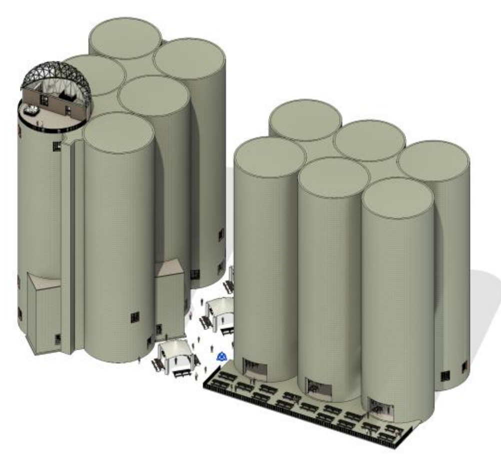
Grain Silo Adaptive Reuse Project
Bondurant, Iowa

Brady Adams
Jack Muller
Alexa Vekich
Alexis Isenberg



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<u>Section I</u> Executive Summary

The Grain Silo Adaptive Reuse Project evaluated the feasibility of repurposing two six-pack grain silo structures located at 101 Lincoln St. NE, Bondurant, Iowa, into multi-purpose conference, residential, and community-oriented spaces. The objective of the project was to preserve the site's agricultural character while supporting future economic development and community growth.

The project was completed by a multidisciplinary student design team of University of Iowa Civil and Environmental Engineering students with experience in civil engineering, site planning, and structural evaluation. The team was tasked with developing feasible adaptive reuse concepts and providing planning-level design and analysis to support stakeholder decision-making.

As part of the project, the team delivered three conceptual redevelopment alternatives, along with supporting materials including conceptual site plans, structural feasibility evaluations, building code considerations, and redevelopment strategies. These design services were intended to guide future phases of development, including detailed design, permitting, and construction planning.

The recommended approach was to pursue adaptive reuse of the silo structures while maintaining their exterior appearance and historical identity. Proposed uses included conference and event space, residential units, public gathering areas, small-scale commercial opportunities, and interpretive exhibits highlighting Bondurant's agricultural heritage. The silos were also positioned as a central feature within a larger development plan, with potential integration alongside a future hotel.

A key constraint for this project was developing a design that integrates with the existing silo layout while preserving its historic architectural character, which limited the extent of modifications due to the size, geometry, and structural boundaries of the existing facility. Time was also a significant constraint, as the project was completed within a single spring semester, requiring efficient coordination and decision-making to produce a complete and viable design. In addition, any new structural elements outside the silo footprint needed to be properly tied into the existing structure, and future development plans for the Grain District, including potential condominiums and hotels, further influenced design limitations. Key challenges included selecting an appropriate floor framing system, such as column-supported framing versus a C-channel ring system and addressing insulation requirements while minimizing loss of usable interior space. Incorporating a tempered glass dome for the penthouse also presented structural and detailing challenges. Finally, all additions needed to align with the district's agricultural aesthetic and preserve the silos' visual significance.

The project was completed over a multi-week design period and included regular stakeholder coordination, bi-weekly meetings, and site visits to support design development. These efforts ensured that the proposed concepts aligned with stakeholder priorities and site-specific conditions.

Financial considerations were addressed at a conceptual level. While detailed construction cost estimates were not developed, the project provided planning-level cost awareness and emphasized phased, incremental development strategies to minimize financial risk. This approach allowed flexibility in implementation and supported the pursuit of external funding sources, including state and federal grants.

Overall, the project determined that adaptive reuse of the grain silos was feasible and provided significant potential benefits. The proposed designs preserved the historical identity of the site while introducing functional and economically viable uses.

Based on these findings, it was recommended that stakeholders proceed with the next phase of development, including detailed structural analysis, refined design, and funding acquisition.

Section II Organization Qualifications and Experience

1. **Organization and Design Team Description**
   As a team we strive to deliver the best products for our clients and help their ideas become a reality. We work closely with our clients to determine their value and what is important to them for their projects. From there, our team works together to come up with a design that exceeds our client's expectations and meets all of their needs. The design team assigned to this project is uniquely equipped to tackle the many challenges of a fully involved design like this one. The design team consists of Brady Adams, Jack Muller, Alexis Isenberg, and Alexa Vekich. They are all senior civil engineering students at the University of Iowa in a capstone design course. Brady is pursuing a focus in structures, mechanics, and materials. He will be overseeing any structural engineering work. Jack has a pre-architecture focus in his coursework and will therefore be leading the architectural design elements of this project. Alexis is focusing on her engineering studies on management. She will lead the site design for the surrounding area. Alexa is seeking a structural focus area and will be working on structural engineering elements.

2. **Description of Experience with Similar Projects**
   The design team brings a range of relevant experiences to this project. All members have completed relevant coursework in engineering design. Furthermore, they have solidified their education with abundant employment experience.

   Brady worked as a structural engineering intern at Nelson Engineering, where he contributed to both residential and commercial structural design projects. In this role, he assisted with the development, analysis, and documentation of structural systems using industry-standard software, including RISA and Civil3D. He supported the engineering team in preparing design calculations, reviewing structural models, and ensuring that project deliverables aligned with applicable codes and client requirements.

Jack worked as an operations intern at McCown Gordon Construction, where he supported the design and coordination of the installation of a commercial sprinkler system. In this role, he contributed to project planning and collaboration efforts to ensure the system design aligned with construction requirements and site conditions. He also gained experience developing design concepts for rail projects, producing client-facing deliverables that communicated engineering ideas and project solutions.

Alexis has academic experience in structural engineering, hydraulics and hydrology, and transportation systems. She has applied technical tools such as AutoCAD, Civil 3D, MATLAB, and ArcGIS Pro to support analysis and design development throughout the project. Prior coursework experience contributed to the ability to evaluate structural systems, interpret site and spatial data, and develop engineering design solutions aligned with real-world constraints. In addition to technical skills, she brought experience in leadership, coordination, and time management from previous professional roles in fast-paced environments, which supported efficient collaboration and task completion throughout the design process.

Alexa worked as a site civil intern and was included in the design process of over 20 projects across the Midwest. She has helped engineer parking lots and exterior spaces for multiple projects like this one including event spaces, restaurants, and country clubs. In addition to these experiences, multiple team members worked on survey crews to develop their spatial understanding and ability to create successful construction documents.

Section III Proposed Services

1. **Project Scope**

   The scope of the Grain Silo Adaptive Reuse Project included the evaluation and conceptual redevelopment of two existing six-pack grain silos located at 101 Lincoln St. NE in Bondurant, Iowa. The primary goal of the project was to assess the feasibility of repurposing the silos into functional mixed-use spaces while also preserving their historical agricultural character.

   The project scope consisted of developing planning-level design concepts for adaptive reuse of the silo complexes, including potential conference spaces, residential units, and small-scale commercial opportunities. The proposed designs were required to maintain the exterior appearance of the silos and integrate cohesively with the surrounding Grain District redevelopment plan. Including a potential future hotel development adjacent to the project site.

   The design team was responsible for preparing three conceptual alternatives supported by preliminary site layouts, floor plans, and architectural recommendations. The scope also included a structural feasibility evaluation to identify constraints associated with the existing silo geometry, potential floor framing systems, and the reinforcement of existing and proposed openings required for the layout. Additional scope items included evaluation of insulation strategies, building code considerations, and the integration of exterior additions.

The project also included stakeholder coordination through bi-weekly meetings and a site visit to confirm project priorities, verify site conditions, and ensure alignment with project goals. The final deliverable for the project included a conceptual design accompanied by planning documents, and a recommended redevelopment approach to help guide future phases of detailed design and construction planning.

2. **Methods and Design Guides**
   In the case of repurposing the two six-pack silo structures, the 2021 International Building Code (IBC) has been adopted and will be used to design the structure. Other specifications required for design include:
   - The City of Bondurant Ordinance No. 250407-205
   - City of Bondurant Code of Ordinances chapters 145-167, 175-182
   - Iowa Statewide Urban Design and Specifications (SUDAS)
   - ASCE 7-22 Minimum Design Loads and Associated Criteria for Buildings and Other Structures
   - AISC 360-22 Specification for Structural Steel Buildings 16^{th} Ed
   - ACI 318025 Building Code Requirements for Structural Concrete.

<u>Section IV</u> Constraints, Challenges, and Impacts

1. **Constraints**
   A key constraint of this project was developing a design that integrated into the existing silo layout while preserving the original appearance and character of the structures. The silo geometry and limited interior space created design constraints, as majority of the modifications had to remain within the existing structural boundaries. Time was also a significant constraint. Given the limited duration of the spring semester, the project required efficient coordination and decision making to produce a complete design by the deadline.

   Another major constraint was the incorporation of any new structural elements outside of the existing silos. All exterior additions had to align aesthetically with the proposed Grain District redevelopment while also accounting for the potential future construction of a hotel near the silos. Additionally, the planned Silo Commons located south of the two silo complexes restricted the available outdoor space and required that the proposed design remained compatible with the surrounding development plans.

