Structural Analysis of Overhead Crane in Simpson Strong-Tie Laboratory

http://www.omnidesigngroup.com/currentprojects-cpsimpson.aspx

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Introduction

Since its opening in 2010, the Cal Poly's Simpson Strong-Tie Material Demonstration Laboratory has been used for multiple activities by the College of Architecture and Environmental Design. Currently, the bottom floor is used for Construction Management laboratory classes. These classes allow students to practice construction techniques at full scale. The size of these projects is problematic when assemblies need to be moved or when the finished structure needs to be removed from the building. For example, one class builds a small portion of a traditionally constructed house. The roof for this structure is constructed on the ground and lifted into place atop walls. Currently, the lifting is accomplished with the student's brute force and a forklift, which is ineffective, dangerous, and requires advance coordination with the Facilities Department. Installing a crane overhead would allow projects to be maneuvered safely.

Scope of the Report

This report investigates the existing building's structural ability to accommodate an overhead crane. There are several ways to add an overhead crane to a building. This report investigates attaching a crane to the roof beams. For an explanation of the decision, see the Estimate of Existing Capacity section. The ability to support a crane is dependent on the available capacity of the existing structure. Specifically, this report considers the capacity of the existing roof and columns for the new crane loading. The concrete walls and foundations were not evaluated because of impending and unknown loading from cantilever load racks. The analysis is also limited to the gravity system. The crane represents about a 7% increase in the weight of the roof. Therefore, it is likely the lateral system has the capacity for the increased forces. The report only considers the structural impact of a crane and does not consider electrical or MEP systems.

This report also covers working in an interdisciplinary design team and managing a project. Working collaboratively requires professional communication and time management. The techniques used helped ensure timely completion of the project despite the lack of structured class time.

Report Overview

The final deliverable for this senior project was a set of calculations that will be used in the proposal process to Cal Poly Facilities. The report opens with a discussion of project management techniques for small projects and how they were applied to create the calculation package. Then the report outlines an analysis procedure for building additions illustrated by examples from this project. The procedure begins with the verification of existing conditions, followed by estimating the existing capacity to decide which design to pursue. Once the general direction is decided the new loading needs to be defined. Then, if all the previous steps indicate the addition is possible, the report describes how complete the final design.

Project Management

A successful project depends not only on the individual skills of the team members, but also on their ability to work together towards a common goal. A well-managed project facilitates collaboration through communication of clear objectives. The management process for small projects can be outlined through the following steps:

Management Process for Small Projects

Initial Tasks

- Choosing Design Team
- Kick-Off Meeting
    - Project Defined
    - Project Deliverables and Due Dates
    - Establish Methods of Communication
- Determine Project Priorities

During Project

- Communication Between Team Members: RFI Process
- Scheduling
    - Schedule
    - Weekly Status Reports

Brief Descriptions of the Steps:

Design Team

Traditionally, a design team consists of an owner, contractor, architect, and engineer. Each member contributes their specific expertise to the project. The design team for the Simpson Strong-Tie building project consisted of Amy Poehlitz, as structural engineer, and Paul Redden, as both owner's representative and contractor. The project did not need an architect. An architect's role might have included aesthetic decisions or maintaining accessibility and egress.

Kick-Off Meeting

A successful project needs a kick-off meeting. This meeting clearly defines the scope of the project and the responsibilities of each team member. Responsibilities could include deliverables at specific due dates or expectations for communication throughout the project. This project was defined as the structural impact of installing an overhead crane in the Simpson Strong-Tie Building. The crane should access the first six bays of the bottom floor and need not extend outside of the building perimeter. It was agreed the final deliverable would be a set of calculations created by Amy Poehlitz. The calculation would form part of a proposal by the Construction Management Department to the Cal Poly Facilities Department. Communication between the engineer and owner-contractor would mainly occur through weekly status reports. There would be weekly meeting between the engineer and her advisor.

Project Priorities

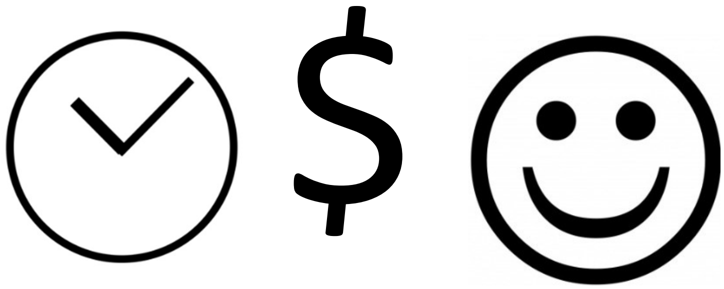


Figure 1: The Three Project Priorities

At the start of a project, it is important to define the client's priorities so the team knows what to focus on. A project can only optimize two of the three priorities:

- Time
- Cost
- Quality

For example, a project can have a short timeline and low cost but the deliverable won't be good quality. On this project, time and quality were priorities. Time was required because the project had to be finished in a quarter. Quality was required because the calculations were going to be used for the internal Cal Poly approval process. In an actual firm, the project would have had a premium on the price because the high likelihood of additional work to stay on schedule. The additional work is often finding alternative ways to get information that another team member is behind schedule in providing. For example, in this project the crane vender had not delivered their quote and I needed the load information to evaluate the existing structure. To stay on schedule, I sized all the crane members to get a load estimate. If the project had a longer timeline I could have waited for the quote and saved the extra work. For more information on the process of estimating loading see the Estimating New Loading in Projects with Strict Deadline subsection in New Loading.

Communication

Methods of communication are as varied as projects. This project used weekly status reports for the majority of project communication. Status reports are used in different forms in many different industries. For this project, the reports included:

- A summary of any meetings that occurred that week
- Action items for each team member
- Progress on action items from the previous weeks

Meeting notes are valuable to update team members not at the meeting and to act as reference material for meeting attendees. The action items provide discrete goals that can be updated on a weekly

basis. The flexibility lets the goals reflect the actual progress of the project, compared to the rigid overall schedule.

The weekly status reports were also used in place of RFI (Request for Information) procedures. RFIs are a set procedure used for information transfer between team members. RFIs were not used due to the size of the project. In large projects, RFIs offer a way to organize the large and varied amounts of information shared. In a small project, like this, a less formal system is sufficient and the exchange of information happens weekly, either in meetings or over email.

For more information on RFIs:

Brazee, Art. " The Anatomy of a Request for Information (RFI)." *Jobsite*. Procore Technologies, Inc, 29 Mar. 2016. Web. 14 June 2017.

Scheduling

Scheduling is designed keep a project on track, by making agreed deliverable dates for all to see. Reverse scheduling is often useful for projects with hard deadlines. A reverse schedule is created working backward from the completion date instead of forward from the present. It is recommended to include flux time in the schedule to allow the project to recover from unforeseen complications. Reverse scheduling was used for this project because the deadline of the end of the academic quarter could not be moved.

Schedules for projects are dependent on actions of all team members.

- Reverse Scheduling: used with a deadline that cannot be moved
- Information Based Scheduling: individual discipline progress is dependent on the results of other disciplines (team members). This type of scheduling benefits from the use of a facilitator. This facilitator is commonly seen as a construction manager (as opposed to construction contractor).
- Goal Based Scheduling: benefits of meeting short terms goals are used as the basis for deadlines. For example, seasonal changes can affect a project.

The above list of methods of forming a schedule has not been vetted, but is the experience of the advisor.

Often, one team member needs another member’s task to be completed before they can start their own. For example, the architect might need information from the client. Instead of declaring a hard deadline, the architect can ask if it works with the owner’s schedule. This allows gives the owner flexibility. Another approach is to communicate why the information is needed quickly. Often, the owner is invested in completing the project on schedule.

Structural Analysis

Working within an existing building brings additional challenges. Significant amounts of time go into verifying the existing structure before the design can begin. The structural analysis process for additions is outlined through the following steps:

Structural Analysis Process for Additions

- Verify Existing Conditions (discussed on page 8)
    - Structural Members
    - Connections
    - Loading Conditions
- Estimate of Existing Capacity (page 11)
- New Loading and Loading Location on Existing Structural Elements (page 12)
    - Estimating New Loading in Projects with Strict Deadlines
- If above conditions positive, than final design (page 15)
    - Check of Existing Structure's Capacity
    - Design of Connection between Existing Structure and Addition
- If above negative structure is not a good candidate for proposed addition/remodel

Verification of Existing Conditions

The existing state of a building needs to be understood before the effect of any additions can be analyzed. This process includes determining the existing structural members, how those members are connected, and what loading is placed on the members.

This section compares the existing conditions in the Simpson Strong-Tie Building to as-built drawings obtained from California Polytechnic State University, San Luis Obispo dated February 2011. The plans were obtained through Cal Poly's construction drawing archive, Web-Planroom. The existing building has a timber framed roof (heavy timber joists and glulam beams) supported on pre-engineered lumber parallam columns. These columns are supported on a concrete wall which rests on a continuous spread footing. The interior of the building can be seen in figure 2.

Figure 2: Interior of Simpson Strong-Tie Building

Roof Member Sizes:

Sizes of the roof members were inspected visibly and appeared in accordance with the as-built plans. The exposed nature of the roof made it easy to inspect the structure.

Connections:

The connections were also inspected visibly and appeared to match the details in the as-built drawings.

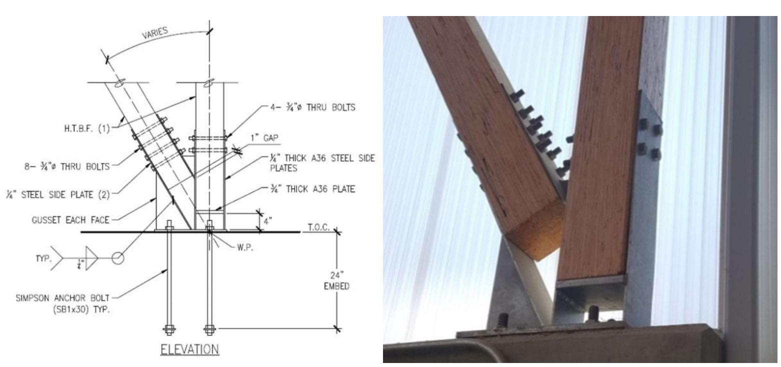


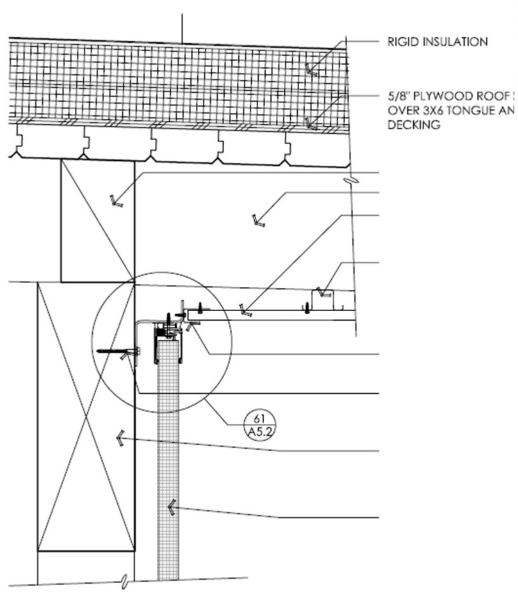
Figure 3: Drafted Detail and Existing Connection

Roof Loading:

The use of the roof has not changed so a live load of 20psf for maintenance is still appropriate.

The general notes state the building was designed for a total roof loading of 20psf in areas without the green roof and 32 psf in areas with the green roof. The green roof was never installed. The layers of the roof could not be verified by inspection but it could be confirmed there is wood decking as the base layer and PVC roofing as the top layer. It is reasonable to assume the roof was constructed as shown in the details in figure 4. Based on the as-built drawings, a take-off of the roof might include:

80 mil PVC Roofing	0.5 psf
Rigid Insulation	1.5 psf
5/8" Plywood Roof Sheathing	1.8 psf
3x6 Douglas Fir Decking	7.6 psf
Sprinklers	5.2 psf
MEP	2 psf
Total	18.6 psf



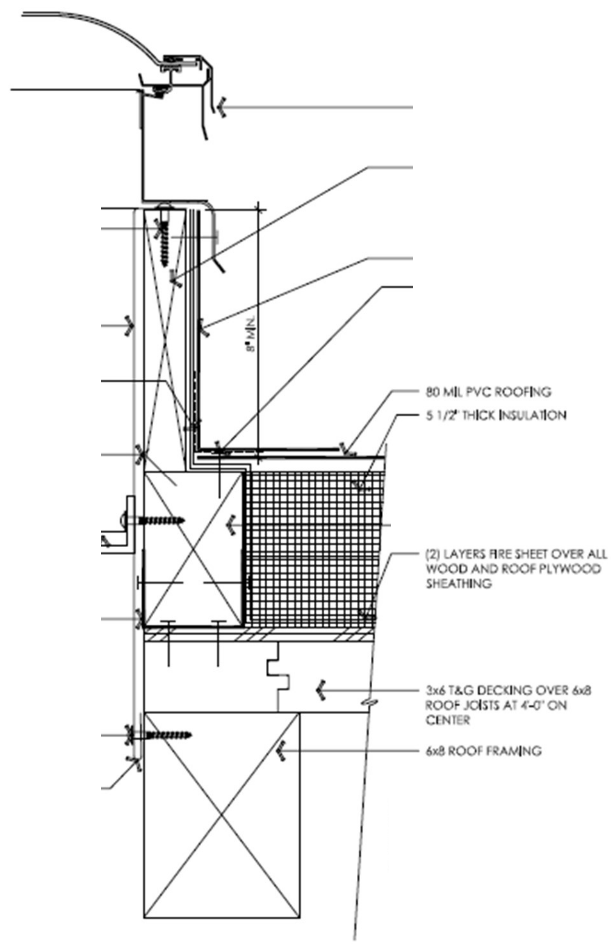


Figure 4: Roof Construction Details

Based on the details in Figure 4, a dead load of 20psf is reasonable. The dead load calculation does not consider is the weight of the skylights. These loads will be applied separately.

An interesting loading condition on the roof is the presence of personal safety restraints (see figure 5). These are hooks that maintenance workers can use while on the roof to attach themselves and hang from the building. These loads transfer directly to the glulam beams.

Figure 5: Personal Safety Restraints

Drainage also needed to be considered when determining existing loads on the roof. If ponding occurs, the weight of the water can quickly become significant. The Simpson Strong Tie Building does not have a problem with debris build up, as there are no nearby overhanging trees to deposit debris on the surface. However, after rain there is a small amount of pooling near the down sprouts (see Figure 6).

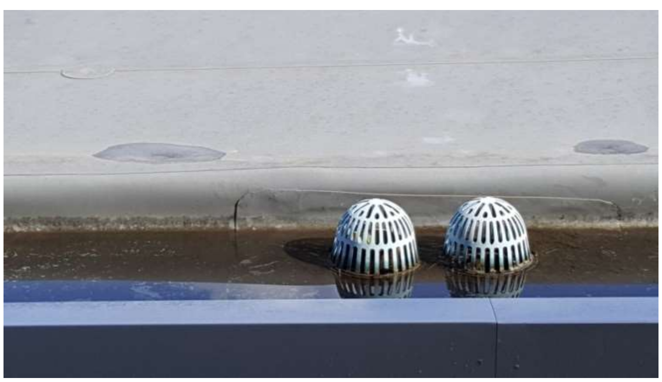


Figure 6: Pooling Near Drain Pipes

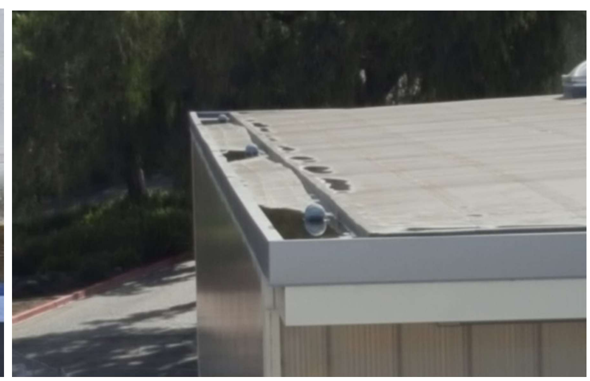


Figure 7: Extent of Pooling Relative to Roof Area

The water is not deep and the nets protecting the down sprouts seem clear of debris, which limits the possible depth of puddling. The sitting water is a result of the roof sloping slightly toward the edge of the roof instead of in toward the drainpipes. The puddling only occurs near the downspouts and only within the gutter channels on the edge of the roof. The limited extent of the pooling can be seen in Figure 7 on the next page. The loading effect of these puddles will be neglected because their shallow depth and narrow extent results in limited impact on the capacity of the structural members.

Estimate Existing Capacity

Often there are multiple ways that an addition can be attached to an existing structure. To decide which option to investigate, start by estimating whether the structure has the capacity for each option. This will reduce the time spent pursuing options that will not work. After the options have been reduced to only those that are plausible the final selection should be based on other factors. These factors will vary from project to project but may include minimal impact to the structural system, minimal impact to usable space, and least cost. If after further investigation the first option is unworkable, the other options can be returned to.

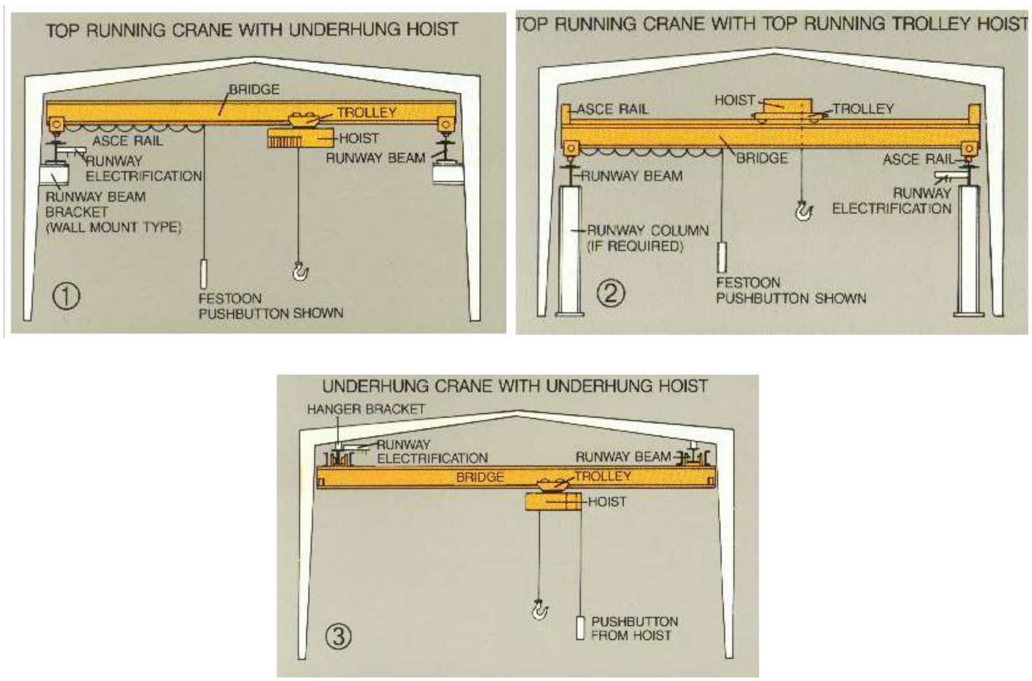


Figure 8: Possible Methods for Attaching Overhead Crane to Structures

The addition for this project was an overhead crane. There are several ways to add an overhead crane to a building as shown in figure 8:

- attaching the crane runways to the columns
- installing separate columns (stanchions) that would support the crane completely apart from the existing structure
- attaching the runways to the roof

A quick calculation shown on page 31 in Appendix E showed the existing columns did not have the capacity for the eccentric loading in option one. The separate frame is possible but not preferred as it would take up floor space and require foundation work. Therefore, this report investigates installing a crane to the roof beams. For this option, the ability to support a crane is dependent on the capacity of the existing structure.

