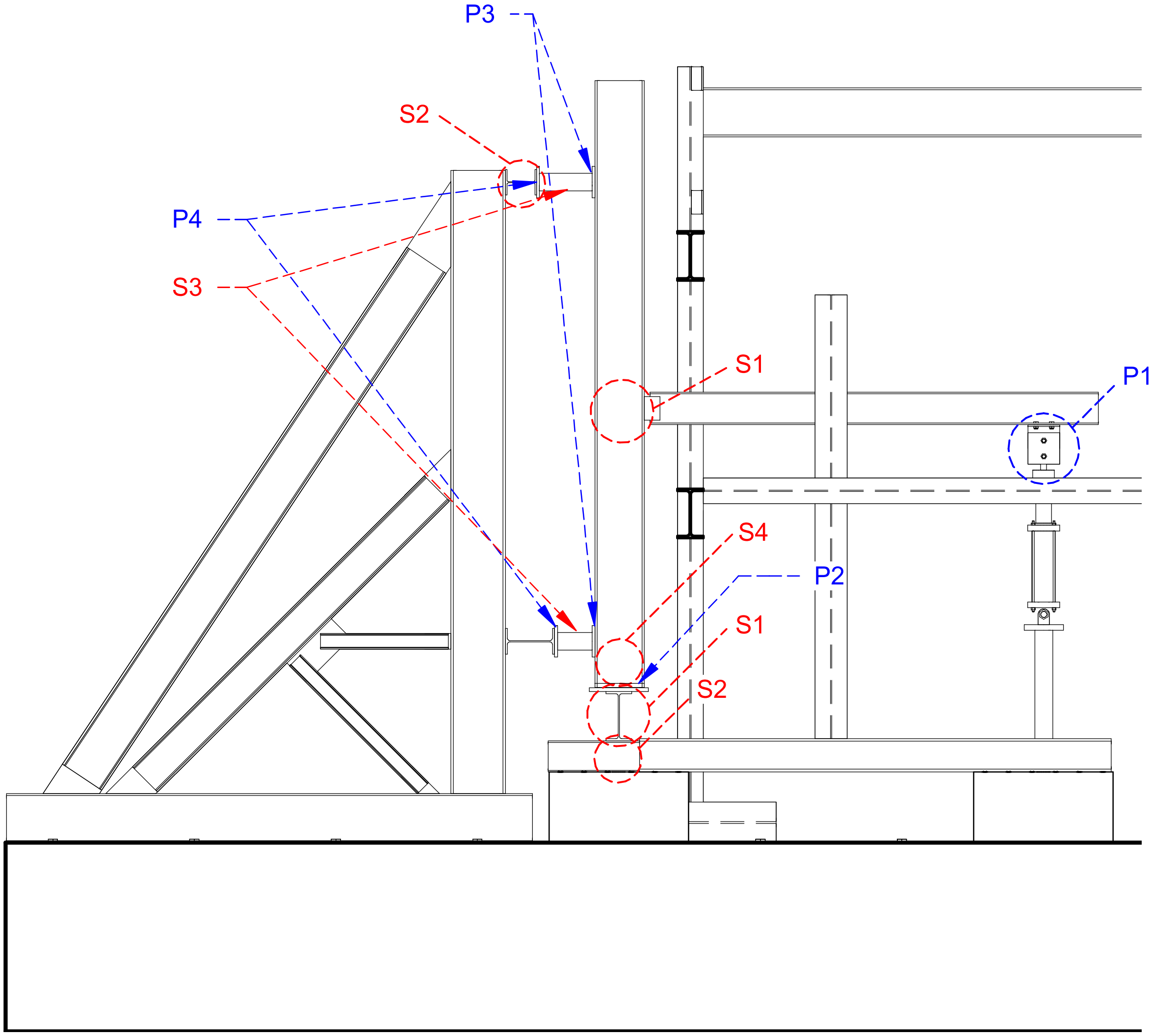
SCALE: 3" = 1'-0"