2. **Challenges**
   One of the main challenges we encountered during the project was determining an effective way to frame the floors at each level. This included evaluating potential support systems, such as interior column supports or a C-channel ring beam capable of supporting beams spanning the silo and the associated floor systems. Another challenge involved selecting an insulation option that provided the required R-value while minimizing the

loss of usable interior space within the silos. A third challenge was incorporating a tempered glass dome above the penthouse sleeping quarters.

The City of Bondurant placed a strong emphasis on preserving its agricultural history, and the existing grain silos served as a prominent visual representation of their heritage. As a result, all new structural additions had to properly tie into the existing silos while maintaining their original appearance and avoiding significant alterations to the exterior. The final major challenge was reinforcing existing openings in the silos and designing additional openings required for the proposed floor layout.

3. **Societal Impact within the Community**

This project has the potential to create meaningful societal benefits for the Bondurant community by transforming underutilized industrial structures into active, accessible spaces that support downtown vitality. By repurposing the ground floors of the grain silos for commercial or public use, the project encourages local economic growth, fosters small business opportunities, and creates places for community gathering and interaction. Through the incorporation of the western silo complex, the preservation of the community's agricultural history can be put on display and offer a reminder to the community of the town's origins.

In addition to its historical and social significance, the project is also expected to have an economic impact on the community. In the short term, the development could impact residents through potential tax increases, as the City of Bondurant needs to fund various aspects of the project. However, the City could pursue grant opportunities and alternative funding sources to reduce the financial burden on the community. In the long term, the project has the potential to support economic growth by attracting visitors and increasing activity within the local area. The proposed silo redevelopment, along with the broader Grain District, is anticipated to become a central destination within Bondurant.

The project is also expected to have environmental impacts. Paving around the exterior of the silos could have an effect on stormwater runoff patterns and drainage on the site. Additionally, the inclusion of restaurants introduces potential impacts associated with building ventilation and localized air pollutants. Proposed landscaping around the silos is intended to help mitigate these effects by improving stormwater retention and reducing overall environmental impact.

Section V Alternative Design Options

There were various alternative options that could've been incorporated into these versatile silo bases. These ideas included:

- Speakeasy/Mocktail lounge to provide a hidden, moody atmosphere that can later turn into the hotel lobby bar/lounge after its construction.
- Interchangeable interactive entertainment. This space can be transformed into escape rooms, and seasonal attractions such as a haunted silo (fall) or a holiday market with lights (winter).

This adaptable area will be ever-changing, keeping visitors and locals on their toes throughout the year.

- Multiple pods can be transformed into a food-truck style restaurant, where local food joints can come together for periods of time and showcase their food, products, and culinary identity to the community. These periodic restaurants could then be converted into one hotel restaurant after its final construction.
- One pod can be used as an immersive projection experience, where themes can include history, seasons, art, agricultural history, etc. The experience allows for a movie-theatre type atmosphere that engulfs the audience in sound and visuals.
- Aligning with the long-term plan: We could repurpose the western set of six packs to promote the future of the silos. A hotel lobby integration could be used as a welcome or concierge space for visitor orientation and event support, as well as ticketing/check in for silo experiences located in the eastern set of silos. Areas in the future hotel lobby could outline the construction and future of the Grain District Redevelopment Plan that way locals can envision the future of their home.

See below for visual representations of the three alternatives that the design team has proposed.

Alternative 1 focuses on creating a unique dining experience by utilizing the space located between the two six-packs. The design offers space for five individual restaurants acting as an outdoor food hall, with covered dining and seating options for visitors. Along with the dining options the design incorporates retail space and a large play area for children within the silos. Lastly, in alternative 1 the remaining space within the western complex offers the potential for a future hotel lobby with support spaces branching off either side of the lobby. Some benefits of alternative 1 include a strong emphasis on community gathering through the outdoor food hall concept, high foot traffic potential that supports surrounding businesses, and flexible outdoor space that can host events and seasonal activity. On the contrary, some challenges this alternative faces include a heavy reliance on outdoor use may limit year-round functionality, less defined interior programming within some silos, and that hotel integration is less developed compared to other alternatives

Figure 1: Alternative 1

Alternative 2 offers more retail space within the Eastern complex, providing local businesses with more opportunities. Like alternative 1 there is a children's play area, however it is located next door to the restaurant offering entertainment to families dining at the restaurant with children. With alternative 2 a drive through/car port is located between the two silo complexes. This allows easy access to the restaurant and the future hotel lobby located across from the restaurant. Next to the potential hotel lobby is a speakeasy/mocktail lounge that can be incorporated into the lobby and offer a more adult scene. The last silos in the western complex would be transformed into a coffee shop and bakery next to each other. Benefits of this alternative include strong support for local businesses through expanded retail space, improved accessibility, and circulation with drive-through access, as well as well-integrated hotel amenities create a cohesive visitor experience. While some negatives of this alternative include less emphasis on large, shared community gathering spaces, drive-through may reduce pedestrian-oriented atmosphere, and more structured layout may limit flexibility for future changes.

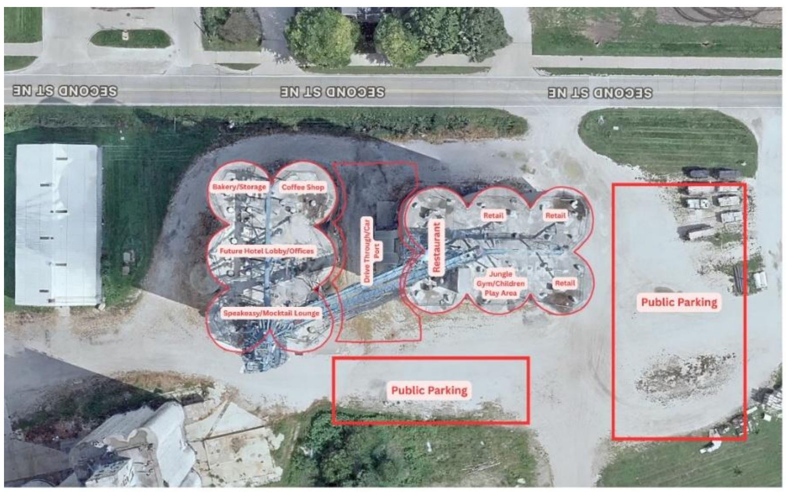


Figure 2: Alternative 2

Alternative 3 offers a combination of the first two alternatives. The space between the two silo complexes would be turned into a covered outdoor space that would allow visitors to relax in an outdoor setting. Like alternative 2 the open space between the silos would allow access to both the restaurant and the future hotel lobby, giving guests of both a unique place to spend their time. The main difference with alternative 3 is the incorporation of an interchangeable entertainment space that could house seasonal events or displays. As well as an immersive projection experience providing visitors and the community with a unique activity within the new development. Benefits of this alternative include a balanced mix of community space, commercial use, and entertainment, flexible programming allows the site to evolve over time, and unique attractions like immersive projection and event space increase appeal. Downsides of this alternative include a more complex design may increase cost and implementation challenges, it requires careful management to maintain flexibility over time, and it may be harder to define a single strong identity compared to simpler concepts

Figure 3: Alternative 3

<u>Section VI</u> Final Design Details

Level 1: Eastern Six-Pack

For our final design, we designated the southern three silos in the eastern six-pack as restaurant space because it creates a strong food destination that draws people into the site, with outdoor patio seating oriented toward the silo commons to connect directly to the proposed amphitheater and community space. We designated one silo as a children's play area because it makes the development more accessible for families, with a direct connection to the restaurant to create a seamless experience for parents and children. We designated another silo as a retail space because it supports small businesses and allows for flexibility through permanent tenants or rotating pop-up shops, and we designated the final silo in the eastern six-pack as an interchangeable entertainment space because it allows the site to adapt over time with different interactive uses and events.

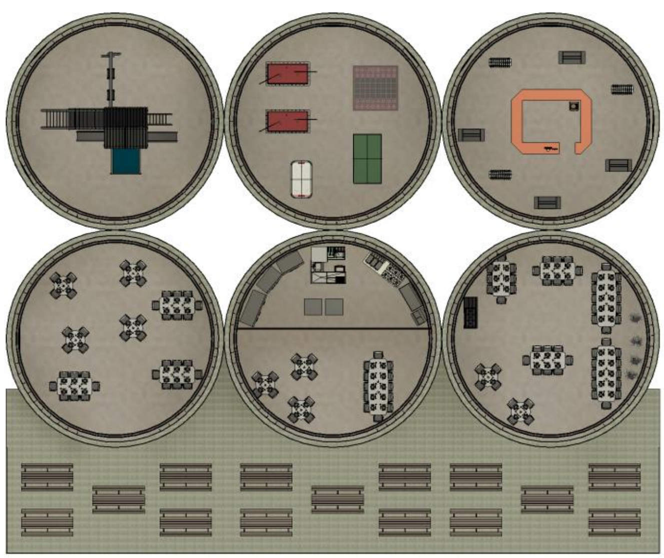


Figure 4: Level 1, Eastern Six-Pack

Level 1: Western Six-Pack
For the western six-pack, we designated two silos as a primary and secondary hotel lobby because it creates a clear point of arrival while improving circulation and providing direct access to elevators leading to the conference level and penthouse suite. We designated the remaining silos for amenities including a hotel gym, coffee shop and bakery, BBQ restaurant with drive-through, and a speakeasy because these uses serve both hotel guests and the public, helping maintain activity throughout the day. We also designated the space between the silos as shaded picnic-style seating because it provides a comfortable, informal gathering area that ties the entire development together and encourages people to stay and engage with the space.