New Loading

Once the general approach to installing the addition has been defined, the impact on the structure needs to be quantified. Often, the impact is a load in a critical location. If the location of the addition is flexible the placement should consider the existing building systems, such as MEP and structural. The ideal location has the minimum impact on the existing systems. For example, the Simpson Strong-Tie building has fire sprinklers installed above where the crane members will be installed. To minimize interference with water coverage, the runways should be placed directly below the first row of sprinklers as shown in figure 9. By placing the beams directly below the sprinklers, the umbrella water sprinkling effect will be minimally impacted.

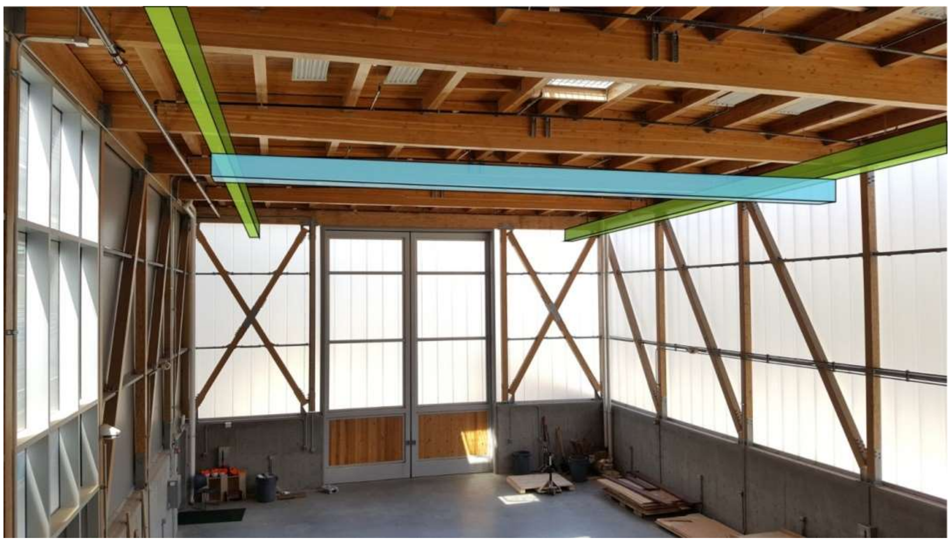


Figure 9: Approximate Location of Crane Members: Runways in Green, Bridge Girder in Blue

Estimating New Loading in Projects with Strict Deadlines

Sometimes tasks in projects are discipline dependent. For example, in this project the new loading could not be fully defined until the quote came in from the crane vender. In projects with longer or more flexible deadlines a delay may have minimal effect. However, this project needed to stay on schedule and meet a hard deadline. Luckily, if information is delayed the situation is often not hopeless, but does require unscheduled or uncompensated effort. Often, the information can be found an alternative way. In this case load information was needed. To increase accuracy, loads can be based on a similar existing condition. For more information on how this effects the project’s cost see the Project Priorities subsection in Project Management.

As mentioned, on this project the crane quote was delayed and analysis of the existing structure needed to begin for the project to stay on schedule. To estimate the load I needed to size the crane members, a task that would also be completed by the crane vender. To keep the numbers realistic, the design was based off an existing crane, the overhead crane in Cal Poly’s High Bay Laboratory, shown in figure 10. The most important value from the existing crane was the deflection criterion. The deflection criterion

for cranes is stricter than the code value because any slope in the beam affects the crane's ability to function. The actual deflection of the crane was found and divided by the span to give a deflection criterion of $\Delta < l/586$, four times stricter than the applicable IBC code value. The criterion is still reasonable, it agrees with the masonry code criteria of l/600, per TMS 402-13 Section 5.2.1.4.1. The masonry code limits deflection to mitigate cracking. The deflection criterion from the existing crane was used while sizing the crane members. The crane members were sized for bending, shear, and deflection.

Figure 10: Overhead Crane in High Bay Laboratory

Shortly after the loads had been calculated, the quote came through from the vender. The estimate was only 60% of the load given by the vender (crane quote in Appendix D). The estimated structure was lighter because a deeper member was chosen for the bridge girder that was more efficient for bending. The vender may have chosen a different, heavier section to limit torsional concerns or to maintain ceiling heights. The difference in values illustrates why designs using estimates should be conservative to limit redesign.

Influence Lines in Sizing Crane Members

The demand placed on members comes from the loading. In this case, determining the loading was complicated by the crane's ability to change positions. Each position is a distinct loading condition. To capture all these conditions, the analysis utilized influence lines. Influence lines show how a reaction or internal force at a specific location changes as a load moves across the structure. For example, an influence line for the moment at the center of the beam, as shown in figure 11, gives the value of the moment in that specific spot for each possible location of the load. This means there could be many different moment influence lines for the same beam. In comparison, moment diagrams give the moment at every different location on the beam for a single load configuration. Influence lines are not the same as considering the increase in forces from the dynamic effects of the crane's movements. The dynamic effects were neglected in this project because the crane has a limiter on its speed to lessen those effects.

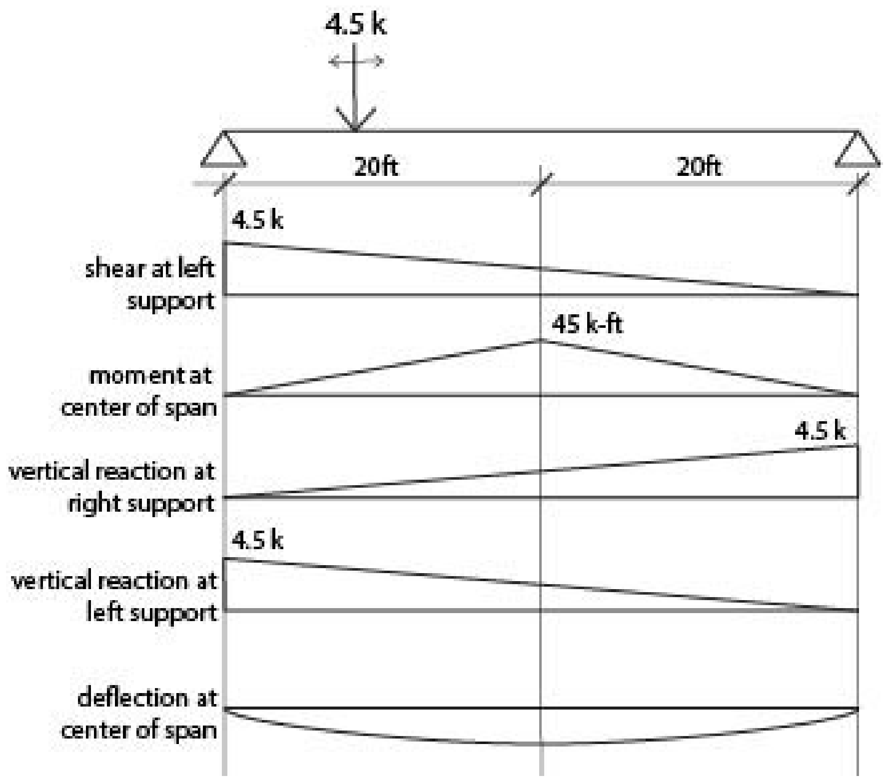


Figure 11: Influence Lines for Bridge Girder

In this project, the moving load was the crane housing and hoist, applied on the bridge girder. Influence lines were found for the moment at the center, the shear at the end, the deflection, and the reactions at the ends. The maximum values from the shear, moment and deflection influence lines were used to size the bridge girder (see figure 11). The vertical reactions were also found and used to load the glulam beams. The influence lines for bending, shear, and the vertical reactions were found using the Müller-Breslau Principle (explanation in Appendix C). The influence line for displacement was found using Maxwell's Reciprocal Displacement (explanation in Appendix C).

Final Design

Once all the work has been done to define the existing structure and new loading the design can be finalized. This includes the final check of the building's capacity as well as any connections that need to be designed between the addition and the existing structure.

Check of Existing Structure's Capacity

To check the existing capacity, follow the load flow through structure and check the capacity of each member affected by addition. For each member check shear, moment, and deflection. The capacity of any connections also needs to be checked.

The crane load enters the structure on the glulam beams. The crane loading is expressed as a function instead of a discrete value because of the movement of the hoist (functions in figure 11). Two loading cases were considered to capture the extremes of the functions, shown in figure 12: the entire load on one hanger and the load split between both hangers. The beam is symmetric so the concentrated load only needs to be considered on one side.

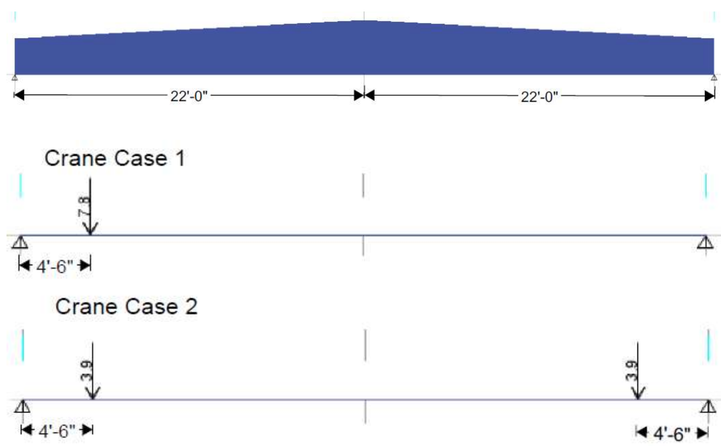


Figure 12: Crane Load Cases on Glulam Beam

The analysis of the beam was furthered complicated by the tapered cross-section. The taper means the moment of inertia changes along the length of the beam. To capture the change, the moment of inertia needs to be expressed as a function in the deflection equation. To avoid an unnecessary complex integration, the deflection of the beam was found by computer structural analysis software (ETABS).

The tapered cross-section also complicated the bending capacity check. Bending was checked using stress. However, the equation to relate moment to bending stress is dependent on the height of the section and the moment of inertia, both which change in tapered cross-sections. Therefore, the place of highest moment, in this case the center of the beam, might not be the location of highest stress. A spreadsheet was made (in Microsoft Excel) that calculated the section properties at increments along the beam. The bending stress was checked at each increment. The graph shown in figure 13 illustrates how the stress changes along the beam. The peak stress occurs approximately twelve feet from the end

of the beam, not at the center where maximum moment occurs. The uncertainty in the location of maximum demand is why demand on changing cross-sections should be checked across the length of the beam. The glulam beams had sufficient capacity in bending. The beam capacity calculations can be found in Appendix E.

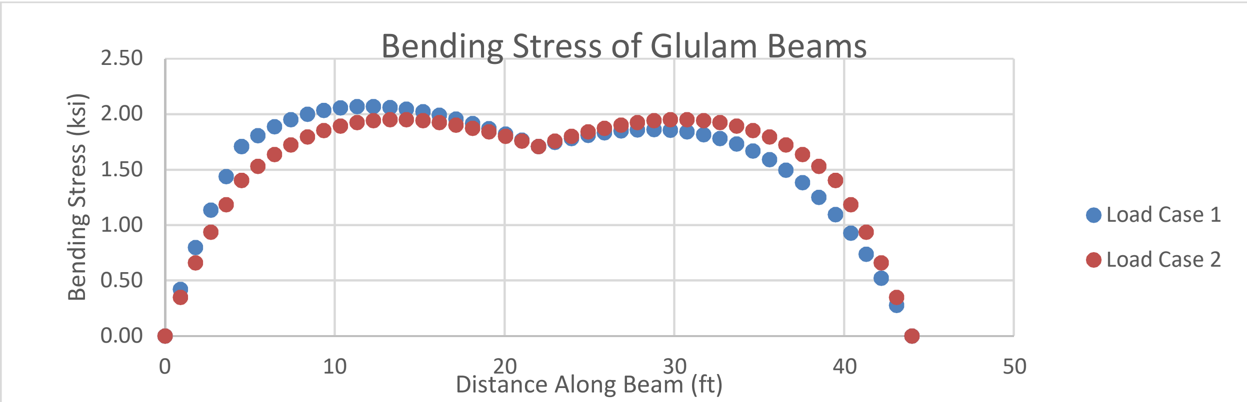


Figure 13: Bending Stress on Tapered Glulam Beams (load cases correspond with figure 12)

The glulam beams are connected to the columns by a bearing connection, shown in figure 14. The governing capacity for this connection was the axial capacity of the parallel strand laminated column.

The column capacity governs and additional loading cannot be applied, such as future storage racks.

The axial capacity of the column was low because its large unbraced length makes it susceptible to buckling. The column had just enough capacity to support the additional weight of the crane. The existing capacity of the structure is governed by the axial capacity of the columns. The conclusion of the analysis is the roof and columns have enough capacity to support the proposed crane. It is noted that the columns were efficiently designed by the original engineer and the additional capacity is most likely due to the green roof loading, which never built.

If the crane is installed, a green roof cannot be installed in the future.

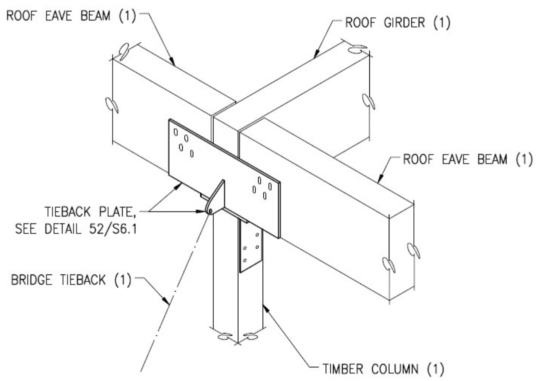


Figure 14: Glulam Beam to Column Connection

Design of Connection between Existing Structure and Addition

Once it has been determined that the structure can support the addition, the connection to the structure can be designed. The following connection design is a suggestion and shows that a connection is possible. A connection is dependent on the actual crane selected to be installed and the connection needs to match the manufacturer's specifications.

The only connection for this addition was between the runways and the glulam beams. The vender's quote only gave a vertical gravity load for the connection (see crane quote in Appendix D). If the connection only took vertical load, it is unclear what would prevent the bridge girder and runways from rotating. A connection needs to resist 2% of a member's axial capacity along the member's axis to prevent the member from failing in torsion per AISC 318. To account for the uncertainty in the load flow, two connections were designed, one that only resisted gravity load, shown as connection one in figure 15, and a connection that provides torsional resistance to the bridge girder, shown as connection two in figure 15. The connection that provides torsional resistance is much wider than the connection only resisting gravity. The lateral load resisted by connection two created a large moment because of the eccentricity between the connection and the bridge girder. That moment is resisted by the couple formed by two lines of the bolts. The connection had to be wide to reduce the forces on each line of bolts to allowable levels.

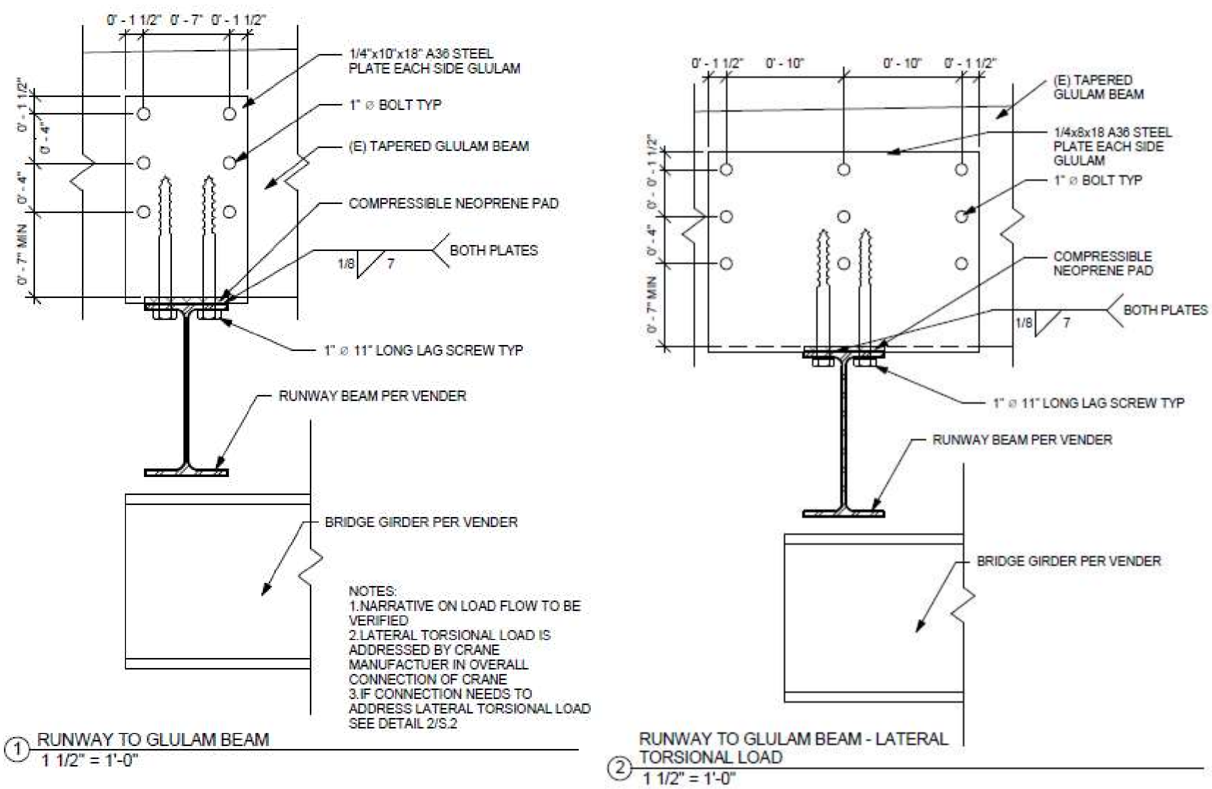


Figure 15: Initial Connection Design

The design of both connections was governed by the bearing of the bolts in the glulam beams. This was expected because the loading was perpendicular to the grain; the direction timber is the weakest. Connections in glulam beams under constant stress that load the beam perpendicular to the grain tend to fail over time through the delamination of layers. The initial plan to prevent this was to install two lag screws directly connecting the flange of the runways to the glulam beam, shown in figure 15. The lag screws were sized to carry the entire unfactored gravity load as a backup system separate from the plate connection. However, the failure area of the lag screws interfered with the through bolts. It was decided that instead of adding a separate back up system, a system to prevent delamination should be included. A strap was added on each side of the connection that passes around the entire perimeter of the glulam, shown in figure 16. The strap will hold the layers of the glulam together, not allowing them to delaminate. The strap will need to be sized to accommodate the swelling and shrinking of the wood throughout seasonal humidity changes.