SKETCH TITLE: FORMWORK DETAILS

DWG REF. S5

ANDERS JOHNSON
THOMAS LITTLE

Appendix A2.2: Test Jig Drawings



1 Plate Callouts
1/4" = 1'-0"

CAL POLY
SAN LUIS OBISPO

PROJECT:
GERT CONNECTION TESTING

ISSUE DATE:
6/10/2021 6:57:20 PM

SCALE:
1/4" = 1'-0"

DRAWN BY:
MJD
AMJ
TJL

SKETCH TITLE:
PLATE STEEL ORDER KEY

DWG REF.
S10

CALLOUT	DESCRIPTION	THICKNESS	DIMENSIONS	NUMBER REQUIRED
P1	BEAM TO ACUATOR CONNECTION	3/4"	4"x8"	x2
P2	COLUMN BASE PLATE	3/4"	12-1/2"x8"	x1
P3	COLUMN TO PIPE PLATE	3/4"	8"x9"	x2
P4	PIPE TO REACTION FRAME PLATE	3/4"	8"x9"	x2
	TOTAL LENGTH OF 3/4" PLATE REQUIRED (8" WIDTH)		**56.5"** (4' - 8 1/2")	
S1	W12x35 STIFFENERS	3/8"	11-1/2" x 3-1/8"	x4
S2	W8x24 STIFFENERS	3/8"	3-1/8"x7-1/8"	x4
S3	RADIAL PIPE STIFFENERS	3/8"	3" x 4"	x8
S4	COLUMN BP STIFFENER	3/8"	8" x 3-1/8"	x2
	TOTAL LENGTH OF 3/8" PLATE REQUIRED (8" WIDTH)		**47.75"** (3' - 11 3/4")	

1 Plate Steel Order
1" = 1'-0"

CAL POLY
SAN LUIS OBISPO

PROJECT: GERT CONNECTION TESTING

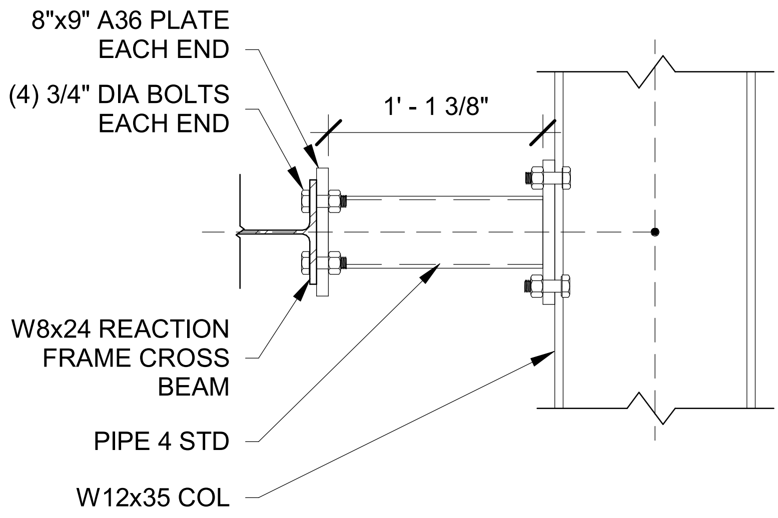
ISSUE DATE: 6/10/2021 6:57:20 PM

SCALE: 1" = 1'-0"