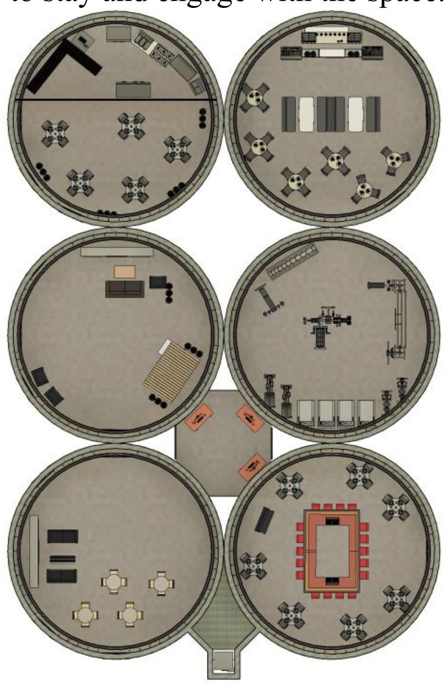


Figure 5: Level 1, Western Six-Pack

Level 2: Western Six-Pack
We designated the entire second level of the western six-pack to be a conference level, where companies could reserve the space for meetings & other collaborative purposes. This creates a shared space that local businesses, organizations, and community groups can access for meetings, events, and gatherings, helping activate the site and support nearby economic activity. We designated this as a flexible, reservable space because it encourages consistent activity throughout the site, helping bring people in on a regular basis rather than limiting the area to a single use.

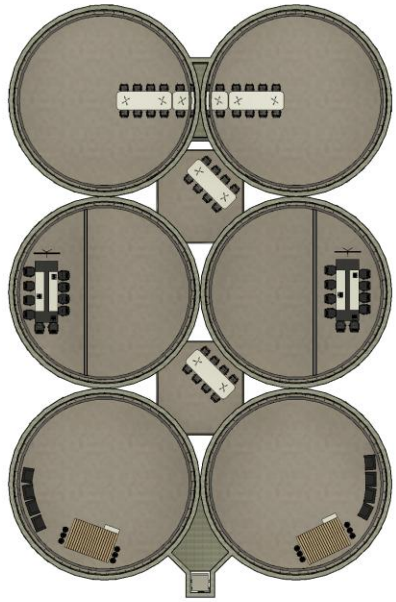


Figure 6: Level 2, Western Six-Pack

Levels 4 and 5: Western Six Pack

We designated levels 4 and 5 as a penthouse suite because it brings together the full range of uses envisioned for the silo project into one cohesive, high-value space. The lower level of the was adapted into comfortable, functional housing, including a kitchen, bathroom, dining area, and lounge space. We separated the sleeping quarters onto level 5 because it creates a clear distinction between living and resting spaces, improving privacy and making the layout feel more intentional and livable. The two levels are accessible via staircase. We elevated the sleeping quarters above the existing silo height because it maximizes views of downtown Burlington and the future grain district, turning the space into a unique vantage point rather than just another room. We included rooftop and balcony access because it extends the usable space outdoors, giving occupants a direct connection to the surrounding environment and enhancing the overall experience of the penthouse. The roof of the sleeping quarters was designed as a tempered glass half-dome because it allows natural light to fill the space during the day while also creating an opportunity for stargazing at night, making the suite feel distinctive and memorable.

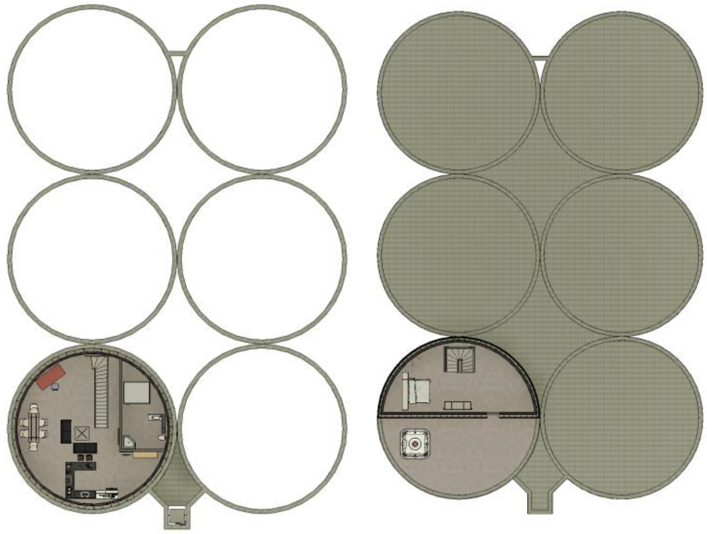


Figure 7: Level 4 and 5, Western Six-Pack

Site Improvements

A virtual soil survey was completed to assess the hydrologic soil profile of the site. A soil can belong to one of four hydrologic soil groups: A, B, C, or D. Group A soils are generally the most permeable and D are the least. The soil survey results are as follows: Nicollet Loam 51.4 percent, and Canisteo Clay 48.6 percent. Nicollet Loam is a type B or D soil, and Canisteo Clay is a type C or D soil. Based on these results, calculations assumed a type D soil. This selection tends to overestimate the amount of runoff on the site to ensure that design selections convey heavy rainfall. Understanding the hydrologic soil profile of the site allowed for proper selection of curve numbers for ground cover types. Curve numbers were read from the NRCS TR-55 Tables provided by the USACE. Results of this research are shown below.

Pre-development calculation of composite curve number

Ground Surface	Area (Acres)	Curve Number
Concrete	0.1193	98
Metal Grates	0.0092	98
Gravel	0.7262	91
Total	0.8547	92

Post-development calculation of composite curve number

Ground Surface	Area (Acres)	Curve Number
Road Concrete	0.0727	98
Patio Concrete	0.2167	98
Grass (Good Condition)	0.5652	80
Total	0.8547	86

In each case, the relative "weight" of each ground cover type was calculated and multiplied by the associated curve number. The sum of these values produced the total composite curve number of the site for the pr- and post-development condition. Sample calculations using values from the post-development condition are as follows:

Compute relative "weight" of each ground surface

Concrete

$$\frac{0.0727}{0.8547} = 0.0851$$

Metal Grates

$$\frac{0.2167}{0.8547} = 0.2536$$

Gravel

$$\frac{0.5652}{0.8547} = 0.6613$$

Compute total composite curve number

$$0.0851(98) + 0.2536(98) + 0.6613(80) = 86$$

As is displayed by the reduction in the composite curve number, the removal of large areas of gravel significantly improved the overall permeability of the site. According to these calculations, a formal stormwater management plan beyond adequate grading to convey runoff away from the silo structures is not necessary. Curb cuts on the north side of the drive thru lane will allow water to drain to the nearby ditch, negating the necessity for an expensive underground conveyance system.

The client specified that Portland Cement Concrete (PCC) be used to pave the patio and drive-thru lane. With the understanding that both these surfaces will experience light loads, minimum pavement thickness for their respective loads and freeze/thaw resistance were selected to keep project costs affordable. The patio area will be paved with 4 inches of PCC on 6 inches of base course. Base course provides load distribution, drainage, and frost protection. A thicker layer helps to ensure longevity of the pavement. The drive-thru lane will be paved with 6 inches of PCC on 6 inches of base course. Both surfaces also include 8 inches of subbase to account for poorly draining soil and provide additional frost protection. To ensure that the site drains properly and complies with ADA standards, both paved areas will be graded with variable slopes between 1 and 4 percent.