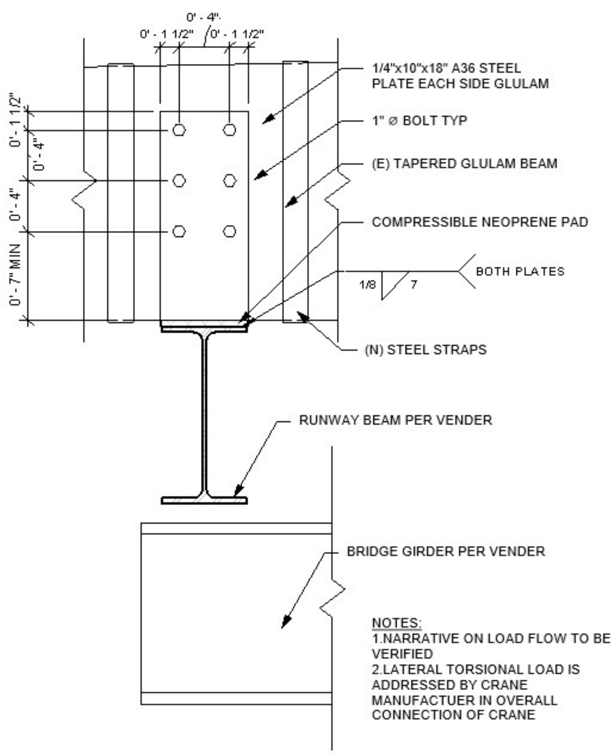


Figure 16: Final Connection Design

Again, the connection was designed to show the ability of the glulam beams to support the crane. Connection redesign and sizing will likely be needed when the design is finalized.

Conclusion

This report found the Simpson Strong-Tie building can support a 2-ton overhead crane. The report specifically investigated hanging the crane from the glulam beams. The type of crane considered was a two-ton hoist with the system specified by the vender, Dearborn Overhead Cranes (more information can be found in the crane quote in Appendix D). More specifically, the existing structure can support a hanger load of 7.8 kips, occurring four and a half feet away from the exterior wall on the glulam beams. While not investigated, supporting the crane on a separate column (stanchion) is also a viable option; however existing foundation coordination will have to be considered.

The capacity of the existing structure is governed by the axial capacity of the parallel strand laminated columns. The columns were originally designed to support a green roof. The green roof was never installed, creating enough extra capacity to allow the installation of the crane. After the installation of the crane the columns will be at capacity and will not have the ability to support additional loads, from either the load racks or any future additions including a green roof.

The connection design included in the report is suggested. It is mainly included to show the ability of the beam to support the crane. Connection sizing and redesign will likely be needed for the chosen crane.

The steps outlined in the project management section were sufficient for this small project. The project was completed on time. I gained experience working in an interdisciplinary team, especially on a project on a short schedule.

Hopefully the calculation package will contribute to Cal Poly Facility's approval for overhead crane installation in the Simpson Strong-Tie building.

Acknowledgements

I would like to thank Paul Redden for his support and expertise on the project. I would also like to thank my advisor, Craig Baltimore, for his guidance and constructive critique.

Appendix A – Example of Weekly Status Report

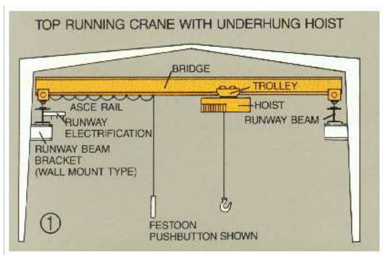
Status Report

4.18.2017

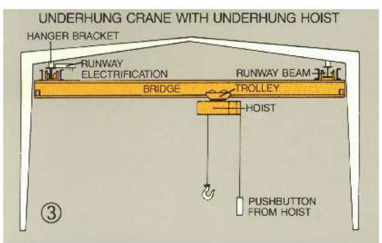
Project: Overhead Crane Installation in Simpson Strong-Tie Material Demonstration Laboratory
Engineer: Amy Poehlitz
Advisor: Craig Baltimore
Owner: Construction Management Department
Owner's Representation: Paul Redden

Summary- 4/18 Meeting with Craig Baltimore

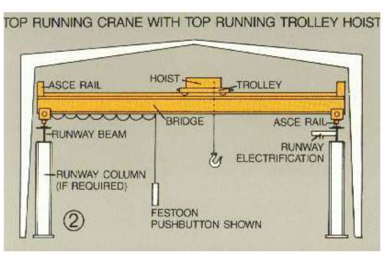
Reviewed crane options



Column Connection: not preferred. Places eccentric loading on columns.



Connection to Glulam Beams: preferred. I am using this option for the initial design



Separate Frame: needs more investigation. Need to see if it would interfere with existing foundations.

Discussed what governing capacity might be

-crushing in connection or beam shear

Discussed potential need for kicker depending on connection detail from crane to glulam beam, AISC Lattice

Discussed loading condition for personal safety restraints, depends on rating of support

Next Meeting: 10 am 4/25; Bldg. 21 Room 108B

Tasks for Week

Paul

- More information on the desired crane. Bottleneck Item - Due 4/21

  Provide specifications and cut sheets

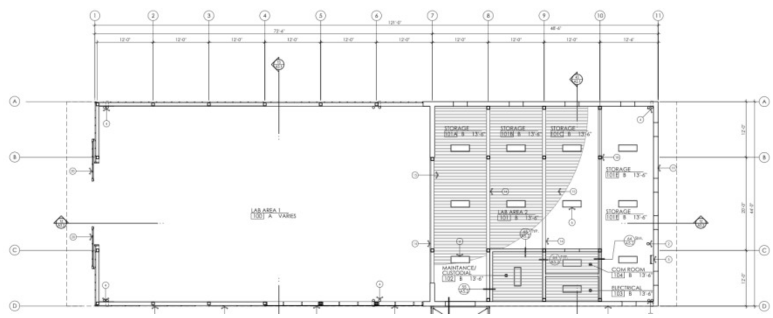
  Weight of the assembly including weight of crane and weight of runways/beams

  Deflection limit for bridge girder (allows me to estimate size of beams while waiting for cut sheets, determined by limit for operation of crane)

  Speed of the crane/dynamic loading from manufacturer

  Connection detail between crane and glulam beam

  Which bays does the crane needs to access (need to access mezzanine?). To you want to access the full width of the building (project north to south)?



Amy

-Write report section on management techniques – Due 4/25

-Check crushing on existing beam connection – Due 4/25

-Placement rail so it does not interfere with fire sprinklers – Due 4/25

-Write report section on influence lines – Due 4/25

-Influence lines for vertical reactions and deflection of bridge girder – Due 4/25

Tasks from Previous Week

Amy

-Outline Senior Project Report – (complete)

-Project Schedule – (complete)

-Moment and Shear Diagram for Tapered Glu-Lams – (complete)

Appendix B – Project Schedule

Schedule

4.11.2017

Project: Overhead Crane Installation in Simpson Strong-Tie Material Demonstration Laboratory
Engineer: Amy Poehlitz
Advisor: Craig Baltimore
Owner: Construction Management Department
Owner's Representation: Paul Redden

4/11 – Weekly Meeting

Senior Project Report Introduction

Calculation Package Building Description

4/13

Outline Senior Project Report

Memo – Verification of As-Builts

4/14

Receive Specifications and Cut Sheets for Crane

4/18 – Weekly Meeting

Shear/Moment Diagrams for Variable Cross-Section Glulams

Questions on Influence Lines

4/25 – Weekly Meeting

Influence Lines for Glulam Internal Forces

Rough Draft Report: Management

5/2 – Weekly Meeting

Influence Lines for Glulam Reactions

Rough Draft Report: Verifying As-Builts

5/9 – Weekly Meeting

Evaluation of Existing Columns

Rough Draft Report: Analysis

5/16 – Weekly Meeting

Design of Connection between Crane and Existing Structure

Rough Draft Report: Design

5/23 – Weekly Meeting

Compile Calculation Package

Rough Draft Report: Conclusion

5/30 – Weekly Meeting

Final Draft of Senior Project Report for Review

Address Comments on Calculation Package

6/2 – Weekly Meeting

Address Comments on Senior Project Report

6/6 – Weekly Meeting

Project Wrap-up

6/8 - Senior Project Presentation

Appendix C – Explanation of Methods to Develop Influence Lines

Müller-Breslau Principle

The Müller-Breslau Principle states the influence line for an action is proportional to the displacement of a structure where the action is replaced with a unit displacement or rotation. Reaction forces and internal shears are replaced with unit displacements. Moment reactions and internal moments are replaced with unit rotations. For example, the internal moment at the center of the beam in figure 16 was released and replaced by a unit rotation. The resulting displacement of the structure, shown in red, is proportional to the influence line for the internal moment. The line is then scaled by solving a single loading case.

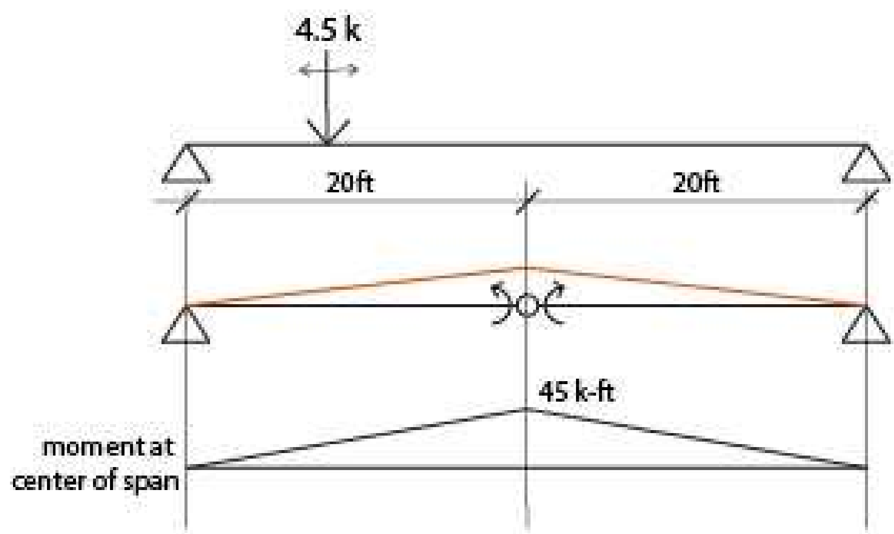


Figure 16: Using Müller-Breslau Principle

For more information:

structurefree. “Influence Lines with Muller-Breslau Principle (part 1) - Structural Analysis.” Online Video Clip. YouTube. YouTube, **23 October 2012. Web. May 15 2017.**

Maxwell's Reciprocal Displacement

This principle states that the displacement at point A resulting from a load at point B is the same as the displacement at point B from that same load at point A. For example, if F_B and F_A from figure 17 were equal than Δ_{AB} and Δ_{BA} would also be equivalent. To this principle to influence lines apply a load to the center of the beam and solve for the displacements at every point on the beam. By Maxwell's Reciprocal Displacement the displacement at any location on the beam from the load at the middle is also the displacement at the middle from the load at that location.

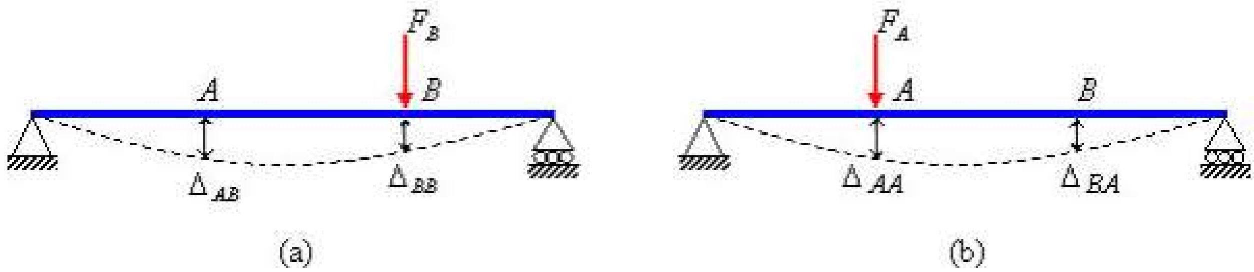


Figure 17: Maxwell's Reciprocal Displacement

For more information:

Parvanova, Sonya. "Influence lines for displacements." *Faculty - Sonya Lubomirova Purvanova-Joncheva*. University of Architecture, Civil Engineering and Geodesy, 2011. Web. 16 May 2017.

Appendix D – Crane Quote

*** Cover Sheet ***

To: Paul Redden
Company: California Polytechnic State University

From: Dan Yeakey

Quote#: 178021

Date: 05/05/17
Pages (including this page): 6
Comments: Bridge Crane

Dearborn Overhead Crane
Since 1947

DESIGN SPECIFICATIONS

Material design and safety features, based on service classification, will conform to the following listed specifications and codes where applicable and appropriate:

Crane Manufacturers Association of America, Inc., CMAA 70-74, Latest Edition
American National Standard, B30.11-1980 “Underhung Cranes”
American National Standard, ANSI-B30.16 “Overhead Hoist”
Hoist Manufacturers Institute – Specification HMI 100-74
Manual of Steel Construction, Latest Edition
National Electrical Code – Latest Edition
Specification for Welding Industrial & Mill Cranes ANSI/AWS D 14.1 – 1997

05/05/17 Proposal No. 178021

ATTN: Paul Redden
California Polytechnic State University

SUBJECT: Bridge Crane

SCOPE

One 2 ton under running, single girder crane per attached specifications. Our quote includes all crane and hoist components, runways, 4 conductor runway electrification, engineering, fabrication, painting.

PRICING

Item Description	Material	Installation	
2 ton under running, single girder crane	18300		
72’ Runways and End stops	5800		
Runway Electrification System	900		
Totals:	**$25,000.00**	By others	

- Remote Radio System Adder: $1,000, installed per crane
- Installation not included.
- We can also provide (1) man to supervise your crew with the install and start-up process @ $800/day plus travel and expenses
- Please add sales tax where applicable
- Freight not included

We hope the above information and enclosures are sufficient to answer any questions you might have regarding this equipment. Should there be any further questions or a need to get a better price quoted, please feel free to call. We would welcome the opportunity to further discuss our proposal with you.

Price quoted is subject to increase to cover any applicable sales or use tax which we are required by law to pay or collect as a result of this transaction. We reserve the right to correct any stenographic errors.
Terms: 35% down payment, 55% prior to delivery
10% net 30 (pending credit approval.)

Due to the volatile steel market, all quotations must be confirmed prior to acceptance of order.
Delivery: 5-6 weeks after receipt of signed approval drawings
and Purchase Order

Ordered by:	
Total Amount:	
PO Number:	

Attached Bid Notes are an integral part of this proposal.
Quoted By: DJY

Respectfully submitted,

Dan Yeakey

Dan Yeakey, DEARBORN CRANE & ENGINEERING CO.

Crane Specifications

Quote No:	178021	**Date:**	05/05/17
Customer:	California Polytechnic State University		
Attention:	Paul Redden		

Crane Data

Crane type	under running, single girder		**# of Cranes**	1
Capacity	2 Tons		**Operation**	Indoors
Span	40'- 0"		**Power**	460/3/60
Lift	22'- 0"		**Reeving**	2/1
Hoist	2 ton wire rope		**Control Encl.**	NEMA 12
Wheel Load	3200 lbs.		**Crane Rating**	CMAA C
End Trucks	5'- 10" wheel base		**Operation type**	Sliding P.B.
Configuration	single girder		**Cross Conductors**	Festooned
Girder Type	standard wide-flange		**Paint**	Yellow
Standard items	2 speed hoist , Weight Watcher Load Cell, Trolley Brake, Trolley Bumpers, Hoist Geared Upper and Lower Limit, Bridge Travel Slow Down Limit Switch, Trolley VFD Control, Bridge VFD Control, Warning Horn, Hoist Inspection Test Record, Wire Rope Certification, Hook Forging Certificate			

Runway Data

Runway	72' supported every 12'		**Runway Conductors**	4 bar Conductor system
Hanger Load	Estimated at 7.8 Kip – to be verified by engineering		**Runway Collectors**	Tandum Spring Shoe type
Paint	Standard primer – gray			
If required, foundations are by others. Loading are provided at time of order.				

Electrical Data

Drive	Motor Type	Speed (FPM)	Brakes
Hoist	Crane Duty	20/3.3 two speed	DC Disc
Trolley	Crane Duty	65 VFD	DC Disc
Bridge	Crane Duty	100 VFD	DC Disc

Bid Notes

The following bid notes are included as an integral part of the quote. Variances and/or exceptions may affect the final price of this equipment.

- Dearborn installation is computed on a straight time basis. Installation requested to be done outside normal working hours may require overtime charges. Pricing also assumes all work is done in a continuous fashion with a single "Start up and Shut down". Additional charges may be incurred if Dearborn is required to pull off the job and restart later.
- Dearborn cannot accept responsibility for any existing support or building structures.
- Proper sizing, location, alignment to CMAA recommendations, and spacing of Runways, columns, support steel and/or ASCE rail is responsibility of others **where Dearborn does not supply these items.**
- Design and supply of material for any concrete work is *not* included unless otherwise called out in quotation.
- All material factory painted using manufacturers standard paint with field paint touch up, by Dearborn.
- Temporary electric power for lighting and welders by others. **Dearborn uses Gas welders as a standard**
- Removal of any obstruction in crane path, by others. **Exception only if agreed to in advance in writing with Dearborn**
- Disposal of leaded paint chips, asbestos, or any other hazardous material which may need to be removed during the crane installation will be the responsibility of the owner.
- Handling and storage of materials due to delays by customer may result in additional charges.
- A complete Operating & Maintenance manual with appropriate spare parts list is included.
- All installation prices assume a clear work area with concrete floors in place. Delays and/or extra time incurred by the field crews due to inaccessibility to work area will be billed at then current rates for men and equipment .**Exception only if agreed to in advance in writing with Dearborn** .
- Adequate headroom above the bridge crane must be available to allow the boom of the mobile crane to set the crane and hoist. **Exception only if agreed to in advance in writing with Dearborn** .
- A fused disconnect and power from source to the disconnect and from disconnect to conductor bar is by others. Current taps and connection at the conductor bar is provided by Dearborn.
- Any permits required, by others.
- Basic operator orientation, including one session of approximately ½ hour at time of installation, is included. A more comprehensive operator and OSHA training is available on request.
- "Testing" in the scope section refers to ‘No Load’ Testing which tests all functions of equipment. Full Load Testing with test weights is available as described on the Load Testing Policy page.
- Limitation of liability: seller shall have no liability to buyer with respect to the sale of products or provision of services hereunder for lost profits or for special, consequential, exemplary or incidental damages of any kind.
- Warranty: Dearborn Overhead Crane provides a full 10 year warranty on all structural components of the crane (and runway system where supplied by Dearborn Overhead Crane). All mechanical and electrical components (such as hoists, end trucks, electrification, attachments, etc.) are supplied with a limited ONE year warranty on material only. Warranty period begins on the date of crane commissioning and signed acceptance by the customer.