DRAWN BY: MJD AMJ TJL

SKETCH TITLE: PLATE STEEL ORDER

DWG REF. S9



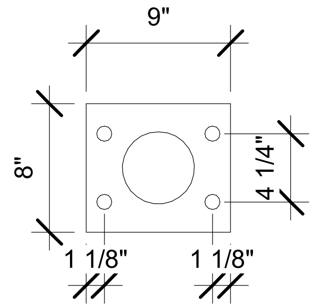
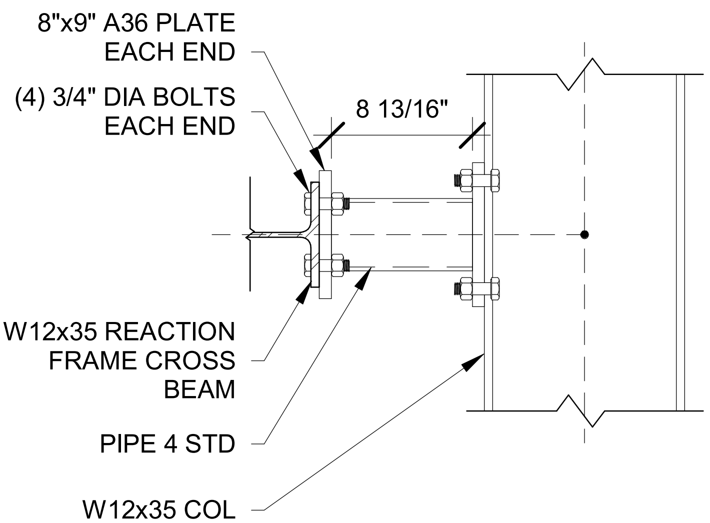


PLATE DETAIL

1 COLUMN TO REACTION FRAME AT TOP
1" = 1'-0"



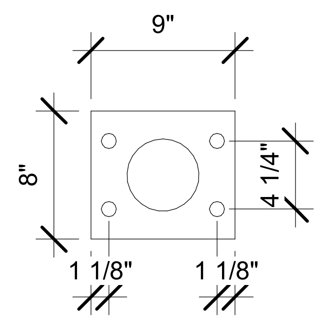
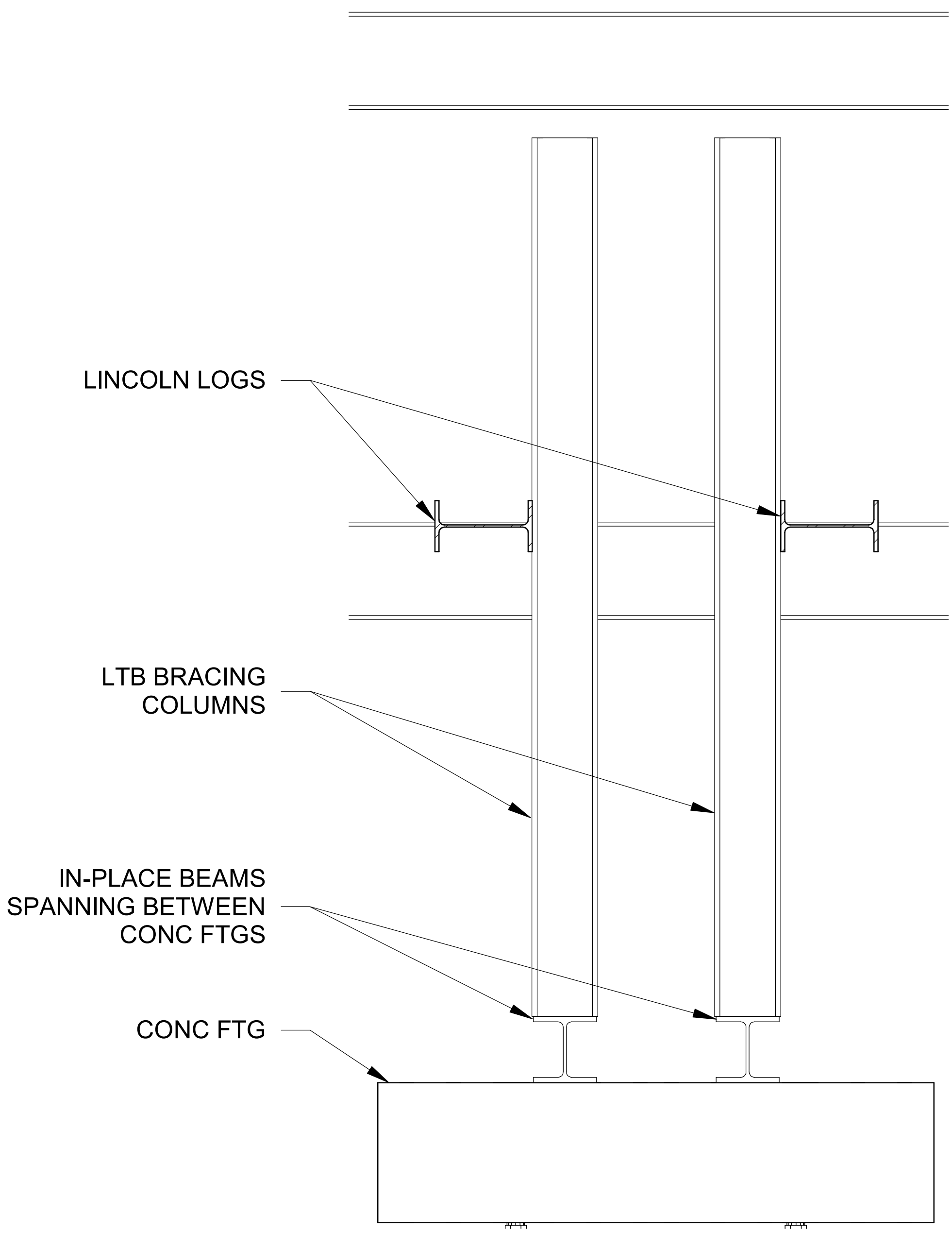