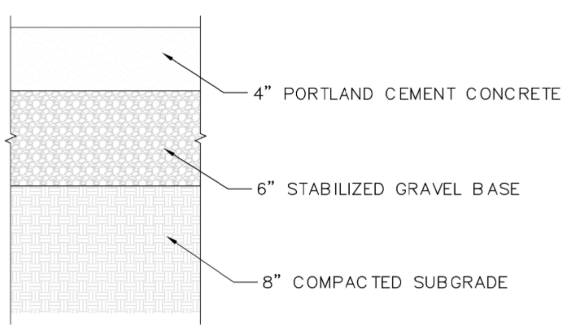


Figure 8: Patio Pavement Detail

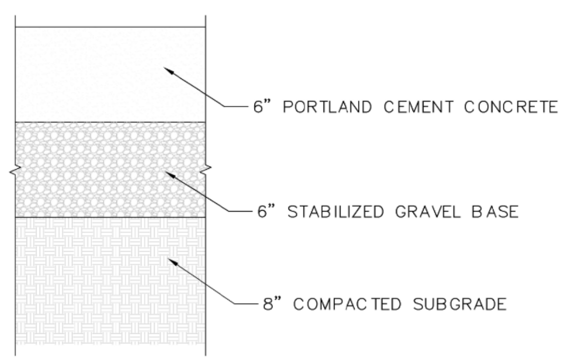


Figure 9: Drive Thru Pavement Detail

To improve site aesthetics and stabilize slopes in some areas, landscaping will be provided. The client requested that any planted greenery be low maintenance. It is also advantageous to plant native species for heartiness and a variety of species to avoid creating monoculture. The decided placement of plants will frame the patio and silo structures without blocking views from the main road. Three large Autumn Blaze Maples shall be planted for shade and a focal point. Near those, 2-4 Arborvitae will provide fullness to the planting area and a medium level to help transition the eye between the larger trees and low shrubs. Low shrub types shall include Boxwood and Ninebark for low maintenance cluster planting. In addition to planted landscaping, light poles will be installed around the patio edge, spaced approximately every 20 feet. The light poles shall be low to moderate lumen output for a subtle, integrated look.

Structural: Steel Framing

The member sizes for each level were designed for the specified loads located in the tables below:

Table 1: Minimum Required Live Loads (ASCE7-22)

Space	Live Loads per ASCE 7-22 (psf)
Bakery (Level 1)	150
Kitchen, other than domestic (Level 1)	150
Offices (Level 2)	100
Residential – All other areas except stairs (Level 4)	40
Residential – Sleeping areas (Level 5)	30

Table 2: Calculated Dead Loads

Material	Dead Loads (psf)
Tempered Glass Dome (1 in thick)	13
3VLI-36 (4.5 inches of concrete)	74.9
3VLI-36 (3.5 inches of concrete)	63.1
Mechanical, Electrical, Plumbing	4
Gypsum wall sheathing (1/2 inch)	2

Table 3: Calculated Snow and Wind Load

Category	Load (psf)
Snow	34.5
Wind	31.5

To optimize design, we took the worst-case scenario live load for each level and designed the floor framing for the entire floor accordingly. The worst-case scenario for the first level was the bakery & commercial kitchen, the second level is entirely office space, with the fourth and fifth level being solely residential space. The calculated snow and wind loads were strictly applied to the penthouse level framing, as these loads will not affect the framing of the other floors within the silo.

The entirety of the silo skeleton is comprised of steel framing. These silos require non-combustible framing due to the nature and risk of the structure. The structural floor framing consists of C-channels and wide flange beams. For the penthouse sleeping quarter edition, square HSS columns were used to structurally support the required snow, wind, and dead loads of the tempered glass dome. For smaller office & computer spaces, C-channels and wide flange beams were welded to plates that are anchored into the walls of the silos.

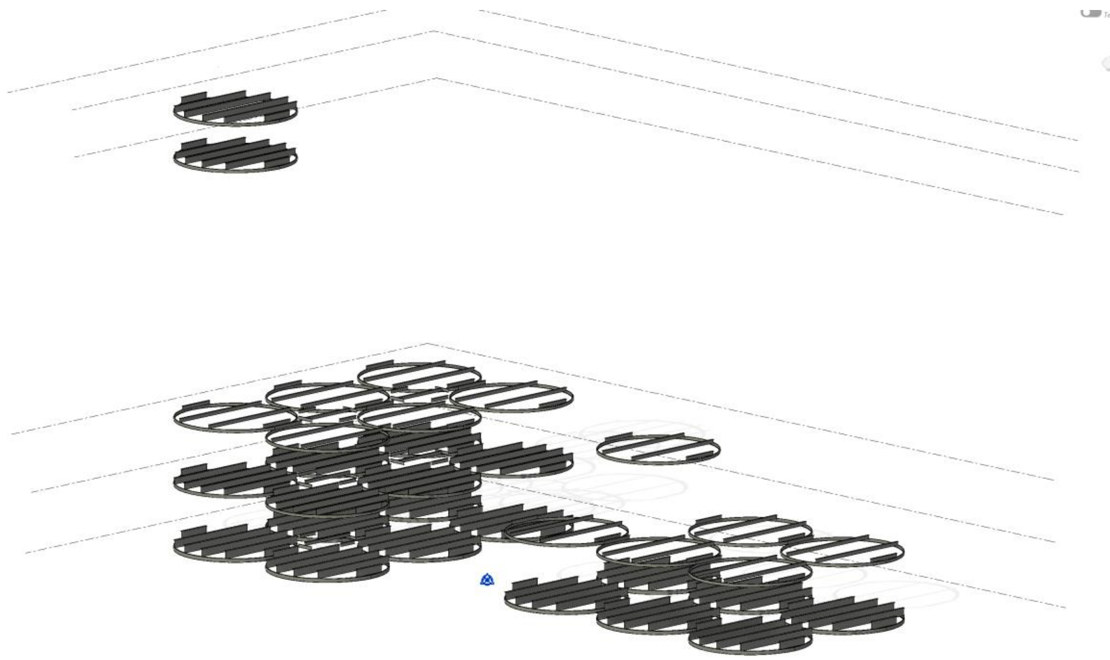


Figure 10: Structural Floor Framing Skeleton

The floor framing consists of a perimeter C-channel anchored into the silo wall, with wide flange beams spanning across the silo, bearing on top of and welded to the C-channel perimeter beam. The loads from each floor will carry from the steel framing to the silo walls, down to its foundation.

Level 1 (multi-purpose space) persists of a C15x33.9 channel as the perimeter beam, transferring the loads from the floor to the silo wall and down to the foundation. Six W30x108 beams were chosen to span the silo interior and bear on the C15x33.9 channel while also supporting the composite metal floor deck. For the small office spaces in-between the silos, we concluded that two C12x25 channels with three W12x40 beams are sufficient to support the composite deck and office loads acting on that space. Wide flange member sizes and composite decks were chosen due to their capacity to resist the deflection, moment and shear forces generated by the live, dead, wind and snow loads acting on each level. C-channels were analyzed for the same limits as the wide flange beams, with the addition of local flange bending from the wide flange beams bearing on the C-channels.

Level 2, 4, and 5 (conference space & penthouse suite) persist of a C12x30 channel acting as the perimeter beam, with six W24x76 beams spanning across the silo interior, bearing on the C12x30 channel while supporting the composite metal floor deck. The small conference rooms on Level 2 are framed identical to the small office space on Level 1.

The roof deck framing on levels 1 (multi-purpose) and 2 (conference) are framed with a C12x30 channel, with 4 C14x43 beams spanning across the interior, supporting the acoustic metal deck that acts as the roof of these levels. See Appendix C for all member structural calculations