OPTIONAL EQUIPMENT AVAILABLE:

- **RADIO CONTROL**
- **ANTI-COLLISION FEATURE**
- **WARNING DEVICES**
- **CRANE LIGHTS**
- **ADJUSTABLE FREQUENCY DRIVES**
- **OUTDOOR APPLICATIONS**
- **WALKWAYS AND/OR SERVICE PLATFORMS**

  (Means to access the above option is not included, but available)
- **SPECIAL PAINT**
- **BELOW HOOK DEVICES (C-HOOKS, MAGNETS, SLINGS, ETC)**

DEARBORN will provide the right combination of PERFORMANCE, RELIABILITY and VALUE, all at the right price. We not only build and install the highest quality cranes; we also have a full time field service crew that repairs all brands of cranes. DEARBORN also provides annual service contracts for those customers who depend on their equipment for critical production needs or who want to be sure their equipment is always in top working order and OSHA inspections are being done in a timely fashion.

Load Testing Advisory

1) Current specifications regarding the load tests of Overhead Crane Systems

ANSI B30.11 requires the following;
11-2.2.2 Rated Load Test
(a) Prior to initial use, all new, extensively repaired, and altered equipment shall be tested and inspected by, or under the direction of, an appointed or authorized person, and a written report should be furnished by such person, confirming the load rating of the system. The load rating should no be more than 80% of the maximum load sustained during the test.

OSHA 1910.179 Paragraph K2 states the following;
Rated load test.
Test loads shall not be more than 125% of the rated load unless otherwise recommended by the manufacturer. The test reports ***shall*** be placed on file where readily available to appointed personnel.

2) Definitions

Hoist
A machinery unit that is used for lifting and lowering a load.
Crane
A bridging structure that spans two or more runways and provides traversing motion.
Runway
The rails, beams, brackets and framework on which the crane operates.

3) Dearborn Overhead Crane Load Testing Offer
(Maximum crane capacity of 10 ton or less)

It therefore is our opinion, that it is the owner's responsibility to load test the overhead bridge crane ***system***. The "system" consists of the hoist, crane, runways, columns and footings. To test the "system", requires that the full system be in place and therefore must occur after the completion of the crane installation. Although it is hoist industry practice to load test every hoist prior to shipping, this practice does not preclude the requirement for the full load testing upon commissioning of the full hoist, crane and runway system.

As a service to our customers, DOC offers to provide this load test free of charge to the purchaser of a Dearborn Crane. To take advantage of this **free** offer, **ALL** the following criteria is required.

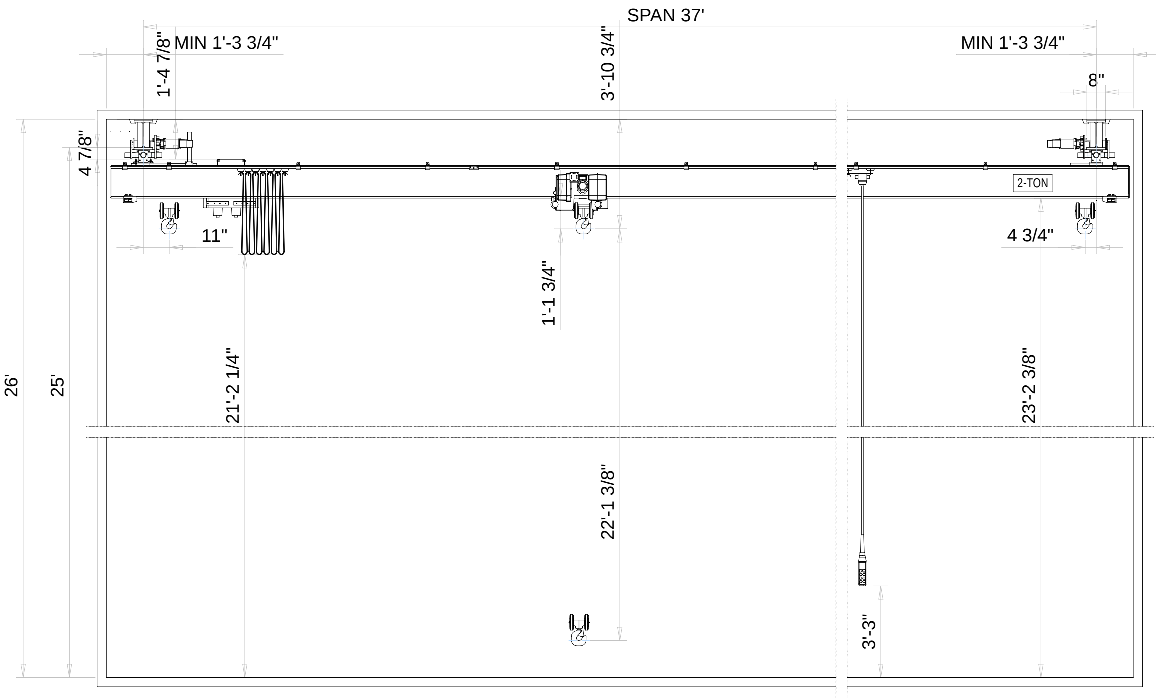
1. Dearborn Must be contracted to install the crane and will perform this load test during the normal installation time period
2. The customer must provide a full capacity test load** to be used for testing purposes. This load must be 125% of the rated capacity of the crane.
3. The load must be of reliable weight, and be easily accessible and in the immediate area of the crane to be tested. The weight of the load should be certifiable. If certification of the load is not practical, DOC can provide a scale to weigh the load for a small fee.
4. All necessary rigging** must be provided by the customer.
5. The load and rigging must be provided while DOC is on site for the installation. (If the customer cannot provide the required materials during the normal installation period, but still would like to have a Certified Load Test performed, a crew can be provided on a time and material basis.)

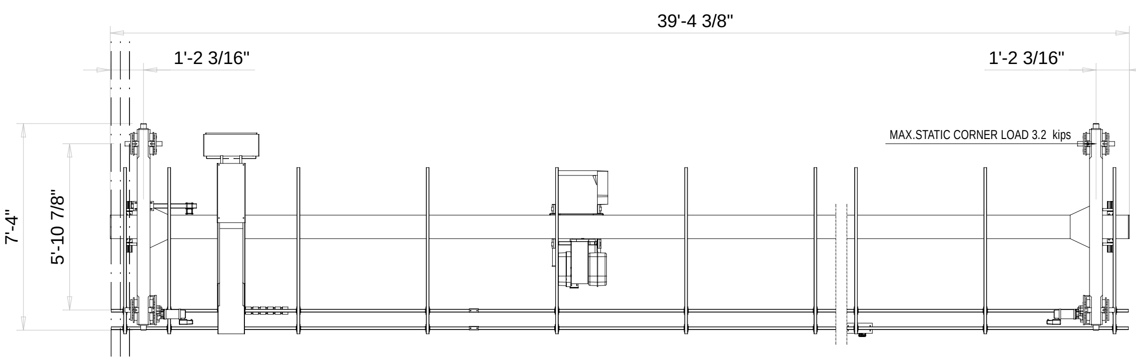
Upon completion of the test, a temporary hand written certificate will be supplied to the owner, so that there is no possible exposure to the owner of not have proper documentation. The official Load Test Certificate to following within 2 weeks. Also, a copy will remain on file at Dearborn.

** Dearborn Overhead Crane is not responsible for any damage that may occur to either the test weights or the rigging used.

Poehlitz 33

DIMENSIONS ARE BASED ON A CLEAR INSIDE BUILDING
HEIGHT OF 26'-0"





SPAN	: 37'-0"
LOAD	: 2 TON
LIFTING HEIGHT	: 22'-1 5/16"
HOISTING SPEED	: 20.0/3.2 ft/min 2-speed
TRAVERSING SPEED	: 65 ft/min stepless
TRAVELLING SPEED	: 100 ft/min stepless
WEIGHT OF TROLLEY	: 520 lbs
WEIGHT OF BRIDGE	: 3617 lbs
CRANE CLASSIFICATION	: CMAA Class C
HOIST GROUP	: ASME H4
CRANE DRIVE GROUP	: CMAA C
MAIN/CONTROL VOLTAGE	: 460 / 115 V; 60 Hz
SUPPLY POWER	: 4.33 Hp
LENGTH OF RUNWAY	: 0

Indoor Use Crane Bridge Color: Safety Yellow
Customer Organizes Safe Platform For Crane Service

Dearborn Overhead Crane
Lifting Expectations Since 1947

Please Indicate North on Compass Below

PRELIMINARY DRAWING SUBJECT TO ENGINEERING REVIEW. ACTUAL DIMENSIONS AND TECHNICAL DATA MAY VARY AND ARE SUBJECT TO CHANGE.

California Polytechnic State Univers

178021

Appendix E- Calculation Package

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Gravity Framing
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- Size Crane Members 38
- Analysis Existing Glulam Beam 45
- Analysis Existing Connection 53

Connection Design
- Runway to Glulam Beam 55

Building Description

Project Description
Location: Campus of California Polytechnic State University, San Luis Obispo, California
General Description: Large laboratory building used for Construction Management classes. Also hosts the College of Architecture and Environmental design DFAB lab.

Codes:
Existing Building: 2007 CBC, ASCE 7-05, ACI 318-05, ASIC 360-2005, NDS 200
New Design: 2016 CBC, ASCE 7-10, ACI 318-11, ASIC 360-2011, NDS 2015

Roof Loading
Dead: 20psf
Live: 20psf

Mezzanine
Dead Load: 55psf
Live Load: 125psf (Light Storage)

Wind Design Criteria
Basic Wind Speed: 85mph
Wind Importance Factor: 1.15
Wind Exposure: B
Applicable Internal Pressure Coefficient: 0.55

Seismic Design Criteria
Seismic Importance Factor: 1.25
Occupancy Category: III
Mapped Spectral Response Accelerations: S_S=1.263 S_1=0.474
Site Class: B
Spectral Response Coefficients: S_{DS}=0.85 S_{D1}=0.32
Seismic Design Category: D

Geotechnical Information
Site Class: B
Allowable Bearing Pressure = 2500psf

Gravity Load Flow – Roof
Load is distributed by the sheathing and 3x decking onto 6x10 Purlins spaced at four feet. These purlins are supported by 6 3/4”x20” taperd glulam beams spaced at 12 feet, which in turn rest on 7”x7” Parallam PSL columns. These columns are supported by a concrete wall which sits atop a continuous spread footing.

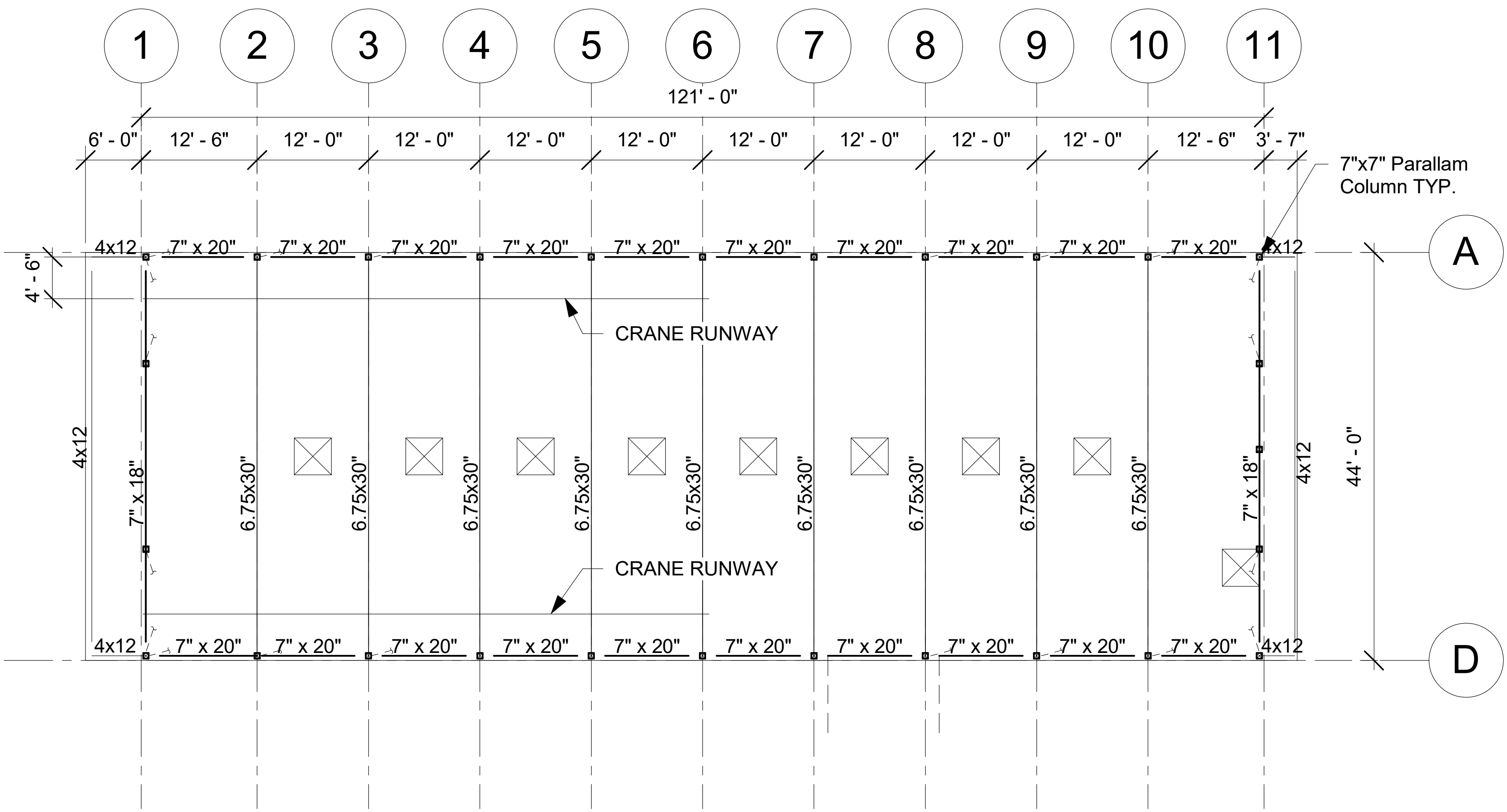
Lateral Load Flow - Roof
The lateral loads on the roof are transferred through the sheathing and 3x decking into the rim beams. The rim beams collects the load into the heavy timber braced frames on the exterior of the building. The braces transfer the load into the concrete shear walls which connect to the foundation. The foundation passes the load into the ground.

Gravity Load Take Off

Dead Load		
80 mil PVC Roofing	0.5	psf
Rigid Insulation	1.5	psf
5/8" Plywood Roof Sheathing	1.8	psf
3x6 Douglas Fir Decking	7.6	psf
Sprinklers	5.2	psf
MEP	2	psf
Misc.	1.4	psf
Total	20	psf

Live Load		
Maintenance	20	psf

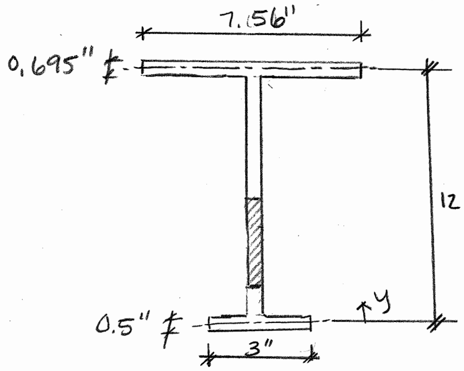
ROOF KEY PLAN



USE EXISTING CRANE IN HIGHBAY LABRATORY TO ESTIMATE DEFLECTION CRITERIA FOR BRIGE GIRDER

EXISTING BRIDGE BEAM

W18x60 castallated into new cross section



CASTALLATED - approximate moment of inertia just using two flanges

$$\bar{y} = \frac{Ay}{A} = \frac{0.5''(3'')(0'') + 0.695''(7.56'')(12'')}{0.5''(3'') + 0.695''(7.56'')}$$

$$\bar{y} = 9.55''$$

FIND MOMENT OF INERTIA

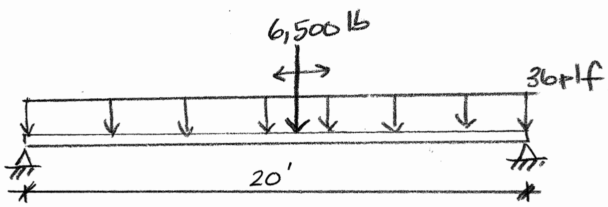
$$I_{xx} = \Sigma(I_{xxi} + Ad_i^2) = \frac{3''(0.5'')^3}{12} + (3'')(0.5'')(9.55'')^2 + \frac{7.56''(0.695'')^3}{12} + (0.695'')(7.56'')(12''-9.55'')^2$$

$$I_{xx} = 168.6 \text{ in}^4$$

LOADING BRIDGE BEAM

SELF WEIGHT BEAM $\approx$ 3/5(60plf) = 36plf

HOIST + CRANE HOUSING 3TONS + 500 lb = 6,500 lb



DEFLECTION FROM DISTRIBUTED LOAD

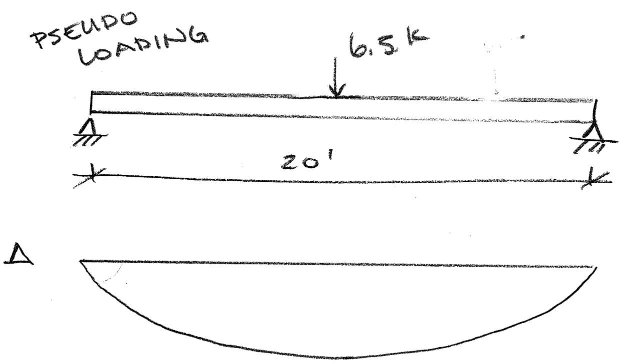
AISC BEAM EQUATIONS

$$\Delta_{center} = \frac{5wl^4}{384EI} = \frac{5(36plf)(20')^4(12''/1')^3}{384(29{,}000{,}000psi)(168.4in^4)} = 0.0265''$$