PLATE DETAIL

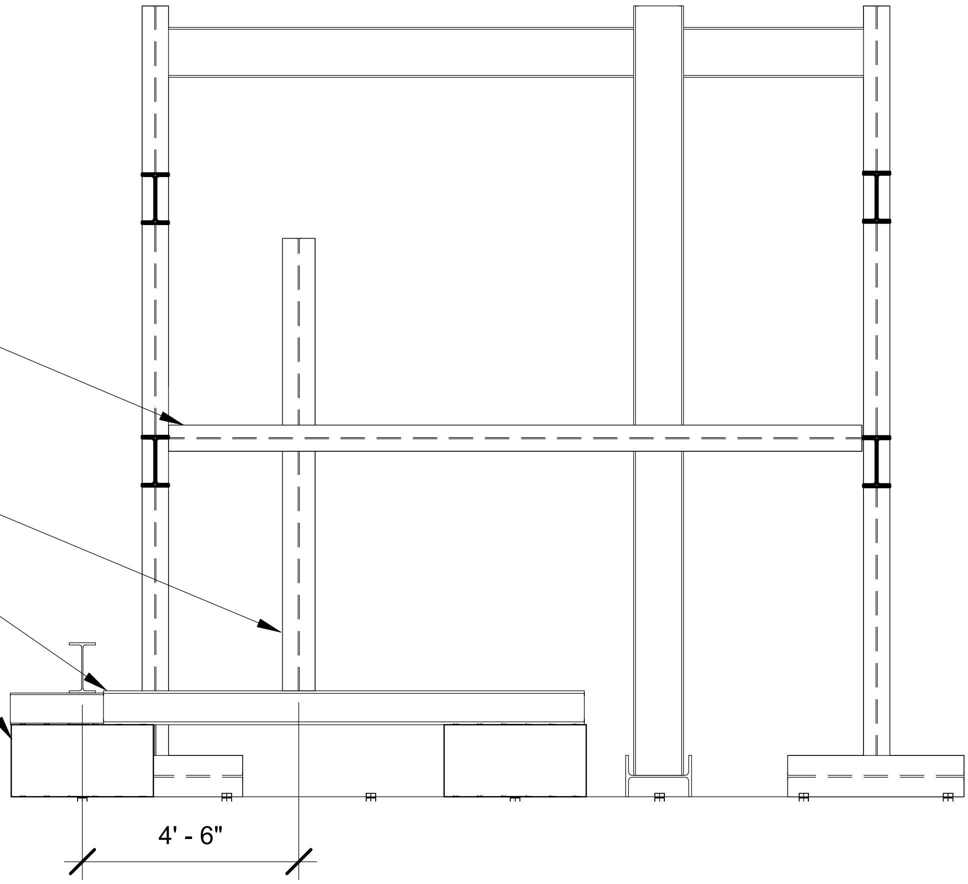
2 COLUMN TO REACTION FRAME AT BASE
1" = 1'-0"