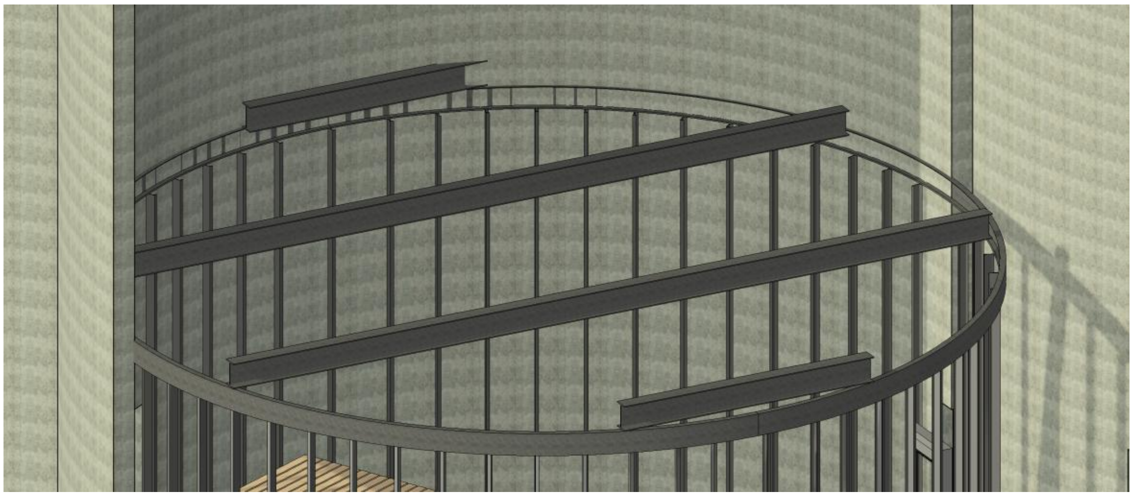


Figure 11: Perimeter C-Channel & Wide Flange Floor Framing

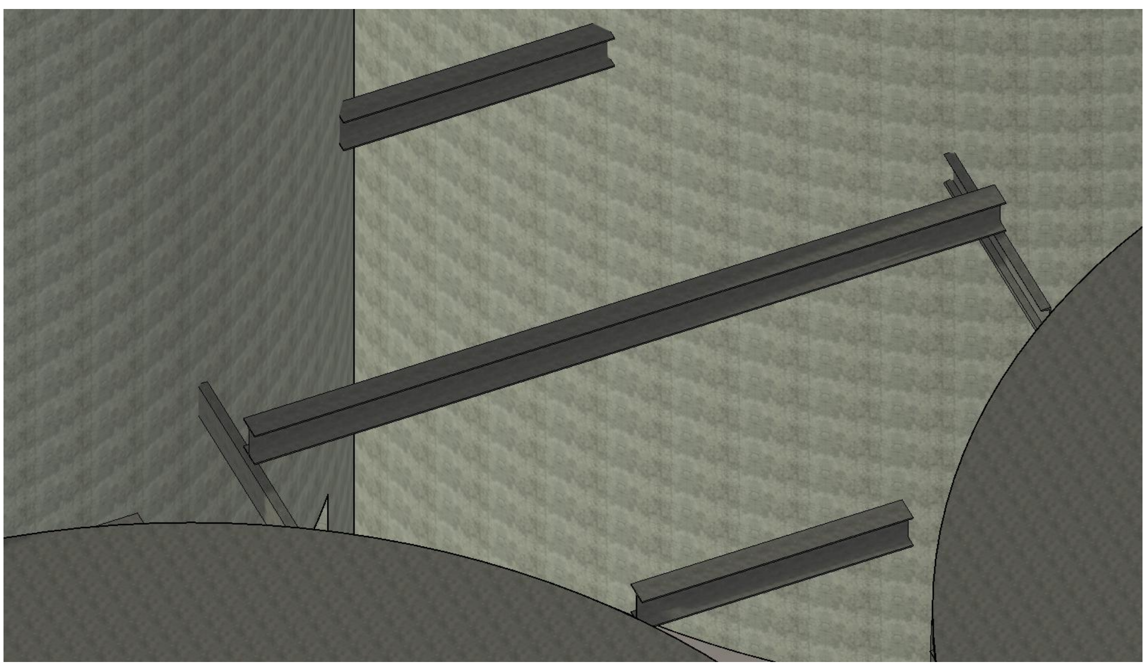


Figure 12: Small Office and Conference Room Framing

The wall framing of the penthouse sleeping quarters consists of square HSS columns, spaced 16” on-center to resist torsion caused by the wind force on the unsymmetric level. Square HSS was also chosen due to its axial load capacity, supporting the load of the glass dome which we calculated to be 13 psf, and the design snow load calculated to be 34.486 psf.

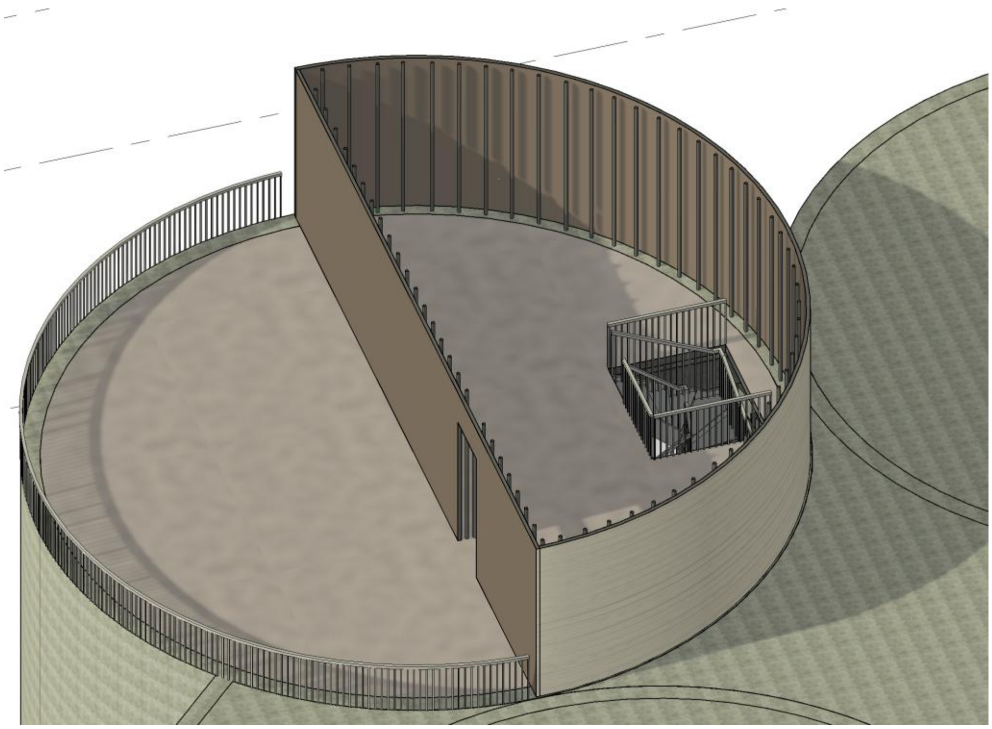


Figure 13: Sleeping Quarter Wall Framing

The final component to the steel framing is the reinforcement needed to support the larger openings created in the silo walls. These members had to support substantial loads while also fitting the natural curvature of the walls and supporting the forces caused by the loads from above. We used arching action theory in masonry walls to determine the load envelope the supports needed to withstand. And this led to the selection of two steel sections for the openings. Both the 16-foot and 15.5-foot openings are supported by HSS6x4x5/16, while the 10-foot openings will be supported by HSS6x4x1/4. The steel members will sufficiently support the loads from above and assist in supporting the large openings.

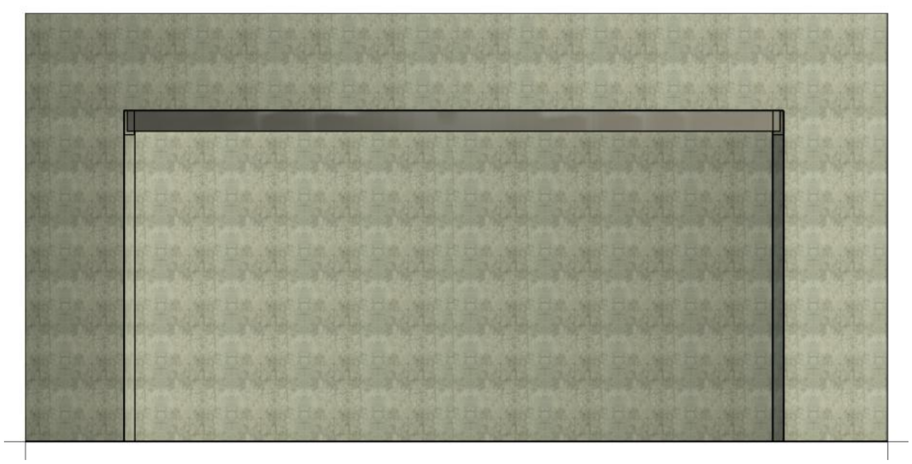


Figure 14: Large Opening Reinforcement

Table 4: Steel Framing Members

Member	Grade (ksi)
Wide Flange Beam	50
C-Channel	50
Square HSS	50
Rectangular HSS	50

All members were designed and chosen using grade 50 steel. Grade 50 steel for this framing allows us to reduce member size and self-weight compared to lower-grade steels.

Structural: Anchoring & Welding

The structural performance of the proposed framing system is fundamentally dependent on the integrity of its connection to the existing concrete silo shell. Proper anchoring and welding are critical to ensure that all applied loads are safely and efficiently transferred into the silo wall. Anchorage into the concrete shell was designed to develop sufficient capacity in moment, shear, deflection, and combined loading conditions, while also accounting for the existing material properties and condition of the silo structure. Similarly, welding of steel components must meet applicable standards to ensure continuity, strength, and durability of the connections.

The anchoring of the C-channel to the silo wall varies by floor depending on the applied live and dead loads per level. In level 1, we decided to fasten the C-channel with 2 anchors, spaced 42 inches on-center, with 3 anchors fastened directly where the wide-flange beam bears on the channel, where shear and moment will be highest.

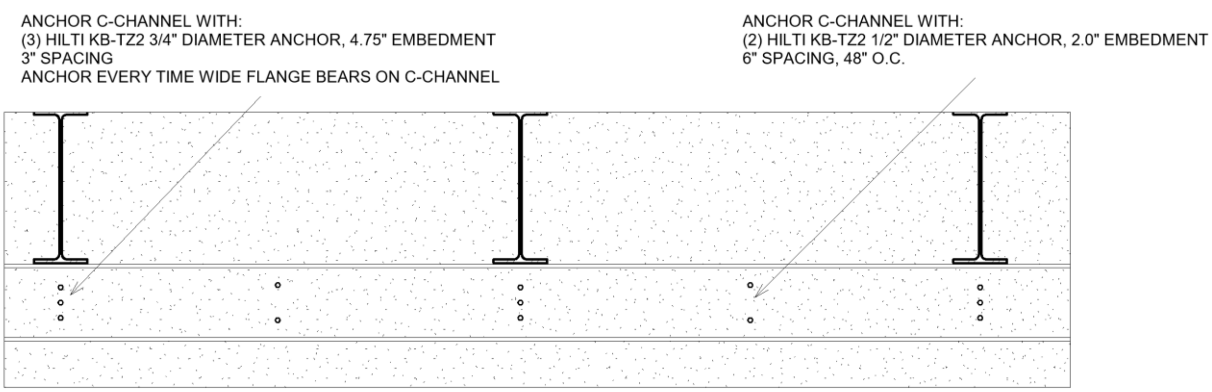


Figure 15: Level 1 Anchor Detail

On Level 2, we fasten the C channel with 2 anchors, spaced 48 inches on-center, again fastening 3 anchors into the C-channel where the beam bears directly onto the channel.