INFLUENCE LINE DEFLECTION AT CENTER OF BRIDGE BEAM

MAXWELL'S PRINCIPAL OF RECIPROCAL DEFLECTION

SHAPE OF THE INFL. LINE FOR DEFLECTION AT CENTER FROM MOVING LOAD IS SAME AS DEFLECTION CAUSED BY LOAD AT CENTER



MAXIMUM DEFLECTION OCCURS WHEN LOAD IS AT CENTER OF BEAM

AISC BEAM EQUATION

$$\Delta_{max} = \frac{Pl^3}{48EI} = \frac{6.5^k (240'')^3}{48(29{,}000\,ksi)(168.6\,in^4)} = 0.3829''$$

SOLVE FOR DEFLECTION CRITERIA

$\Delta_{total} = \Delta_{DIST\ LOAD} + \Delta_{MOVING\ LOAD}$

$\Delta_{total} = 0.0265'' + 0.3829'' = 0.4094$

$\Delta_{ALL.} = \frac{SPAN}{X}$ $\quad X = \frac{SPAN}{\Delta} = \frac{240''}{0.4094''} = 586$

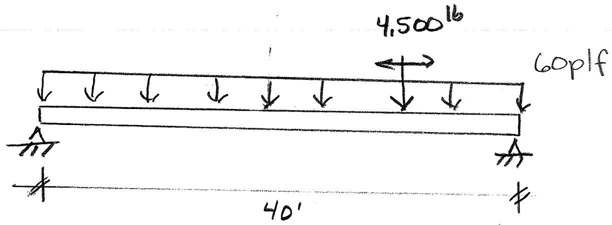
$$\boxed{\text{USE } \Delta_{ALL} \leq \frac{l}{586}}$$

TO ESTIMATE BRIDGE BEAM SIZE FOR SIMPSON STRONG-TIE

ESTIMATE SIZE BRIDGE BEAM

LOADING

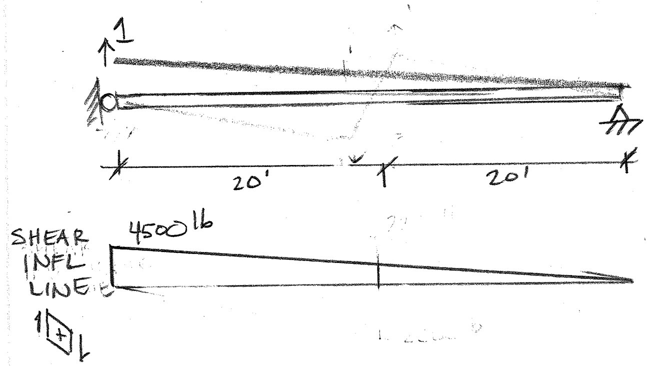
SELF WEIGHT BEAM GUESS 60 plf
HOIST + CRANE HOUSING 2 TONS + 500 lb = 4,500 lb



$436 \text{ in}^4 = I_{MIN}$

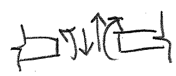
INFLUENCE LINES FROM MOVING POINT LOAD

SHEAR AT END OF BEAM



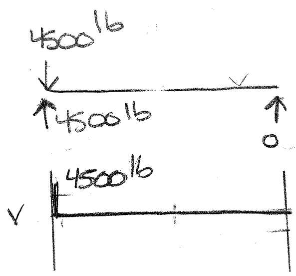
*UNFACTORED

POSTIVE SIGN CONVENTION

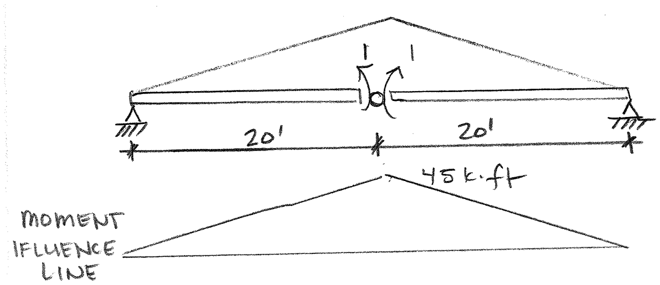


DEFLECTION GIVES QUANITATIVE SHAPE INFLUENCE LINE

SOLVE CASE TO SCALE INFL. LINE

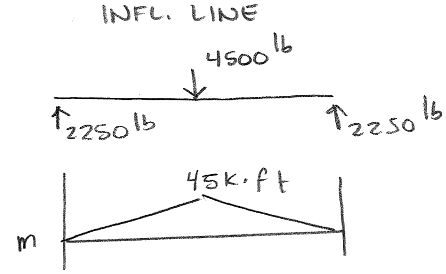


MOMENT AT CENTER OF BEAM



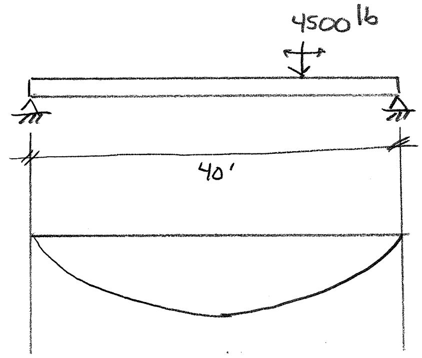
*UNFACTORED

SOLVE CASE TO SCALE INFL. LINE



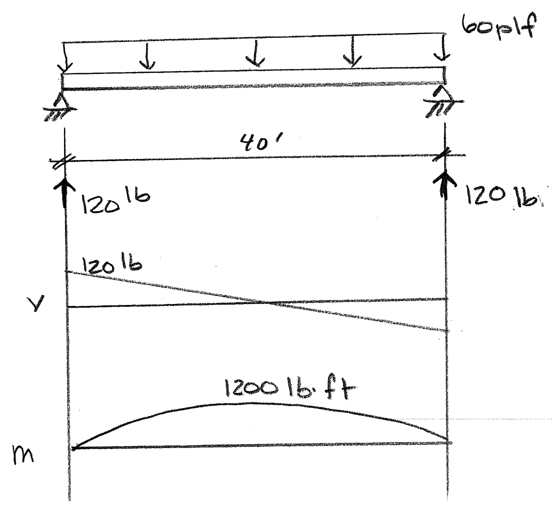
INFLUENCE LINES CONT

DEFLECTION AT CENTER OF BEAM



$$\Delta_{MAX} = \frac{Pl^3}{48EI}$$

DEMAND FROM DISTRIBUTED LOAD



* UNFACTORED

$$\Delta_{MAX} = \frac{5wl^4}{384EI}$$

FIND I_{MIN}

$$\Delta_{ALL} = \frac{l}{586}$$ [FROM EVALUATION OF HIGH BAY]

$$\Delta_{ALL} = \frac{480''}{586} = 0.82''$$ [ALLOWABLE DEFLECTION FOR SIMPSON STRONG-TIE]

$$\Delta_{ALL} \geq \Delta = \frac{Pl^3}{48EI} + \frac{5wl^4}{384EI}$$ SOLVE FOR MOMENT OF INERTIA

$$I_x \geq \frac{Pl^3}{48E\Delta_{ALL}} + \frac{5wl^4}{384E\Delta_{ALL}} = \frac{4.5^k(480'')^3}{48(29{,}000ksi)(0.82'')} + \frac{5(60plf)(40')^4(12\tfrac{in}{ft})^3}{384(29{,}000{,}000psi)(0.82'')}$$

$$I_x \geq 582\,in^4$$

USE W18x40 $I_x = 612in^4$

CHECK W18x40

BENDING CAPACITY

UNBRACED LENGTH $l_u = 40'$

$L_u > L_r = 13.1$ ELASTIC LATERAL TORSIONAL BUCKLING

$$r_{ts}^2 = \frac{I_y h_o}{2S_x} = \frac{19.1in^4 (17.4")}{2(68.4in^3)} = 2.43in^2$$

$r_{ts} = 1.56"$

$J = 0.810 in^4$

$c = 1.0$ for doubly symmetric I-shapes

$$F_{CR} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2} \sqrt{1 + 0.078 \frac{J c}{S_x h_o} \left(\frac{L_b}{r_{ts}}\right)^2}$$

$$= \frac{1.14(\pi^2)(29000 ksi)}{(480"/1.56")^2} \sqrt{1 + 0.078 \frac{(0.81in^4)(1.0)}{68.4in^3(17.4")} \left(\frac{480"}{1.56"}\right)^2} = 8.46 ksi$$

$\phi M_n = 0.9(8.46ksi)(68.4in^3) = 521$ k·in $= 43$ k·ft

$M_u = 1.4D = 1.4(45$ k·ft $+ 1.2$ k·ft$) = 65$ k·ft

$M_u \not< \phi M_n$ FAILS BENDING

TRY 21x44 $I_x = 586in^4 > I_{MIN} = 582 in^4$

$L_u = 40' > L_r = 16.5'$ ELASTIC LATERAL TORSIONAL BUCKLING

$r_{ts} = 1.87"$ $J = 1.11 in^4$ $c = 1.0$ $S_x = 72.7in^3$

$$F_{CR} = \frac{1.14(\pi^2)(29000 ksi)}{(480"/1.87")^2} \sqrt{1 + 0.078 \frac{1.11in^4 (1.0)}{72.7in^3(15.5")} \left(\frac{480"}{1.87"}\right)^2} = 12.2 ksi$$

$\phi M_n = 0.9(12.2 ksi)(72.7in^3) = 797$ k·in $= 66.4$ k·ft

$\phi M_n > M_u$ ✓ OKAY IN BENDING

USE 21x44

CHECK SHEAR

$\phi V_n = 167^k$

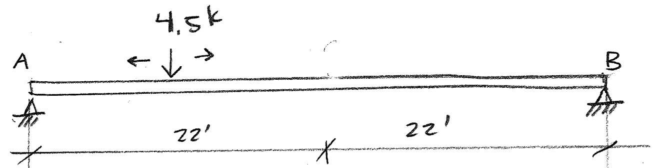
$V_u = 1.4D = 1.4(2.25^k + .12^k) = 3.3^k$

$V_u < \phi V_n$ ✓ OKAY IN SHEAR

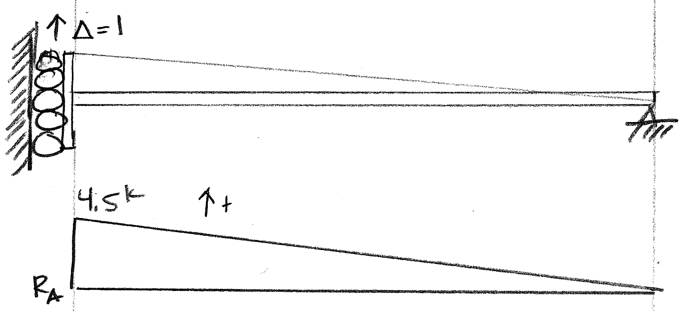
$\boxed{\text{USE } 21\times44}$ INITIAL ESTIMATE SELF WEIGHT = 60plf CONS.

INFLUENCE LINES - BRIDGE GIRDER REACTIONS

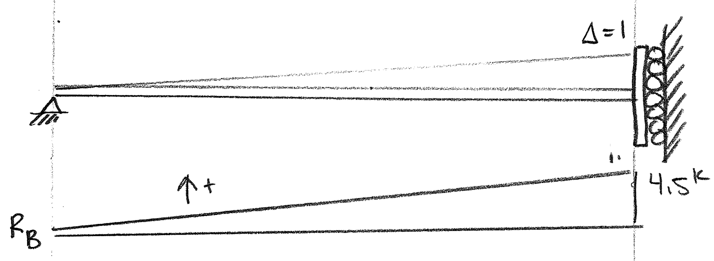
ONLY REACTIONS FROM MOVING LOAD
ADD TO REACTIONS FROM DISTRIBUTED LOAD



VERTICAL REACTION AT A

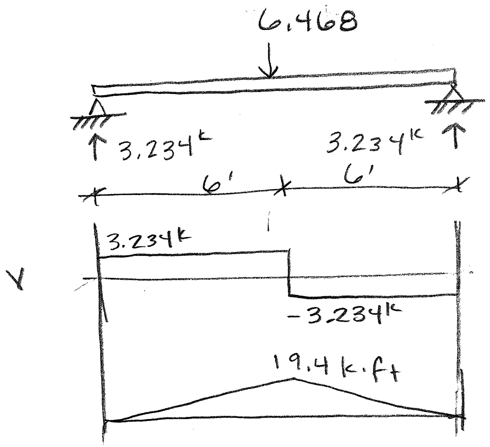


VERTICAL REACTION AT B



SIZE RAILS

BEAM CONTINUOUS BETWEEN SUPPORTS BUT APPROXIMATING AS PIN-PIN FOR QUICK ESTIMATE



* FACTORED

$P = (4.5^k + 1.2^k)1.4) = 6.468^k$

WORST LOADING CASE FOR BENDING IS IN CENTER OF BAY

USE W10 x 22

$L_b = 12''$ L_p

$\phi M_n = 68$ k·ft $> M_u$ ✓

LOAD ONTO GLULAM BEAM

$12'(22\,plf) = 264^{lb}$

CRANE + WEIGHT BRIDGE GIRDER + WEIGHT RAILS

$4.5^k + 0.12^k + 0.264^k = 4.88^k$ [maximum occur AT EITHER HANGER, NOT SIMULTANEOUSLY]

VENDER GAVE LOAD ONTO GLULAM AS $\boxed{7.8^k}$

POSSIBLY USED HEAVIER MEMBERS TO RESIST TORSION

POSSIBLY HAD MORE STRINGANT DEFLECTION CRITERIA

DEMAND GLULAM BEAMS

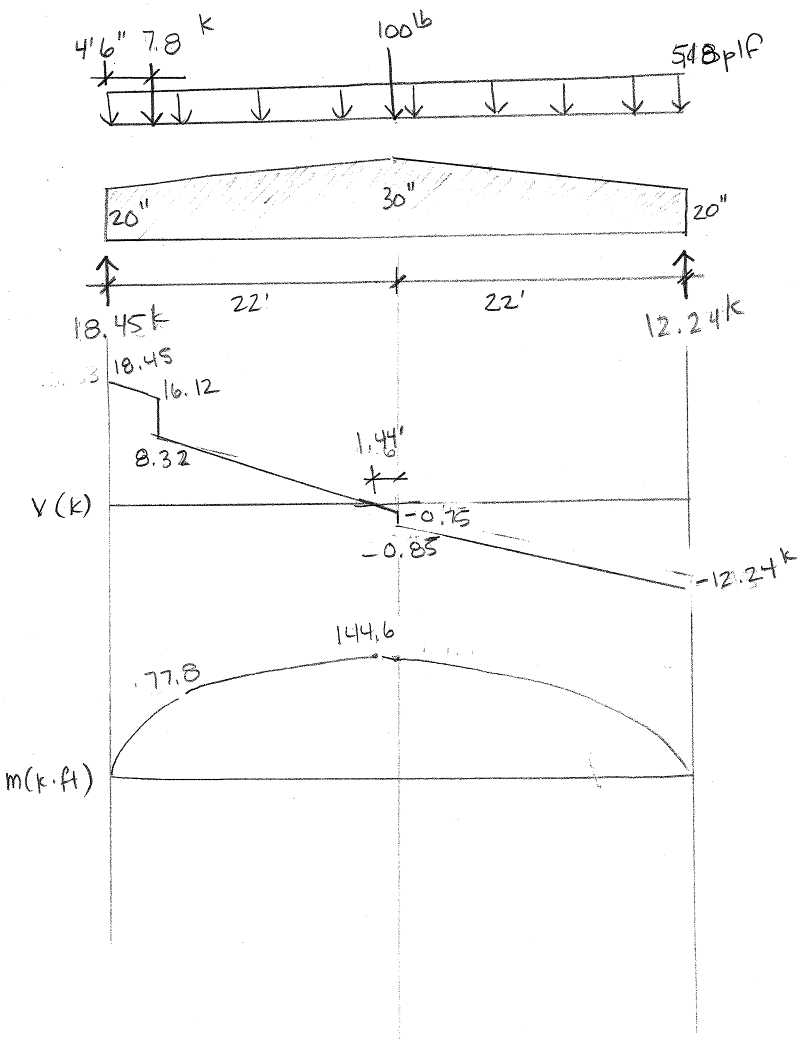
REACTION FROM CRANE HANGERS: 7.8k [PER CRANE VENDER]

ASD D+L TRIBUTARY WIDTH 12'

1. DEAD
   ROOF + SELF WEIGHT = 20psf(12') + 38plf = 278plf
   POINT LOAD 100lb SKYLIGHT

   LIVE
   20psf(12') = 240plf

   DIST. LOAD: D+L = 278plf + 240plf = 518plf



DEMAND GLULAM BEAMS CASE 2

REACTION FROM CRANE HANGERS = 7.8^k [PER CRANE VENDER]

ASD D+L 12' TRIB WIDTH

DEAD

ROOF + SELF WEIGHT = 20psf (12') + 38plf = 278 plf

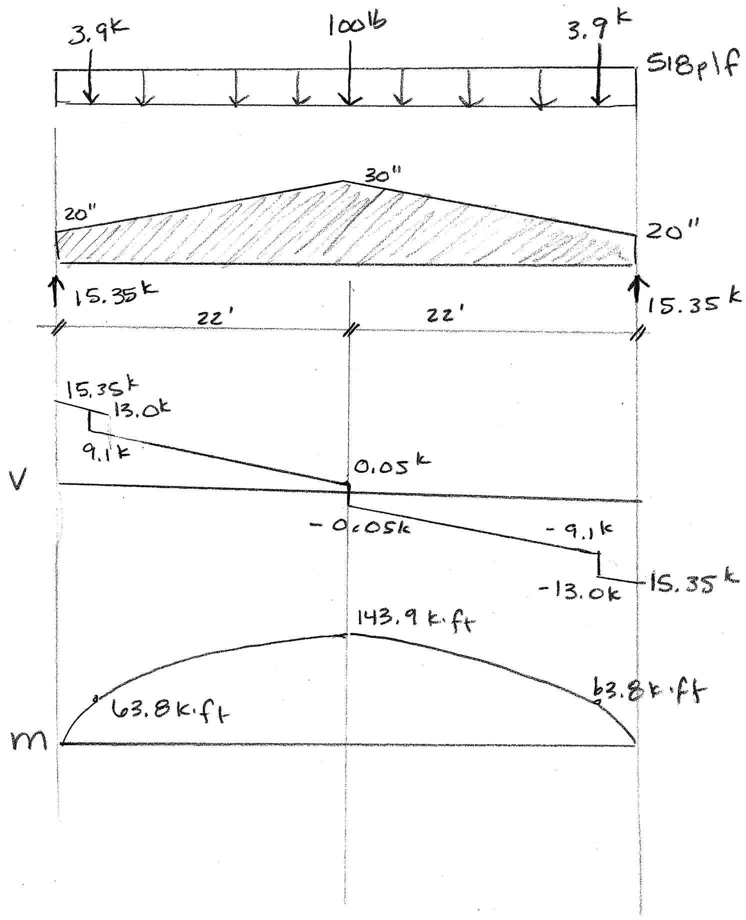
POINT LOAD SKYLIGHT 100lb

LIVE

20psf (12') = 240plf

DIST LOAD

D+L = 278plf + 240plf = 518plf



CHECK CAPACITY EXISTING GLULAM

HSC GLB 6 3/4" x 20" TAPERED TO 6 3/4" x 30"