CAL POLY SAN LUIS OBISPO	PROJECT: GERT CONNECTION TESTING		DRAWN BY: MJD AMJ TJL
	ISSUE DATE: 6/8/2021 8:39:48 PM	SCALE: 1" = 1'-0"	
	SKETCH TITLE: COLUMN TO REACTION FRAME DETAILS		DWG REF. S12



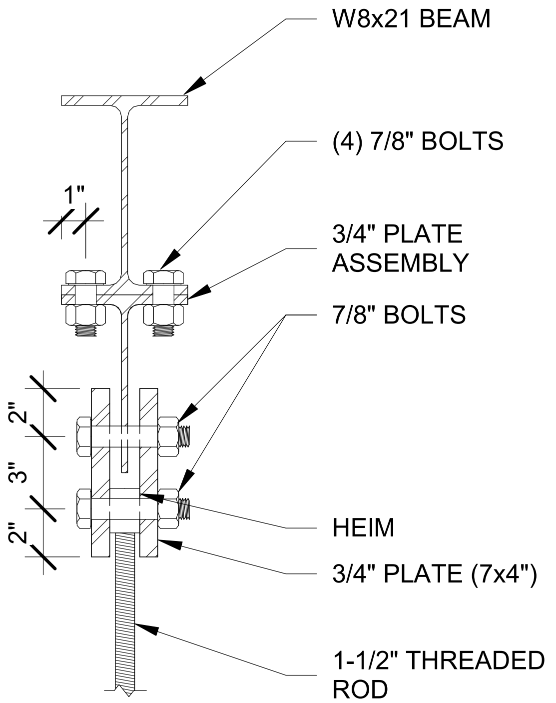
1 LTB NORTH ELEVATION
1/2" = 1'-0"

CAL POLY SAN LUIS OBISPO	PROJECT: GERT CONNECTION TESTING		DRAWN BY: MJD AMJ TJL
	ISSUE DATE: 6/8/2021 8:39:49 PM	SCALE: 1/2" = 1'-0"	
	SKETCH TITLE: LTB BRACING ELEVATION		DWG REF. S13