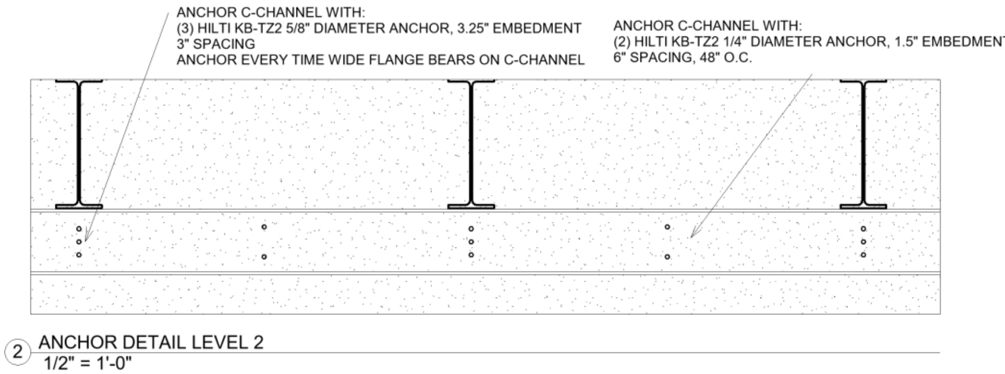


Figure 16: Level 2 Anchor Detail

On Level 3, we don't require as much anchoring due to the fact it is solely supporting the mechanical ducts, self-weight of the beams, acoustic roof deck, insulation, and gypsum roof sheathing. Anchors are spaced 48 inches on-center, while two anchors are fastened where beam bears on C-channel, instead of three for floor framing.

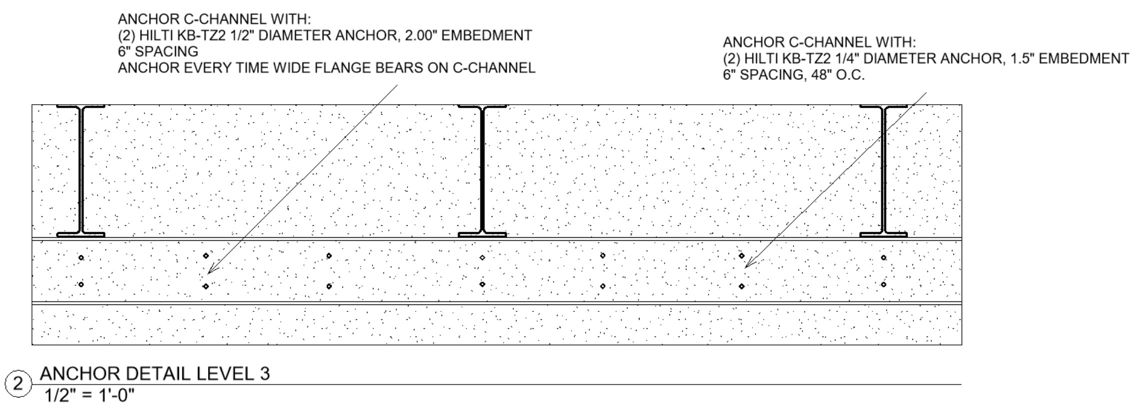


Figure 17: Level 3 Anchor Detail

Level 4 does not consist of as much live load, therefore the anchor fastening is identical to Level 3. Anchors are spaced 48 inches on-center with two anchors fastened every time the wide flange beam bears on the C-channel.

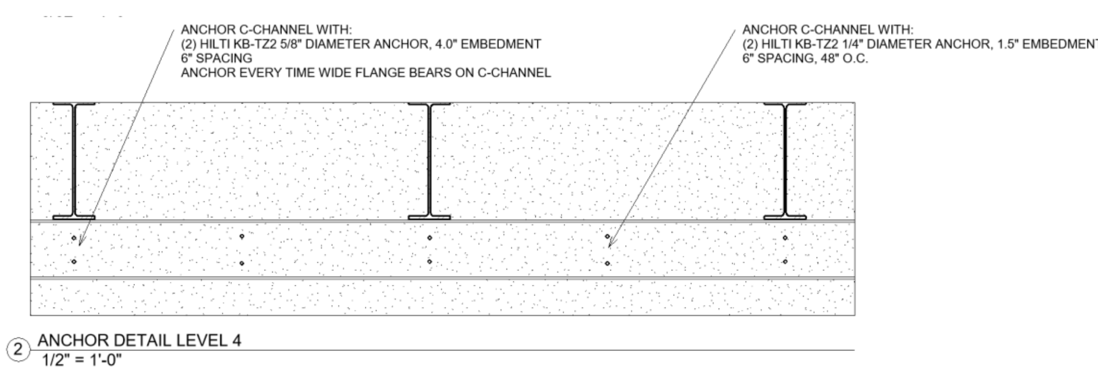


Figure 18: Level 4 Anchor Detail

Due to Level 5 transferring snow, wind, dead, and live loads that are applied to the sleeping quarter level, we decided to fasten the C-channel with 2 anchors, 42 inches on-center, with three anchors fastened where the wide flange beam bears on the C-channel.

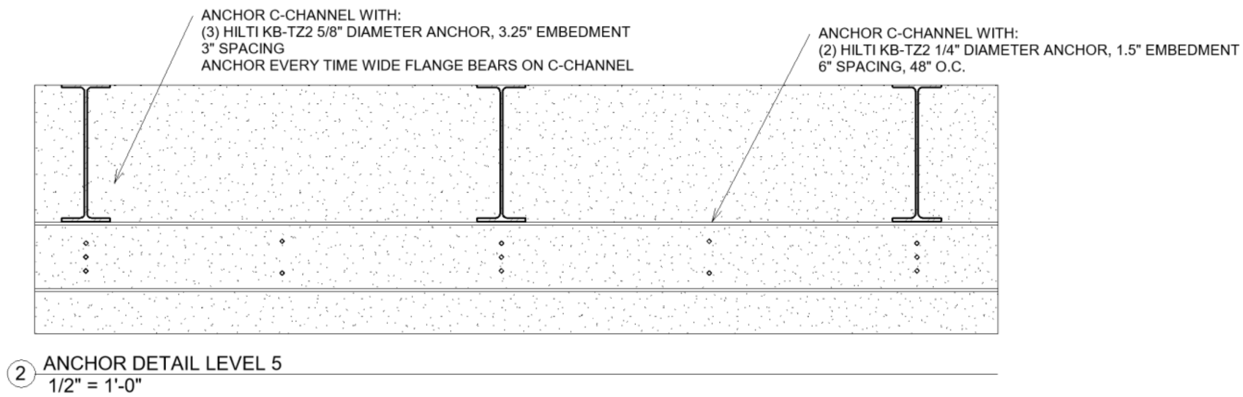


Figure 19: Level 5 Anchor Detail

For the opening reinforcement, the steel plates help to connect the steel section with existing concrete walls, so the system acts as one. To achieve this the plates are bolted to the wall with 6 3/8-in diameter Hilti Carbon Steel anchors. For the two larger opening sizes the anchors are embedded 2-in, while the smaller opening has and embedment depth of 1.5-in. This will allow for the steel beam to transfer loads back to the existing walls.

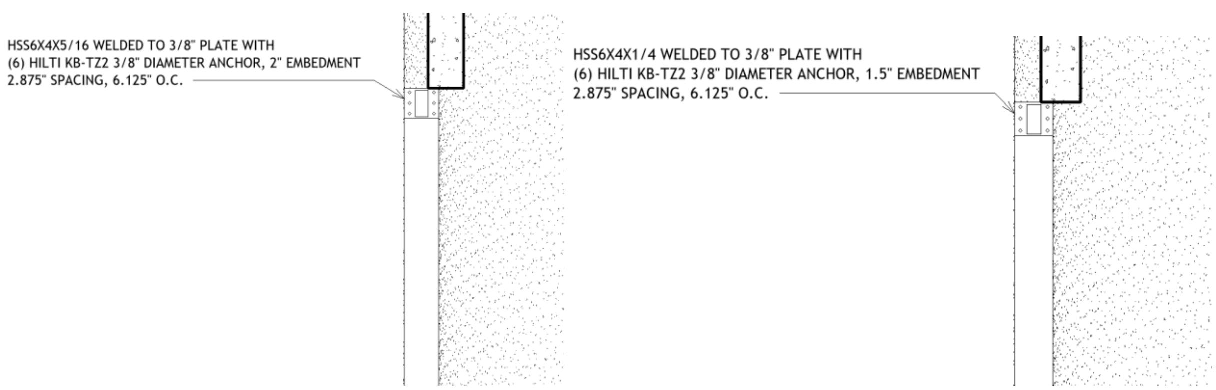


Figure 20: Large Opening Anchor Detail

Anchor capacity was designed according to AISC (Chapter 7) and Hilti Anchor Design Manual. Anchors were checked for failure both at their 42–48-inch span length and where wide flange beams bear on the C-channel. See Appendix C for detailed anchor structural calculations and structural plan sheets for locations and sections.