SHEAR

$F_v = 190\,psi$

$F_v' = F_v C_D C_m C_t C_{vr} = 190\,psi(1.0)(1.0)(1.0)(0.72) = 136.8\,psi$

$C_D = 1.0$ (D+L)
$C_m = 1.0$
$C_t = 1.0$
$C_{vr} = 0.72$ (NON-PRISMATIC MEMBER)

$$f_v = \frac{18{,}450\,lb}{6.75''(20'')} = 136.7\,psi$$

$f_v < F_v'$ ✓

BENDING

$F_b = 3200\,psi$

$F_b' = F_b C_D C_m C_t C_L C_v C_{fu} C_c C_I$

$C_D = 1.0$ (D+L) $C_m = 1.0$ $C_t = 1.0$ $C_{fu} = 1.0$ $C_c = 1.0$
$C_L = 1.0$ (COMPRESSION EDGE BRACED)

$$C_v = \left(\frac{21}{L}\right)^{1/x}\left(\frac{12}{d}\right)^{1/x}\left(\frac{5.125}{b}\right)^{1/x} = \left(\frac{21}{44}\right)^{1/10}\left(\frac{12}{30}\right)^{1/10}\left(\frac{5.125}{6.75}\right)^{1/10} = 0.824 < 1.0\ ✓$$

$x = 10$ (not southern pine)

$$C_I = \frac{1}{\sqrt{1+\left(\frac{F_b \tan\theta}{F_v C_{vr}}\right)^2 + \left(\frac{F_b \tan^2\theta}{F_{c\perp}}\right)^2}}$$

$C_{vr} = 0.72$ (NON-PRISMATIC)
$F_{c\perp} = 510\,psi$
$\theta = \tan^{-1}\left(\frac{10''}{22'(12''/1')}\right) = 2.2°$

$$C_I = \frac{1}{\sqrt{1+\left(\frac{3200\,psi \tan(2.20)}{190\,psi(0.72)}\right)^2 + \left(\frac{3200\,psi \tan^2(2.2°)}{510\,psi}\right)^2}} = 0.744$$

TAPERED ON COMPRESSION FACE
ONLY LESSER OF C_v AND C_I APPLIES PER NDS

$F_b' = 3200\,psi(1.0)(1.0)(1.0)(1.0)(1.0)(1.0)(1.0)(0.744) = 2380\,psi$

SAMPLE CALCULATIONS FOR GLULAM BEAM SPREADSHEET

Distance Along Beam: defined by user (ft)

$$\text{Height} = 20'' + \frac{10''(\text{Dist along Beam})}{22'} \quad (\text{in})$$

$$\text{Moment of Inertia} = \frac{6.75''(\text{Height})^3}{12} \quad (\text{in}^4)$$

Moment: input by user (k·ft)

$$\text{Bending stress} = \frac{(\text{moment})(12''/1')\left(\frac{\text{Height}}{2}\right)}{\text{Moment of Inertia}} \quad (\text{ksi})$$

EXAMPE: Load Case 7, 4.5'

Distance along Beam = 4.5'

$$\text{Height} = 20'' + \frac{10''(4.5')}{22'} = 22.05''$$

$$\text{Moment of Inertia} = \frac{6.75''(22.05'')^3}{12} = 6017.17\,in^4$$

Moment = 77.77 k·ft [FROM ETABS]

$$\text{Bending Stress} = \frac{77.77k\cdot ft\,(12''/1')\left(\frac{22.05''}{2}\right)}{6017.17\,in^4} = 1.71\,ksi$$

Glulam Beam Bending Stress

Load Case 1

Distance Along Beam (ft)	Height (in)	Moment of Intertia (in^4)	Moment (k-ft)	Bending Stress (ksi)
0	20.00	4493.33	0.00	0.00
0.9	20.41	4774.74	16.39	0.42
1.8	20.82	5067.66	32.37	0.80
2.7	21.23	5372.31	47.92	1.14
3.6	21.64	5688.94	63.06	1.44
4.5	22.05	6017.77	77.77	1.71
4.5	22.05	6017.77	77.77	1.71
5.4722	22.49	6386.96	85.61	1.81
6.4444	22.93	6770.95	92.97	1.89
7.4167	23.37	7170.07	99.83	1.95
8.3889	23.81	7584.53	106.20	2.00
9.3611	24.26	8014.66	112.08	2.04
10.3333	24.70	8460.75	117.48	2.06
11.3056	25.14	8923.14	122.38	2.07
12.2778	25.58	9402.04	126.79	2.07
13.25	26.02	9897.76	130.72	2.06
14.2222	26.46	10410.62	134.15	2.05
15.1944	26.91	10940.89	137.10	2.02
16.1667	27.35	11488.92	139.55	1.99
17.1389	27.79	12054.90	141.52	1.96
18.1111	28.23	12639.16	143.00	1.92
19.0833	28.67	13242.01	143.98	1.87
20.0556	29.12	13863.79	144.48	1.82
21.0278	29.56	14504.67	144.49	1.77
22	30.00	15165.00	144.01	1.71

Distance Along Beam (ft)	Height (in)	Moment of Intertia (in^4)	Moment (k-ft)	Bending Stress (ksi)
44	20.00	4493.33	0.00	0.00
43.1	20.41	4774.74	10.81	0.28
42.2	20.82	5067.66	21.20	0.52
41.3	21.23	5372.31	31.17	0.74
40.4	21.64	5688.94	40.72	0.93
39.5	22.05	6017.77	49.85	1.10
39.5	22.05	6017.77	49.85	1.10
38.5278	22.49	6386.96	59.24	1.25
37.5556	22.93	6770.95	68.15	1.38
36.5833	23.37	7170.07	76.56	1.50
35.6111	23.81	7584.53	84.48	1.59
34.6389	24.26	8014.66	91.92	1.67
33.6667	24.70	8460.75	98.86	1.73
32.6944	25.14	8923.14	105.32	1.78
31.7222	25.58	9402.04	111.28	1.82
30.75	26.02	9897.76	116.76	1.84
29.7778	26.46	10410.62	121.74	1.86
28.8056	26.91	10940.89	126.24	1.86
27.8333	27.35	11488.92	130.25	1.86
26.8611	27.79	12054.90	133.76	1.85
25.8889	28.23	12639.16	136.79	1.83
24.9167	28.67	13242.01	139.33	1.81
23.9444	29.12	13863.79	141.38	1.78
22.9722	29.56	14504.67	142.94	1.75
22	30.00	15165.00	144.01	1.71

Glulam Beam Bending Stress

Load Case 2

Distance Along Beam (ft)	Height (in)	Moment of Intertia (in^4)	Moment (k-ft)	Bending Stress (ksi)
0	20.00	4493.33	0.00	0.00
0.9	20.41	4774.74	13.60	0.35
1.8	20.82	5067.66	26.78	0.66
2.7	21.23	5372.31	39.55	0.94
3.6	21.64	5688.94	51.89	1.18
4.5	22.05	6017.77	63.81	1.40
4.5	22.05	6017.77	63.81	1.40
5.4722	22.49	6386.96	72.43	1.53
6.4444	22.93	6770.95	80.56	1.64
7.4167	23.37	7170.07	88.19	1.72
8.3889	23.81	7584.53	95.34	1.80
9.3611	24.26	8014.66	102.00	1.85
10.3333	24.70	8460.75	108.17	1.89
11.3056	25.14	8923.14	113.85	1.92
12.2778	25.58	9402.04	119.04	1.94
13.25	26.02	9897.76	123.74	1.95
14.2222	26.46	10410.62	127.95	1.95
15.1944	26.91	10940.89	131.67	1.94
16.1667	27.35	11488.92	134.90	1.93
17.1389	27.79	12054.90	137.64	1.90
18.1111	28.23	12639.16	139.89	1.87
19.0833	28.67	13242.01	141.66	1.84
20.0556	29.12	13863.79	142.93	1.80
21.0278	29.56	14504.67	143.71	1.76
22	30.00	15165.00	144.01	1.71

Distance Along Beam (ft)	Height (in)	Moment of Intertia (in^4)	Moment (k-ft)	Bending Stress (ksi)
44	20.00	4493.33	0.00	0.00
43.1	20.41	4774.74	13.60	0.35
42.2	20.82	5067.66	26.78	0.66
41.3	21.23	5372.31	39.55	0.94
40.4	21.64	5688.94	51.89	1.18
39.5	22.05	6017.77	63.81	1.40
39.5	22.05	6017.77	63.81	1.40
38.5278	22.49	6386.96	72.43	1.53
37.5556	22.93	6770.95	80.56	1.64
36.5833	23.37	7170.07	88.19	1.72
35.6111	23.81	7584.53	95.34	1.80
34.6389	24.26	8014.66	102.00	1.85
33.6667	24.70	8460.75	108.17	1.89
32.6944	25.14	8923.14	113.85	1.92
31.7222	25.58	9402.04	119.04	1.94
30.75	26.02	9897.76	123.74	1.95
29.7778	26.46	10410.62	127.95	1.95
28.8056	26.91	10940.89	131.67	1.94
27.8333	27.35	11488.92	134.90	1.93
26.8611	27.79	12054.90	137.64	1.90
25.8889	28.23	12639.16	139.89	1.87
24.9167	28.67	13242.01	141.66	1.84
23.9444	29.12	13863.79	142.93	1.80
22.9722	29.56	14504.67	143.71	1.76
22	30.00	15165.00	144.01	1.71

BENDING CONT.

$f_b = 2.07$ ksi [max from spreadsheet]

$F_b' = 2.38$ ksi

$f_b < F_b'$ ✓

DEFLECTION

IBC TABLE 1604.3

ROOF MEMBERS – NOT SUPPORTING CEILING

$$\Delta_{L\ ALLOWABLE} = \frac{l}{180} = \frac{44'(12^{in}/ft)}{180} = 2.93''$$

$$\Delta_{D+L\ ALLOWABLE} = \frac{l}{120} = \frac{44'(12^{in}/ft)}{120} = 4.4''$$

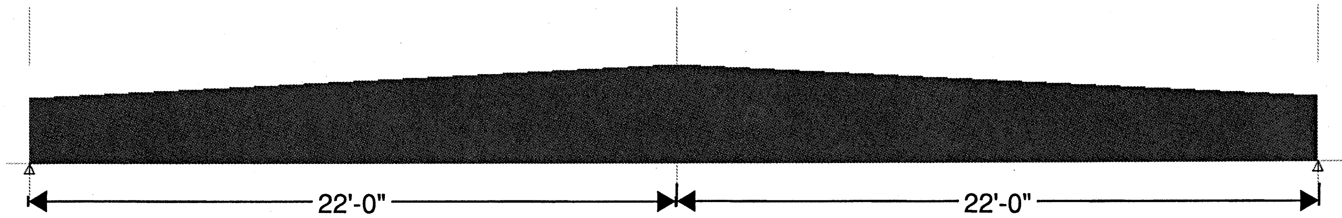
ACTUAL DEFLECTION FROM ETABS OUTPUT ON NEXT PAGE

$\Delta_L = 0.873'' < \Delta_{L\ ALL}$ ✓

$\Delta_{D+L} = 2.228'' < \Delta_{D+L\ ALL}$ ✓

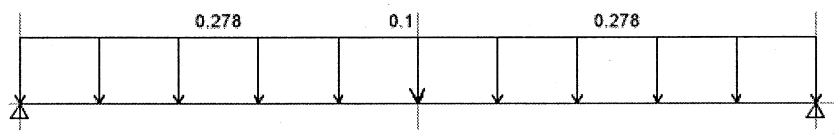
Spring 2017	SST CRANE	Amy Poehlitz

ETABS ANALYSIS - GLULAM BEAM

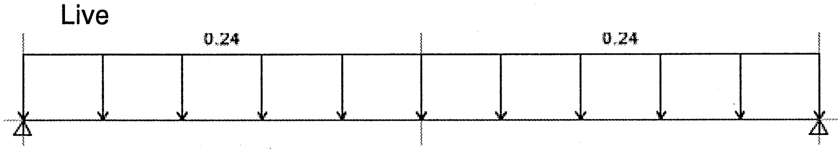


LOAD CASES (loads given in klf and kips)

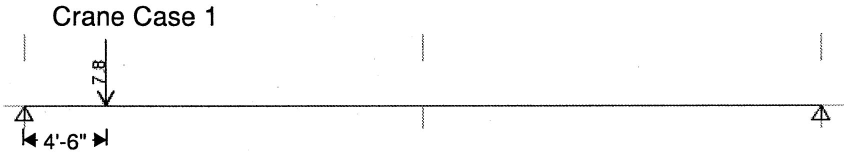
Dead



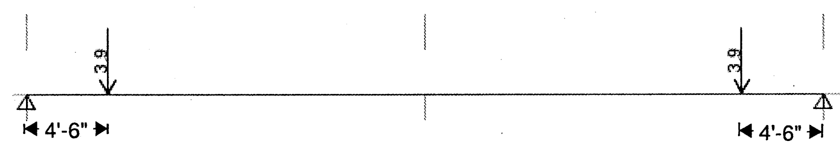
Live



Crane Case 1



Crane Case 2



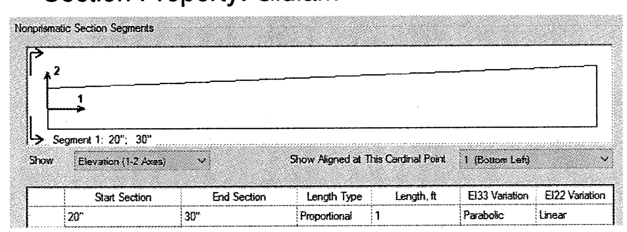
MEMBER PROPERTIES

Material: HSC Glulam
E=2,100 ksi

Section Property : 20"
Material: HSC Glulam
Section Type: Steel Plate
Dimensions: 6.75" x 20"

Section Property : 30"
Material: HSC Glulam
Section Type: Steel Plate
Dimensions: 6.75" x 30"

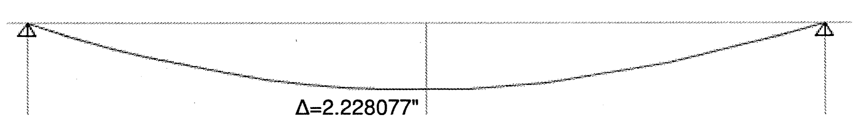
Section Property: Glulam



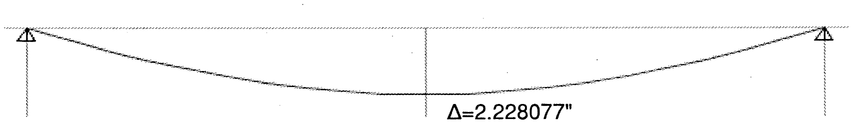
	Start Section	End Section	Length Type	Length, ft	EI33 Variation	EI22 Variation
	20"	30"	Proportional	1	Parabolic	Linear

DEFLECTIONS

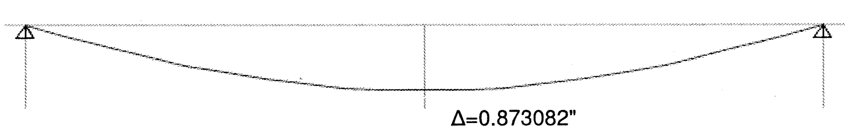
Case 1: Dead + Live + Crane Case 1



Case 2: Dead + Live + Crane Case 2



Case 3: Live



CHECK GLULAM BEARING CONNECTION

BEARING AREA $7''(7'') = 49\,in^2$

$F_{c\perp} = 510\,psi$

$F_{c\perp}' = 510\,psi\,(1.0)(1.0)(1.0)(1.0) = 510\,psi$

$f_{c\perp} = \dfrac{18{,}450^{lb}}{49\,in^2} = 376.5\,psi$

$f_{c\perp} < F_{c\perp}'$ ✓

CHECK COLUMN CAPACITY

21' UNBRACED LENGTH

7" x 7" PSL 1.8E

$P_{all} = 19.1^k$ [$C_D = 1.0$][PER TRUSJOIST CATALOG]

$P = 18.45^k$

$P < P_{ALLOWABLE}$ ✓

Allowable Axial Loads (lbs) for 1.3E TimberStrand® LSL

Column Bearing Type	Effective Column Length	Column Size 3½" x 3½" 100%	3½" x 3½" 115%	3½" x 3½" 125%	3½" x 4⅜" 100%	3½" x 4⅜" 115%	3½" x 4⅜" 125%	3½" x 5½" 100%	3½" x 5½" 115%	3½" x 5½" 125%	3½" x 7¼" 100%	3½" x 7¼" 115%	3½" x 7¼" 125%	3½" x 8⅝" 100%	3½" x 8⅝" 115%	3½" x 8⅝" 125%
On Column Base	3'	12,165	13,665	14,625	15,210	17,085	18,280	19,120	21,475	22,980	25,205	28,310	30,290	29,985	33,680	36,035
	4'	10,745	11,830	12,490	13,435	14,790	15,610	16,885	18,590	19,625	22,260	24,505	25,870	26,480	29,155	30,780
	5'	9,120	9,810	10,215	11,400	12,265	12,765	14,335	15,420	16,050	18,895	20,325	21,155	22,480	24,180	25,170
	6'	7,550	7,985	8,235	9,440	9,980	10,295	11,865	12,550	12,945	15,640	16,540	17,060	18,610	19,680	20,300
	7'	6,235	6,525	6,695	7,795	8,160	8,370	9,800	10,255	10,520	12,915	13,520	13,870	15,365	16,085	16,500
	8'	5,195	5,400	5,515	6,490	6,750	6,895	8,160	8,485	8,670	10,755	11,185	11,430	12,795	13,305	13,595
	9'	4,375	4,525	4,610	5,465	5,655	5,765	6,870	7,110	7,245	9,060	9,370	9,550	10,775	11,150	11,360
	10'	3,725	3,840	3,905	4,655	4,795	4,880	5,850	6,030	6,135	7,715	7,950	8,085	9,175	9,460	9,620
	12'	2,785	2,855	2,895	3,480	3,565	3,615	4,375	4,485	4,545	5,770	5,910	5,995	6,860	7,030	7,130
	14'	2,155	2,200	2,225	2,695	2,750	2,780	3,385	3,455	3,495	4,465	4,555	4,610	5,310	5,420	5,485
On Wood Plate(1)(2)	3'–7'	5,765	5,765	5,765	7,065	7,065	7,065	8,740	8,740	8,740	10,785	10,785	10,785	12,830	12,830	12,830
	8'	5,195	5,400	5,515	6,490	6,750	6,895	8,160	8,485	8,670	10,755	10,785	10,785	12,795	12,830	12,830
	9'	4,375	4,525	4,610	5,465	5,655	5,765	6,870	7,110	7,245	9,060	9,370	9,550	10,775	11,150	11,360
	10'	3,725	3,840	3,905	4,655	4,795	4,880	5,850	6,030	6,135	7,715	7,950	8,085	9,175	9,460	9,620
	12'	2,785	2,855	2,895	3,480	3,565	3,615	4,375	4,485	4,545	5,770	5,910	5,995	6,860	7,030	7,130
	14'	2,155	2,200	2,225	2,695	2,750	2,780	3,385	3,455	3,495	4,465	4,555	4,610	5,310	5,420	5,485

(1) Wood plate bearing is based on compression perpendicular-to-grain stress of 425 psi adjusted per the NDS®, 3.10.4.