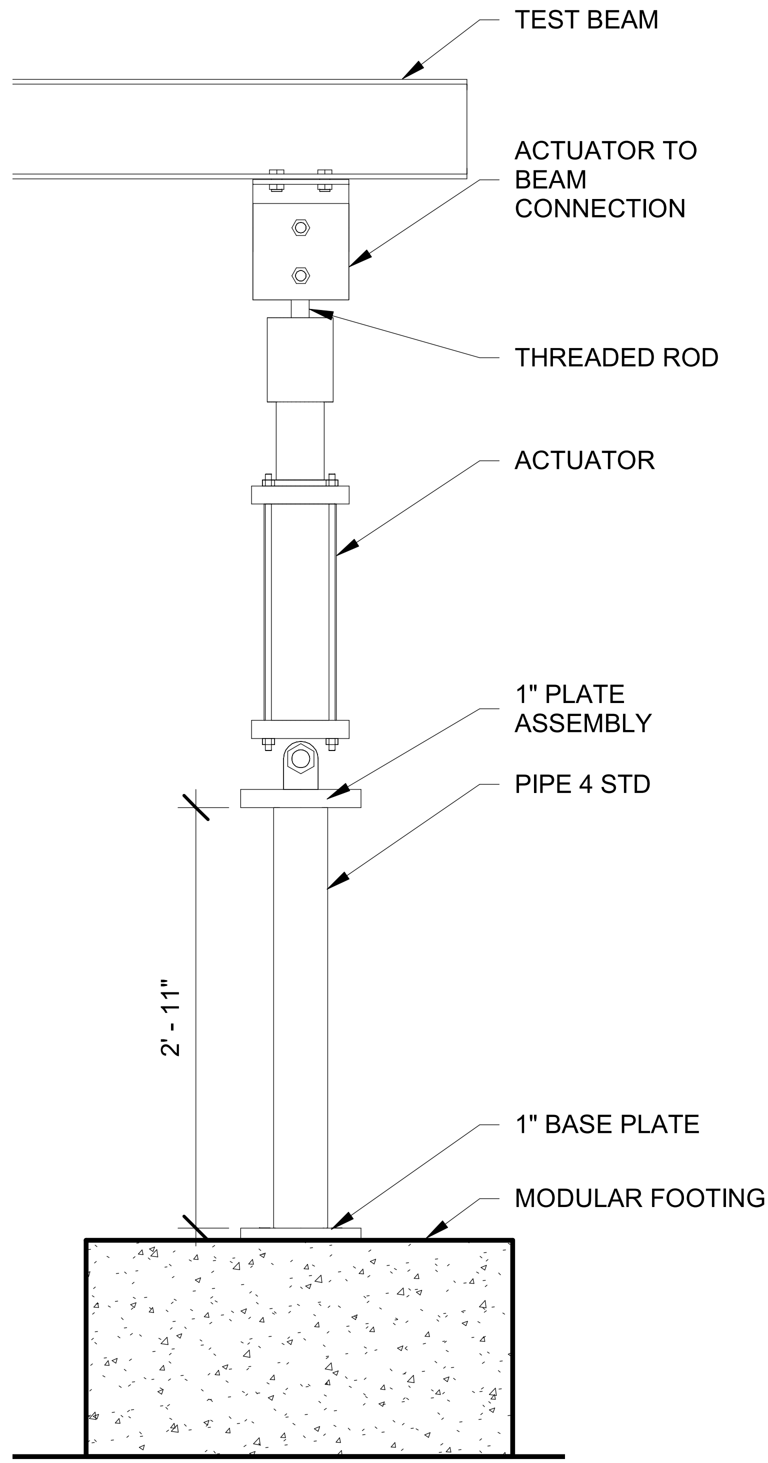


1 LTB EAST ELEVATION
1/4" = 1'-0"

CAL POLY SAN LUIS OBISPO	PROJECT: GERT CONNECTION TESTING		DRAWN BY: MJD AMJ TJL
	ISSUE DATE: 6/8/2021 8:39:49 PM	SCALE: 1/4" = 1'-0"	
	SKETCH TITLE: LTB BRACING ELEVATION		DWG REF. S14