Welding detail and plates were designed in accordance to AISC 16th Edition (Chapter 8 & 14). The small conference rooms & level 1 computer spaces require framing that connects beams to plates through sufficient welding practice, with the plates being bolted into the exterior concrete wall. Plates & welds were designed to withstand the forces the beams experience due to the dead and live loads of these spaces. These spaces all consist of the same loads, therefore the detail for level 1, 2, and 3 are identical in design. The wide flange plates include a thickness of 1/8-inch and a 1/16-inch flange weld with a two-bolt flush pattern anchoring the plate to the silo wall. The C-channel plates include a 3/8-inch thickness with a 1/8-inch flange weld thickness, with the same two-bolt flush anchor pattern fastening the plate to the silo wall. Finally, for the HSS column framing in the sleeping quarters, a 1/8-inch-thick plate and 1/16-inch flange weld thickness. See Appendix C for detailed weld calculations and structural plan sheets for locations and sections.

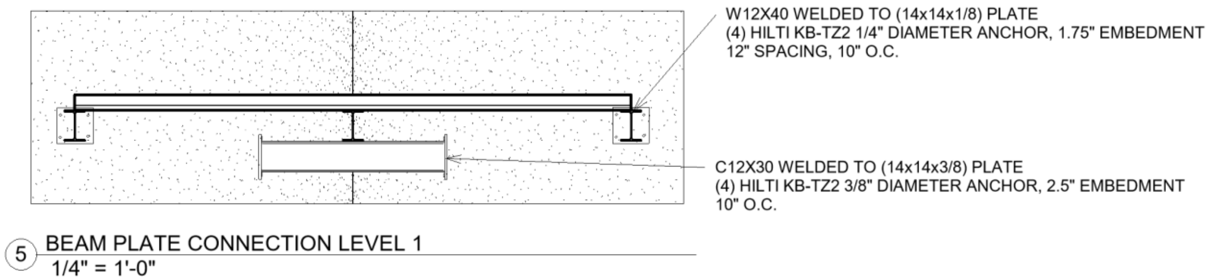


Figure 21: Level 1, 2, and 3 Plate and Weld Detail

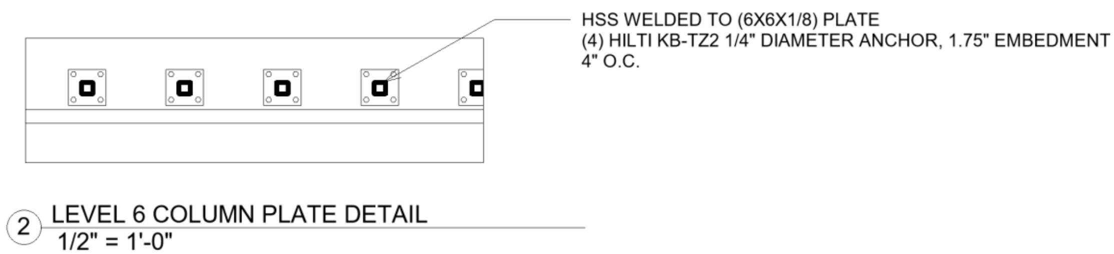


Figure 22: Level 6 Column Plate and Weld Detail

The plates for the opening were designed to connect the steel beam with the existing silo walls to create a cohesive reinforcement system. The 16 and 15.5-foot openings will be connected by 6x7x3/8-in plates at both ends. The steel beam is then welded to the plates with a 5/16-in weld around the sides and bottom of the beam. For the 10-foot opening the plates are 6x7x1/4-in and will be connected to the member by 3/16-in welds around the sides and bottom of the member.

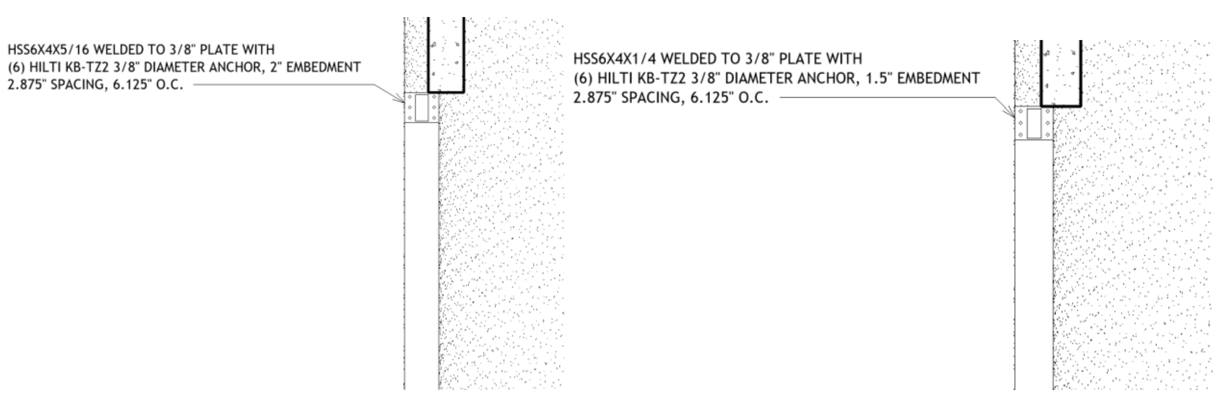


Figure 23: Large Opening Plate and Weld Detail

Foundation and Shear Wall Design

To meet the requirement for additional exterior fire escape stairs, continuous footings were designed to prevent overturning and structural failure of the stairway walls. Based on the governing snow, wind, and dead loads, a 2.5 ft by 1 ft footing was selected to resist the applied loads. The footing is placed below the frost line at a depth of 4 ft 2 in and is reinforced with five #6 longitudinal bars spaced at 6 in, along with #6 transverse reinforcement at 6 in on center to provide adequate flexural capacity.

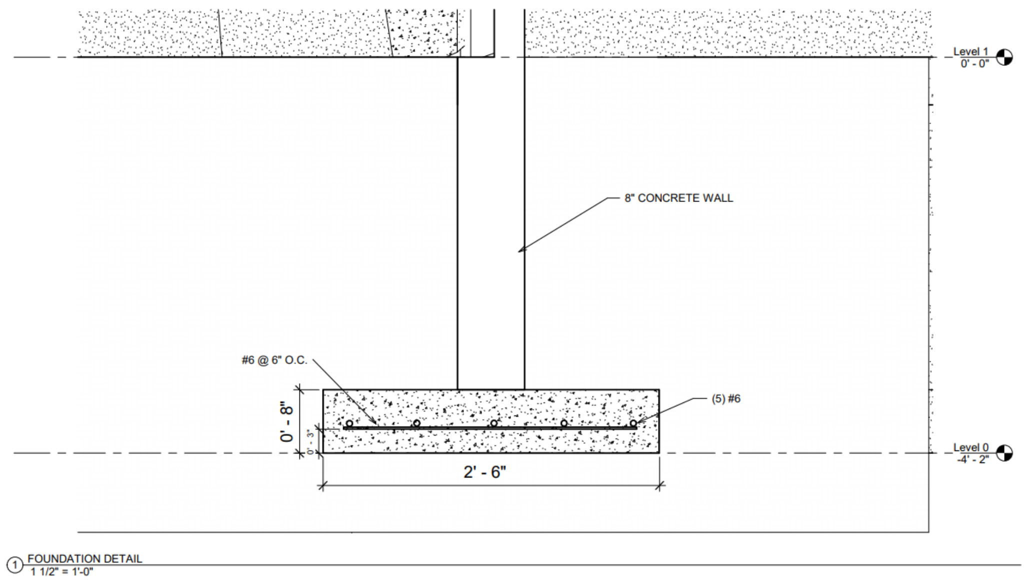


Figure 24: Foundation Detail

The shear wall located on the south side of the penthouse sleeping quarters was designed to resist shear, moment, and drift induced by wind and snow loads. A Simpson Strong-Tie SSW12x10 steel shear wall was selected as the primary lateral force-resisting system at this level.