(2) See connection details below.

Allowable Axial Loads (lbs) for 1.8E Parallam® PSL

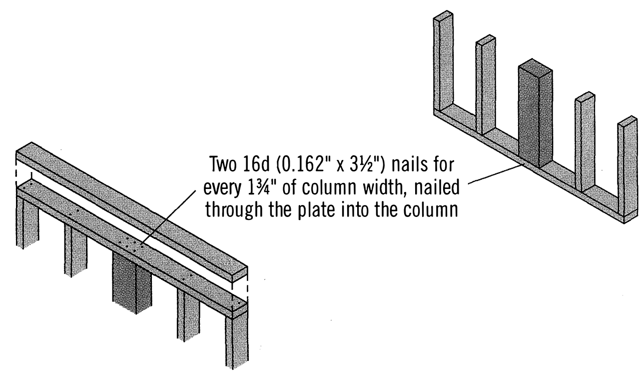
Column Bearing Type	Effective Column Length	Column Size 3½" x 3½" 100%	3½" x 3½" 115%	3½" x 3½" 125%	3½" x 5¼" 100%	3½" x 5¼" 115%	3½" x 5¼" 125%	3½" x 7" 100%	3½" x 7" 115%	3½" x 7" 125%	5¼" x 5¼" 100%	5¼" x 5¼" 115%	5¼" x 5¼" 125%	5¼" x 7" 100%	5¼" x 7" 115%	5¼" x 7" 125%	7" x 7" 100%	7" x 7" 115%	7" x 7" 125%
On Column Base	6'	10,595	11,200	11,545	15,890	16,800	17,320	21,190	22,395	23,095	33,295	36,675	38,735	40,000	40,000	40,000	40,000	40,000	40,000
	7'	8,735	9,140	9,370	13,105	13,710	14,060	17,475	18,280	18,745	30,010	32,545	34,030	40,000	40,000	40,000	40,000	40,000	40,000
	8'	7,265	7,550	7,715	10,900	11,325	11,570	14,535	15,100	15,425	26,650	28,490	29,555	35,530	37,985	39,410	40,000	40,000	40,000
	9'	6,115	6,320	6,440	9,170	9,480	9,660	12,225	12,640	12,880	23,475	24,835	25,620	31,300	33,115	34,165	40,000	40,000	40,000
	10'	5,200	5,355	5,445	7,800	8,035	8,170	10,400	10,715	10,895	20,660	21,695	22,290	27,545	28,925	29,725	40,000	40,000	40,000
	12'	3,885	3,980	4,030	5,825	5,965	6,050	7,765	7,955	8,065	16,160	16,805	17,175	21,545	22,405	22,900	40,000	40,000	40,000
	14'	3,000	3,065	3,100	4,500	4,595	4,645	6,005	6,125	6,195	12,890	13,315	13,560	17,185	17,755	18,080	34,155	35,785	36,720
	16'										10,480	10,775	10,950	13,970	14,370	14,595	28,485	29,640	30,300
	18'										8,670	8,885	9,010	11,560	11,850	12,010	24,020	24,860	25,345
	20'	Slenderness ratio exceeds 50									7,285	7,445	7,535	9,710	9,925	10,050	20,475	21,110	21,475
	22'																17,630	18,125	18,405
	24'																15,325	15,715	15,935

General Notes

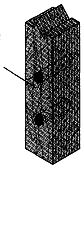
- Tables are based on:
  - Solid, one-piece column members used in dry-service conditions
  - Bracing in both directions at column ends
  - NDS®
  - Simple columns with axial loads only. For side loads or other combined bending and axial loads, see the NDS®
- Allowable loads have been adjusted to accommodate the worst case of the following eccentric conditions: ⅙ of column thickness (first dimension) or ⅙ of column width.
- Beams and columns must remain straight to within $5L^2/4608$ (in.) of true alignment. L is the unrestrained length of the member in feet.

For column allowable design stresses see page 5.

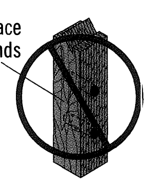
Top or Bottom Plate Connection



The column and connector values listed are for dry-service conditions ONLY. When wet-service conditions exist, contact your Weyerhaeuser representative for other product solutions.



In order to use the manufacturer's published capacities when designing column caps, bases, or holdowns for uplift, the bolts or screws must be installed perpendicular to the wide face of strands as shown at left.



DO NOT install bolts or screws into the narrow face of strands

FORCE REQUIRED TORSIONAL RESTRAINT IN CONNECTION

BRIDGE GIRDER

W14x82

$L_u = 40'$ $k = 1.0$

FIND AVALIABLE COMPRESSIVE STRENGTH

$$\frac{kL}{r} = \frac{1.0(40')(12''/1)}{2.48''} = 193.5 > 113.4 = 4.71\sqrt{\frac{E}{F_y}} \qquad 193.5 < 200 \checkmark$$

$$F_e = \frac{\pi^2 E}{\left(\frac{kL}{r}\right)^2} = \frac{\pi^2(29000\text{ksi})}{(193.5)^2} = 7.64\text{ksi}$$

$$F_{CR} = 0.877F_e = 0.877(7.64\text{ksi}) = 6.70\text{ksi}$$

$$P_c = A_g F_{CR} = 24.1\text{in}^2(6.70\text{ksi}) = 161.6^k$$

TORSIONAL RESTRAINT FORCE = 2% OF AVALIABLE COMP. STRENGTH [PER AISC 318 § E6.2 LACING REQUIREMENTS]

$$161.6^k(0.02) = \boxed{3.23^k}$$

RAILS

W14x34 (ESTIMATE)

$L_u = 12'$ $k = 1.0$

FIND AVALIABLE COMPRESSIVE STRENGTH

$$\frac{kL}{r} = \frac{1.0(12')(12''/1)}{1.53''} = 94.1 \leq 113.4 = 4.71\sqrt{\frac{E}{F_y}}$$

$75.4 < 200 \checkmark$

$$F_e = \frac{\pi^2 E}{\left(\frac{kL}{r}\right)^2} = \frac{\pi^2(29000\text{ksi})}{(75.4)^2} = 32.3\text{ksi}$$

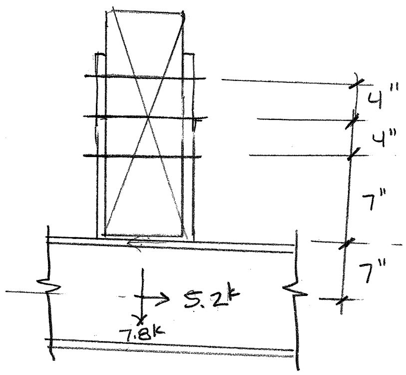
$$F_{CR} = \left[0.658^{F_y/F_e}\right]F_y = \left[0.658^{50/32.3}\right]50\text{ksi} = 26.2\text{ ksi}$$

$$P_c = A_g F_{CR} = (10.0\text{in}^2)(26.2\text{ksi}) = 262^k$$

TORSIONAL RESTRAINT FORCE = 2% OF AVALIABLE COMP STRENGTH [PER AISC 318 § E6.2 LACING REQUIREMENTS]

$$262^k(0.02) = \boxed{5.2^k}$$

BOLTS FOR RAIL TO GLULAM CONNECTION

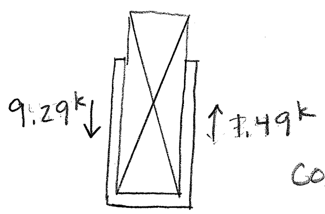


ASD D+L

MOMENT FROM TORSIONAL FORCE RESISTED BY TWO PLATES

$$\frac{5.2^k(7'')}{6.75''} = 5.39^k$$

$$5.39^k + \frac{7.8^k}{2} = 9.29^k \qquad \frac{7.8^k}{2} - 5.39^k = -1.49^k$$



CAPACITY BOLTS

1" ϕ

CONSIDERING AS A SINGLE-SHEAR CONNECTION B/C SHEAR UNEQUAL

NDS T12.D [6 3/4" MAIN MEMBER, G = 0.5
1/4" ASTM A36 STEEL PLATE]

$Z_\perp = 1420$ lb

$C_D = 1.25$ [load duration less then 7 days]

$C_m = 1.0 \quad C_t = 1.0$

C_Δ END DISTANCE $\frac{7''}{1''} = 7D \geq 7D \therefore C_\Delta = 1.0$

SPACING IN ROW - LOADING $\perp$ - GOVERNED BY STEEL

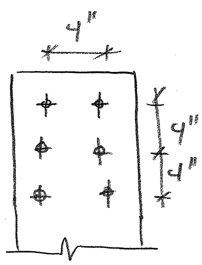
$C_\Delta = 1.0$

$C_g \approx 0.75$ [GUESS TO ESTIMATE # BOLTS]

$Z_\perp' = 1420^{lb}(1.25)(0.75) = 1331^{lb}$

$$\# \text{BOLTS} = \frac{9.29^k}{1331^{lb}/\text{BOLT}} = 6.9$$

TRY 2 ROWS OF THREE



CALCULATE C_g

$n = 3$

$$R_{EA} = \frac{E_s A_s}{E_m A_m} = \frac{29000\text{ ksi}(2)(1/4'')(8'')}{2{,}100\text{ ksi}(6.75'')(20'')} = 0.409$$

$\gamma = 270{,}000\, D^{1.5} = 270{,}000(1'')^{1.5} = 270{,}000$

$s = 4''$

$$u = 1 + \gamma \frac{s}{2}\left[\frac{1}{E_m A_m} + \frac{1}{E_s A_s}\right] = 1 + 270{,}000\, \frac{4''}{2}\left[\frac{1}{2{,}100{,}000(6.75)(20)} + \frac{1}{29\times10^6(2)(1/4)(8)}\right]$$

$u = 1.006$

$$m = u - \sqrt{u^2 - 1} = 1.006 - \sqrt{(1.006)^2 - 1} = 0.892$$

CONTINUED NEXT PAGE

CONT. CALCULATE C_g

$$C_g = \left[\frac{m(1-m^{2n})}{n[(1+R_{EA}m^n)(1+m)-1+m^{2n}]}\right]\left[\frac{1+R_{EA}}{1-m}\right]$$

$$C_g = \left[\frac{0.892(1-0.892^6)}{3[(1+0.409(0.892)^3)(1+0.892)-1+0.892^6]}\right]\left[\frac{1+0.409}{1-0.892}\right]$$

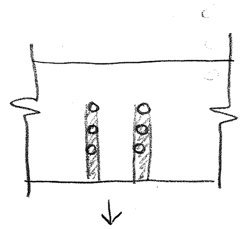
$$C_g = 0.989$$

$$Z_\perp' = 1420^{lb}(1.25)(0.989) = 1755^{lb}/\text{BOLT}$$

$$\#\ \text{BOLTS} = \frac{9.29^k}{1.755^k/\text{BOLT}} = 5.29\ \text{BOLTS}$$

$\boxed{\text{USE (6) BOLTS}}$

CHECK ROW TEAR OUT



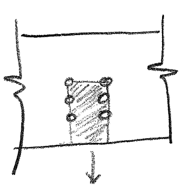
$$Z_{RT_i}' = \frac{F_v' t}{2}[n_i S_{critical}](2)$$

$$= \frac{136.8psi(6.75'')}{2}[3(4'')](2)$$

$$= 11.1^k$$

$$Z_{RT}' = 2 \text{ rows}(Z_{RT_i}) = 2(11.1^k) = 22.2^k > 9.29^k \checkmark$$

CHECK GROUP TEAR OUT

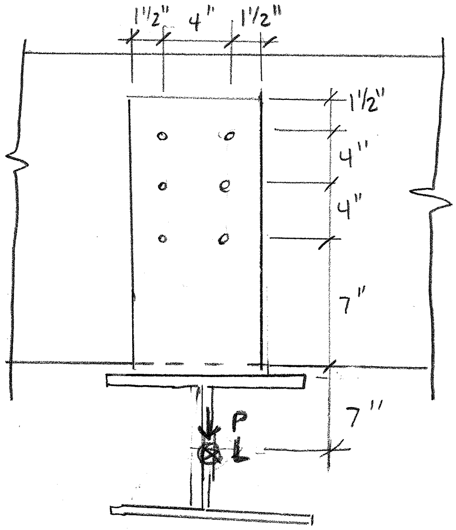


$$Z_{GT}' = \frac{Z_{RT-1}'}{2} + \frac{Z_{RT-2}'}{2} + F_t' A_{group}$$

$$Z_{GT}' = \frac{11.1^k}{2} + \frac{11.1^k}{2} + F_t' A_{group}$$

$$= 11.1^k + F_t' A_{group} > 9.29^k \checkmark$$

CHECK STEEL PLATE RAIL TO GLULAM CONNECTION



(2) ¼" THICK ASTM A36 PLATES

LRFD

$1.2D + 1.6L$

$P = 1.2(7.8^k - 4^k) + 1.6(4^k) = 10.96^k$

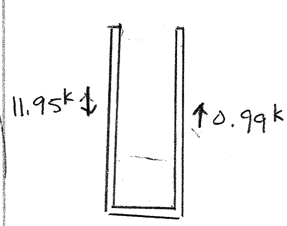
$L = 1.2(5.2^k) = 6.24^k$

MOMENT FROM TORSIONAL FORCE RESISTED BY COUPLE FORMED BY TWO PLATES

$$\frac{6.24^k(7'')}{6.75''} = 6.47^k$$

$$T_{MAX} = \frac{10.96^k}{2} + 6.47^k = 11.95^k$$

$$T_{MIN} = \frac{10.96^k}{2} - 6.47^k = -0.99^k$$



EDGE DISTANCE

AISC T J3.4

$\varnothing = 1'' \rightarrow$ MIN EDGE DISTANCE 1¼"

$\boxed{\text{USE } 1\tfrac{1}{2}'' \text{ EDGE DISTANCE}}$

CHECK TENSION

YIELD IN GROSS SECTION

$$\phi P_n = \phi F_y A_g = 0.9(36ksi)(1/4'')(7'') = 56.7^k >> 11.95^k \checkmark$$

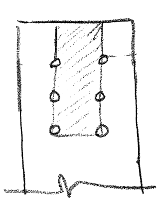
RUPTURE IN NET SECTION

$$\phi P_n = \phi F_u A_e = \phi F_u A_n U = 0.75(58ksi)((1/4'')(7'') - (2)(1'' + 1/8'')(1/4''))(1.0)$$

$$\phi P_n = 51.7^k >> 11.95^k \checkmark$$

OKAY

CHECK BLOCK SHEAR



CHECK RUPTURE

$$\phi R_n = \phi[0.6 F_u A_{nv} + U_{bs} F_u A_{nt}]$$

$$= 0.75[0.6(58ksi)(1/4'')(2)(9.5'' - 2.5(1'' + 1/8'')) + 1.0(58ksi)(1/4'')(4'' - (1'' + 1/8''))]$$

$$= 119^k$$

CHECK PROGRESSIVE FAILURE

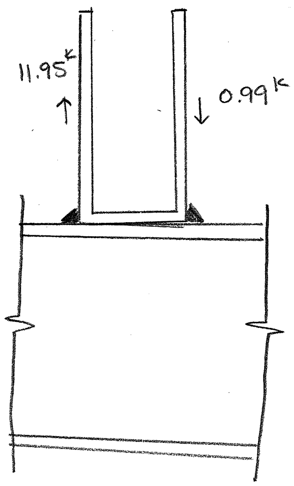
$$\phi R_n = \phi[0.6 F_y A_{gv} + U_{bs} F_u A_{nt}]$$

$$= 0.75[0.6(36ksi)(1/4'')(2)(9.5'') + 1.0(58ksi)(1/4'')(4 - 1\tfrac{1}{8}'')]$$

$$= 108^k > 11.95^k \checkmark$$

OKAY

WELD DESIGN FOR RAIL TO GLULAM CONNECTION



E-70XX WELD

FILLET WELD
STRENGTH PER 1/16 OF AN INCH

$$70\text{ksi}\,(0.6)\left(\tfrac{1}{16}''\right)\left(\tfrac{1}{\sqrt{2}}\right) = 1.856^k \big/ \left(\tfrac{1}{16}''\right)(\text{in}^2)$$

REQUIRED WIDTH

$$\frac{11.95^k}{\dfrac{1.856^k\,(7'')}{\left(\tfrac{1}{16}''\right)(\text{in}^2)}} = 0.91$$

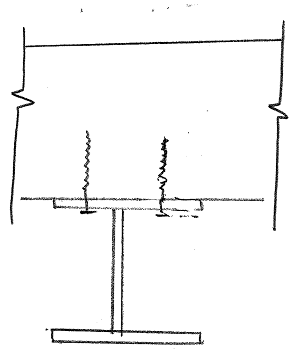
ONLY NEED 1/16" FILLET WELD

$\boxed{\text{USE } 1/8'' \text{ E-70XX FILLET WELD}}$

FOR CONSTRUCTIBILITY

LAG SCREWS FOR RAIL TO GLULAM CONNECTION

* MEANT TO BE A SECONDARY FAILSAFE SYSTEM DESIGNED JUST FOR GRAVITY LOADING FROM CRANE



NDS T12.2A [WITHDRAWAL CAPACITY LAG SCREW]

SPECIFIC GRAVITY = 0.5 (DOUG FIR-LARCH)

LAG SCREW DIAMETER = 1"

$W = 636 \text{ lb/in}$

$W' = W C_D C_m C_t C_{eg}$

$C_D = 1.0$ [D+L]

$C_m = 1.0$ $C_t = 1.0$

$C_{eg} = 1.0$ [SCREW AXIS PERENDICULAR TO WOOD FIBERS]

$W' = W = 636 \text{ lb/in}$

LENGTH THREAD PENETRATION

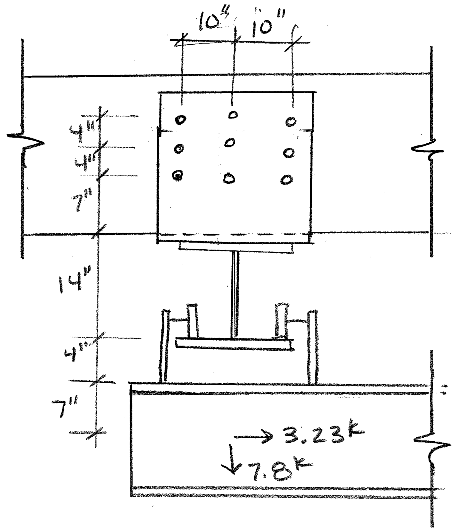
$$P_t = \frac{P}{W'} = \frac{7800 \text{ lb}}{(2 \text{ lag screws})(636 \text{ lb/in})} = 6.13"$$