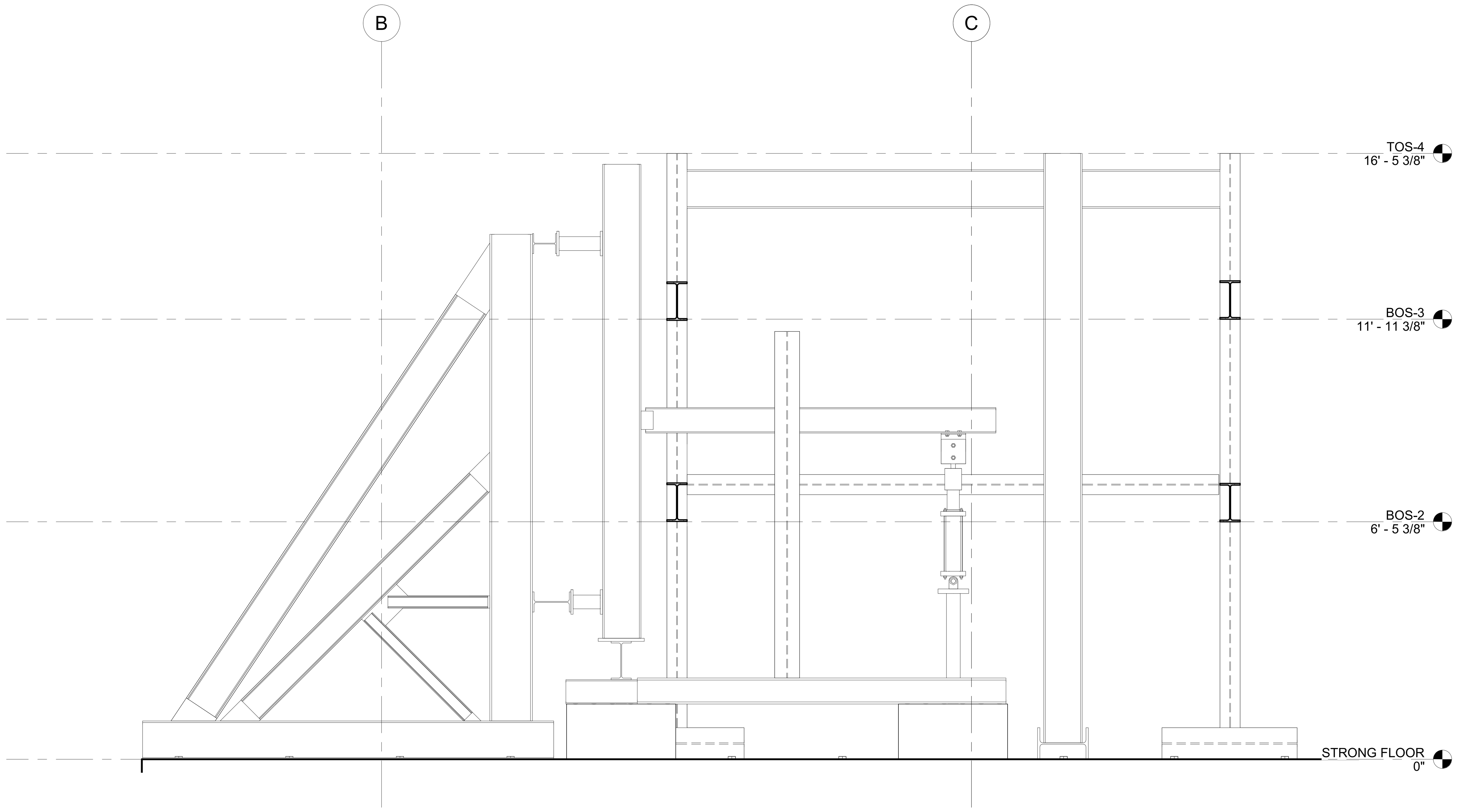
1 BEAM TO THREADED ROD
1 1/2" = 1'-0"