$\boxed{\text{USE 11" LONG 1"}\varnothing\text{ LAG SCREW}}$

NDS T.L2 $P_t = 6"$

BOLTS FOR CONNECTION BETWEEN RAIL AND GLULAM

* ALTERNATE CONNECTION THAT PROVIDES TORSIONAL RESTRAINT TO BRIDGE GIRDER



ASD
D+L

MOMENT CAUSED BY LATERAL FORCE RESISTED BY COUPLE FORMED BY ROWS OF BOLTS

$$\frac{3.23^k(36'')}{20''} = 5.814^k$$

$$\frac{5.814^k/\text{LINE}}{3\text{ BOLTS/LINE}} = 1.938^k$$

$$\frac{7.8^k}{9} = 0.867^k$$

$$\frac{3.23^k}{9} = 0.359^k$$

$$V_{bolt} = \sqrt{(0.359^k)^2 + (1.938^k + 0.867^k)^2} = 2.828^k$$

$Z_\perp = 3000^{lb}$ [1" ⌀ BOLT, 6 3/4" GLULAM MAIN MEMBER, G=0.5]

$$Z_\perp' = Z_\perp C_D C_m C_t C_g C_\Delta$$

$C_D = 1.25$ [duration seven days]

$C_m = 1.0$ $C_t = 1.0$

$C_g \approx 0.76$

$C_\Delta = 1.0$

$$Z_\perp' = 3000^{lb}(1.25)(0.76) = 2.850^k$$

$Z_\perp' > Z_\perp$ ✓

Poehlitz 62

Appendix F- Drawings

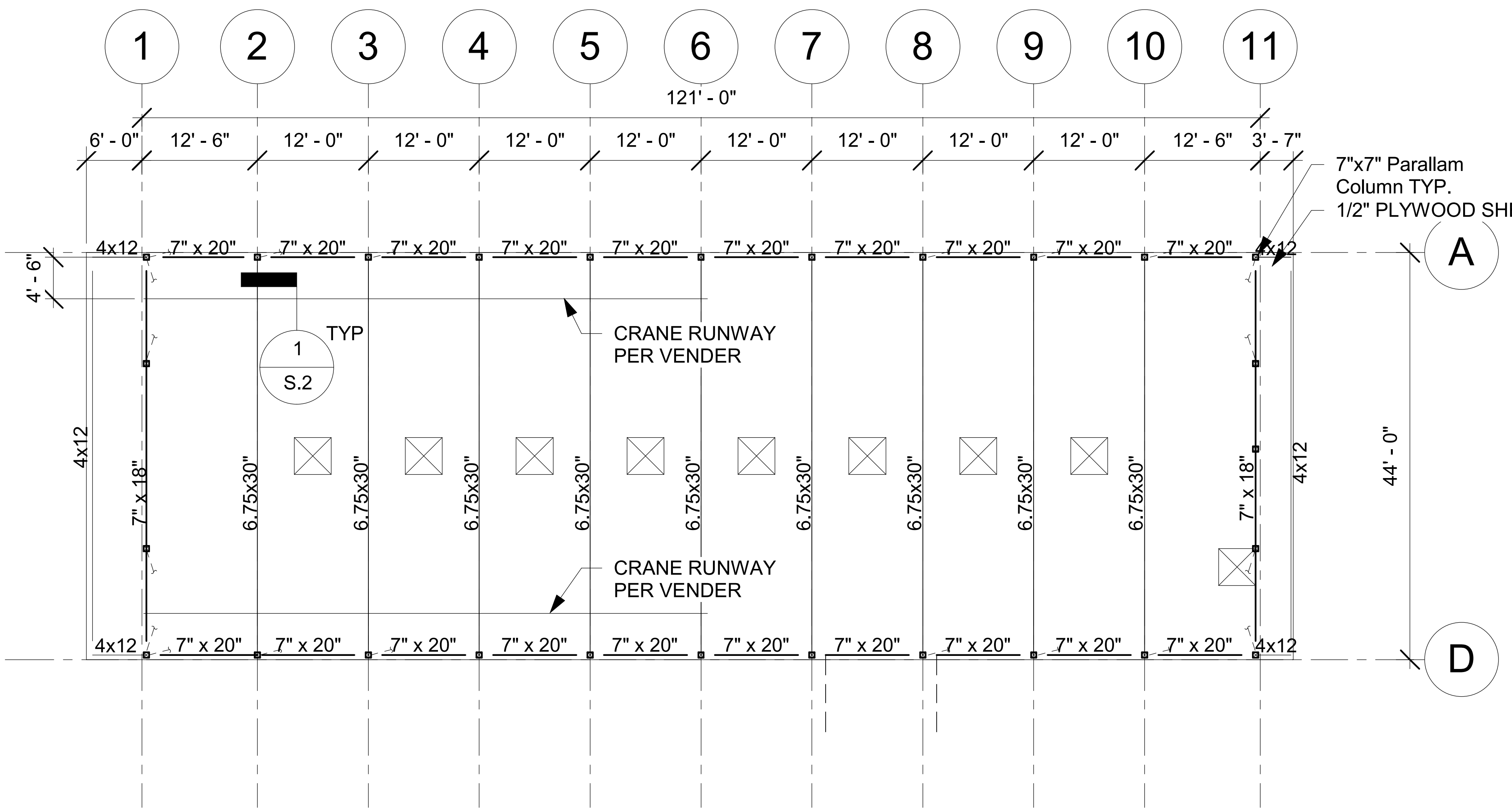
AMY POEHLITZ

Simpson Strong-Tie Crane

1 Grand Ave.
San Luis Obispo CA
93407

ROOF FRAMING PLAN

STRUCTURAL NOTES
1. Sub-Purlins at 4'-0" on center omitted for clarity
2. Verify in field runways are placed directly below fire sprinklers



1 ROOF FRAMING PLAN
1/16" = 1'-0"

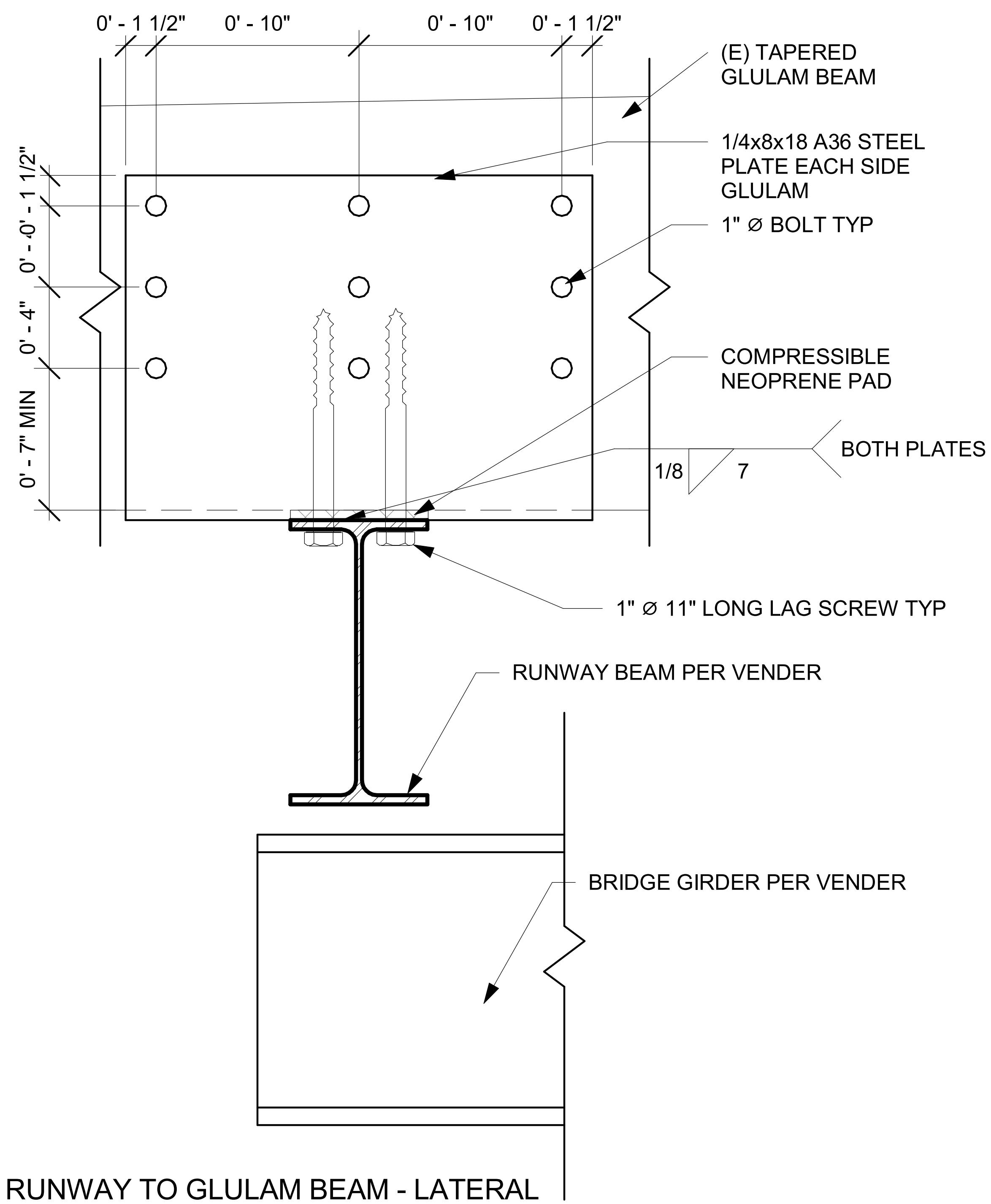
DATE: 6/8/2015

Drawn by: AMP

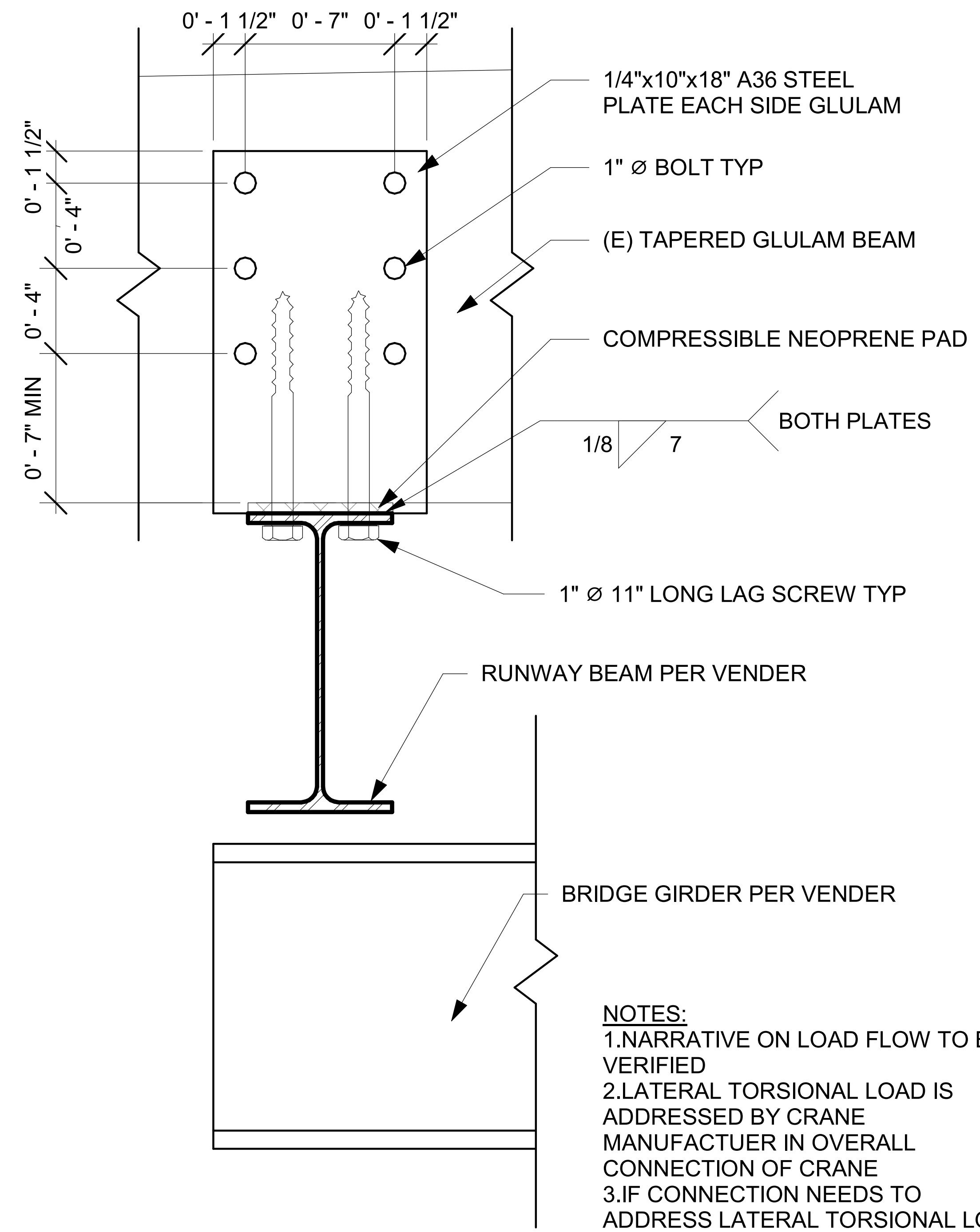
Checked by: Checker

SCALE: 1/16" = 1'-0"

S.1



2 RUNWAY TO GLULAM BEAM - LATERAL TORSIONAL LOAD
1 1/2" = 1'-0"



NOTES:
1.NARRATIVE ON LOAD FLOW TO BE VERIFIED
2.LATERAL TORSIONAL LOAD IS ADDRESSED BY CRANE MANUFACTUER IN OVERALL CONNECTION OF CRANE
3.IF CONNECTION NEEDS TO ADDRESS LATERAL TORSIONAL LOAD SEE DETAIL 2/S.2

1 RUNWAY TO GLULAM BEAM
1 1/2" = 1'-0"

AMY POEHLITZ

Simpson Strong-Tie Crane

1 Grand Ave.
San Luis Obispo CA
93407

DETAILS

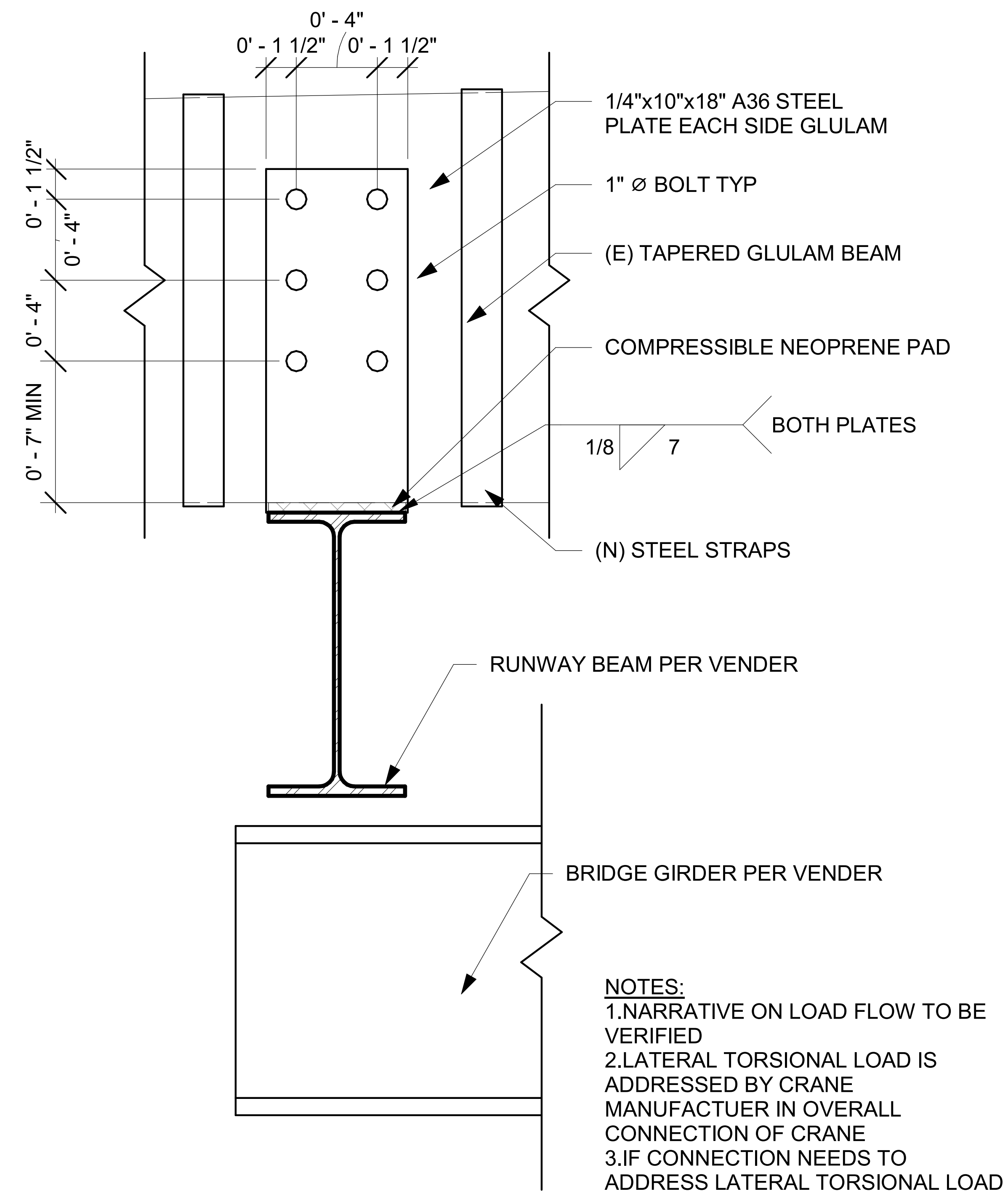
DATE:
6/8/2015

Drawn by:
AMP

Checked by:
Checker

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

S.2



NOTES:
1.NARRATIVE ON LOAD FLOW TO BE VERIFIED
2.LATERAL TORSIONAL LOAD IS ADDRESSED BY CRANE MANUFACTUER IN OVERALL CONNECTION OF CRANE
3.IF CONNECTION NEEDS TO ADDRESS LATERAL TORSIONAL LOAD SEE DETAIL 2/S.2

1 RUNWAY TO GLULAM BEAM - STRAPS
1 1/2" = 1'-0"

AMY POEHLITZ

Simpson Strong-Tie Crane

1 Grand Ave.
San Luis Obispo CA
93407

DETAILS

DATE:
6/8/2015

Drawn by:
Author

Checked by:
Checker

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

S.3