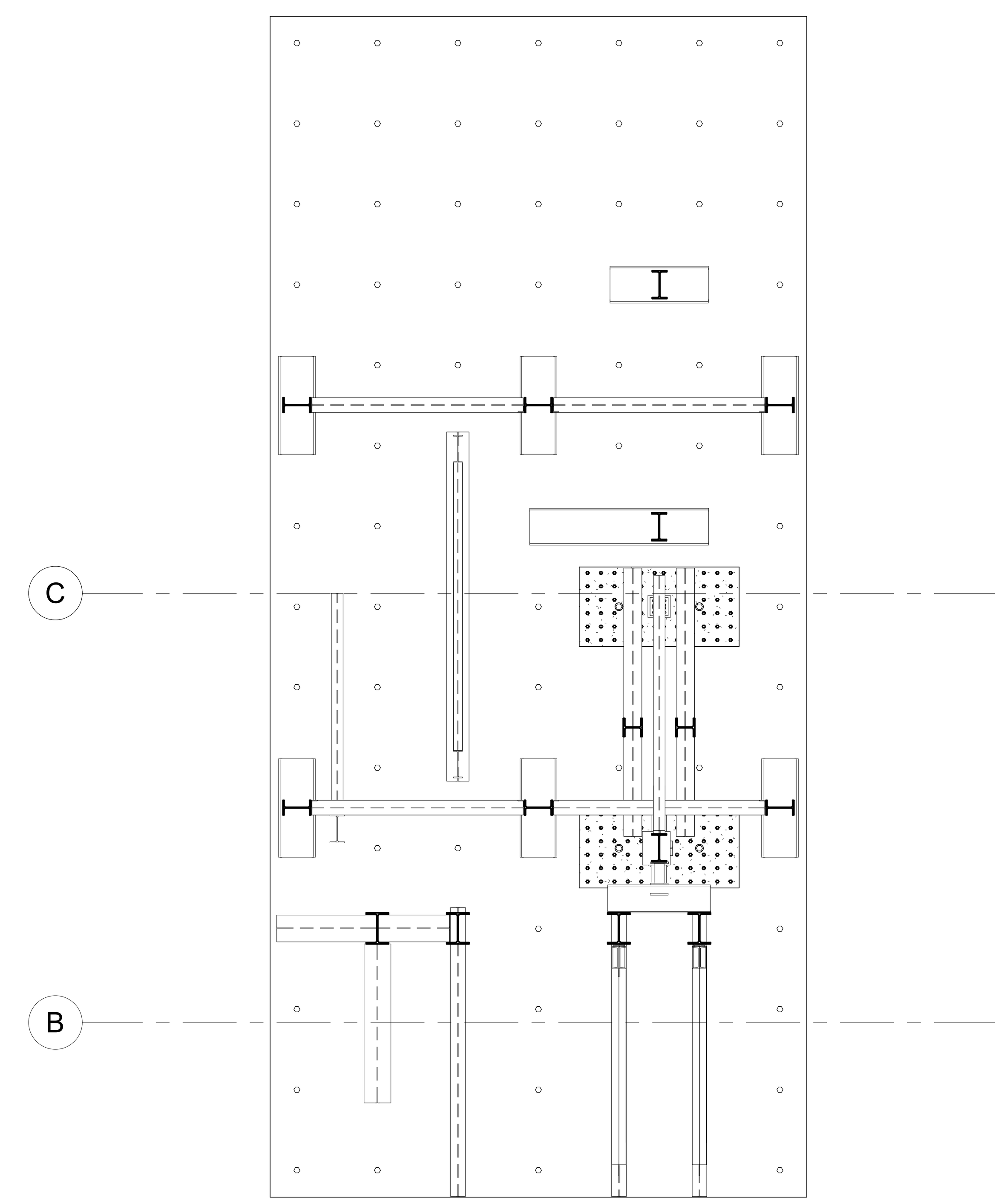
2 ACTUATOR CONNECTION ASSEMBLY
3/4" = 1'-0"

CAL POLY
SAN LUIS OBISPO

PROJECT: GERT CONNECTION TESTING		DRAWN BY: MJD AMJ TJL
ISSUE DATE: 6/8/2021 8:39:49 PM	SCALE: As indicated	
SKETCH TITLE: ACTUATOR CONNECTIONS		DWG REF. S18



1 TEST JIG ELEVATION
1/2" = 1'-0"



2 TEST JIG PLAN VIEW
1/4" = 1'-0"

CAL POLY
SAN LUIS OBISPO

Project:

HIGH BAY LAB STEEL CONNECTION TEST JIG

Sheet Name:

TEST JIG ELEVATION

Date: 6/8/2021 8:41:03 PM

Drawn By: MJD, AMJ, TJL

Scale: As indicated

Sheet No.

S19