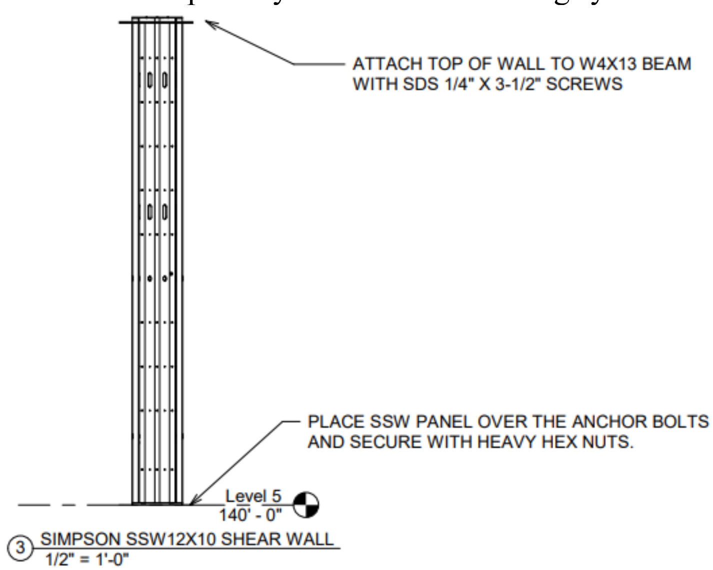


Figure 25: Steel Shear Wall Detail

<u>Section VII</u> Construction Cost

1. **General Instructions**

This section presents the total cost for providing construction services and materials for the project. Costs are based on the scope of work and reflect a combination of labor, materials, equipment, and associated project expenses. All costs are expressed in U.S. dollars and are based on a time-and-materials approach for labor, combined with estimated quantities and unit costs for materials. Labor costs include applicable overhead and profit.

Table 5: Construction Cost Estimate

Items	Cost	
Demolition	$	345,812.18
Site Development	$	200,350.93
Structural/Framing Steel	$	1,656,395.38
Concrete	$	109,670.51
Floor/Roof System	$	299,410.60
Finishes	$	712,070.56
Construction Cost	$	3,323,710.16
Contingency (15%)	$	498,556.52
Administration Fee (10%)	$	332,371.02
Total Project Cost	$	4,154,637.70

The construction cost was developed using a time-and-materials approach based on the estimated level of effort and materials required to complete the project scope.

An average direct labor billing rate of **$35/hour** was used to calculate base labor costs. A fixed multiplier of **2.75** was then applied to account for overhead, indirect expenses, and profit, consistent with industry-standard practice.

Based on these calculations, the total proposed design services cost for the project is **$4,154,637.70**

2. **Cost Proposal Form**

Hourly rates shall represent direct labor costs only and shall be applied consistently across all tasks unless otherwise justified. The multiplier for overhead and profit shall be applied uniformly to all labor costs. This multiplier shall also account for indirect costs and profit. Labor subtotals shall be calculated as the sum of all task totals. Travel, materials, and supplies shall be listed as a separate line item and added to the labor subtotal.

The total cost shall reflect the sum of the labor subtotal and reimbursable expenses. This amount shall represent the full compensation of the design services described in the proposal. We are responsible for ensuring that all calculations are accurate and that all required fields in the cost form are fully completed.

Section VII Proposal Attachments

Structural Calculations: See Appendix C

Section VIII Appendices

Appendix A – Cost Form

UNIVERSITY OF IOWA
DEPARTMENT OF CIVIL & ENVIRONMENTAL ENGINEERING
Project Design & Management
(CEE: 4850:0001)

Final Report # 02-Spring2026

Grain Silo Adaptive Reuse
Cost Form

Budget Summary

Task	Hours	Hourly Salary[a]	Multiplier for Overhead and Profit[b]	Total	
Project Management	100	$35	x2.75	$9.625.00	
Existing Conditions Review	95	$35	x2.75	$9,143.75	
Structural Review	140	$35	x2.75	$13,475.00	
Conceptual Design	210	$35	x2.75	$20.212.50	
Meeting Times	40	$35	x2.75	$3,850.00	
Documentation/ Visuals	50	$35	x2.75	$4,812.50	
				$61,118.75	Sub-total
				$1,000	Travel, Materials, and Supplies
				$62,118.75	Total Cost

[a]Direct Costs are broadly defined as any cost that can be accurately assigned to a specific task, such as wages, fringe benefits, materials, and supplies.

[b]Indirect Costs include overhead costs associated with maintaining a design firm that cannot be accurately attributed to given tasks and profit margins.

Project Design & Management (CEE: 4850:0001) - RFP is for educational purposes only

Appendix B

Contact
Tiffany Luing, Economic Development Coordinator tluing@cityofbondurant.com

Location
Bondurant, IA

Work Products:

The Grain Silos project scope includes evaluation and conceptual design of adaptive reuse options for the ground floors of two six-pack silo structures. The design shall consider structural capacity, code requirements, access, circulation, and potential uses of the ground-level spaces. Integration with surrounding redevelopment and potential future hotel development shall be examined. The design shall support feasibility assessment and incremental redevelopment planning.

Phasing Plan and Cost Estimate:
A phasing plan shall outline potential staged redevelopment approaches for the silo structures. A cost estimate shall be prepared using conceptual quantities and unit costs from the Iowa Public Works Service Bureau bid cost tables, supplemented as necessary for building-related work, and summarized by redevelopment phase.

Level 1 Framing

Floor Framing Calculations **W30x108**

***6 beams, spaced at 7.5 feet O.C. covered by composite metal corrugated deck**

$E := 29000\ ksi \qquad F_y := 50\ ksi \qquad I := 4470\ in^4 \qquad Z_x := 346\ in^3$

Tributary Width $w_T := 7\ ft + 8\ in$

$SW_{beam} := 108\ \frac{lbf}{ft}$

$LL := 150\ psf$ Bakery & Commercial Kitchen Live Load
$DL_{MEP} := 4\ psf$ - ASCE 7 (Worst case scenario)

$w_{conc\&deck} := 74.9\ psf$ Vulcraft 3VLI-36 Composite Deck Slab
19 Gage, 7.5in slab depth

Uniform Member Loads

$$w_L := w_T \cdot LL = 1.15\ \frac{kip}{ft}$$

$$w_D := w_T \cdot \left(DL_{MEP} + w_{conc\&deck}\right) = 0.605\ \frac{kip}{ft}$$

$$w_u := 1.2\ w_D + 1.2 \cdot \left(SW_{beam}\right) + 1.6\ w_L = 2.695\ \frac{kip}{ft}$$ **Ultimate Factored Load**

$L_{girder} := 40\ ft$ *Worst case scenario*

Deflection Check

$$\Delta_{max} := \frac{5\ w_u \cdot L_{girder}^{\ 4}}{384 \cdot E \cdot I} = 1.198\ in \qquad \Delta_{maxAll} := \frac{L_{girder}}{360} = 1.333\ in$$

$$\frac{\Delta_{max}}{\Delta_{maxAll}} = 0.898$$ *Good*

Moment Check

Design Moment $\phi M_n := 0.9 \cdot F_y \cdot Z_x = \left(1.298 \cdot 10^3\right)\ kip \cdot ft$

$$M_{max} := \frac{w_u \cdot L_{girder}^{\ 2}}{8} = 539.096\ kip \cdot ft \qquad \frac{M_{max}}{\phi M_n} = 0.415$$ *Good*

Shear Check

$\phi V_n := 487\ \boldsymbol{kip}$ ASIC Table 3-6

$$V_{max} := \frac{w_u \cdot L_{girder}}{2} = 53.91\ \boldsymbol{kip} \qquad \frac{V_{max}}{\phi V_n} = 0.111 \qquad \textit{Good}$$

Perimeter C-Channel Ring Calculations C15x33.9

Tributary Width $w_T := 7.5\ \boldsymbol{ft}$ $I := 315\ \boldsymbol{in}^4$ $Z_x := 50.8\ \boldsymbol{in}^3$

$SW_{beam} := 33.9\ \dfrac{\boldsymbol{lbf}}{\boldsymbol{ft}}$ $LL := 150\ \boldsymbol{psf}$ $DL_{MEP} := 4\ \boldsymbol{psf}$ Kitchen&Bakery Live Load - ASCE 7

$w_{conc\&deck} := 74.9\ \boldsymbol{psf}$ 3VLI-36 Composite Deck Slab 19 Gage, 7.5in slab depth

$$w_{stud} := \frac{(6.7\ \boldsymbol{plf} \cdot 16.5\ \boldsymbol{ft})}{24\ \boldsymbol{in}} = 55.275\ \boldsymbol{plf} \qquad w_{gyp} := 4\ \boldsymbol{psf} \cdot 16.5\ \boldsymbol{ft} = 66\ \boldsymbol{plf} \qquad \text{1 in board}$$

Uniform Member Loads

$$w_L := w_T \cdot LL = 1.125\ \frac{\boldsymbol{kip}}{\boldsymbol{ft}}$$

$$w_D := w_T \cdot \left(DL_{MEP} + w_{conc\&deck}\right) = 0.592\ \frac{\boldsymbol{kip}}{\boldsymbol{ft}}$$

$$w_u := 1.2\ w_D + 1.2 \cdot \left(SW_{beam} + w_{stud} + w_{gyp}\right) + 1.6\ w_L = 2.696\ \frac{\boldsymbol{kip}}{\boldsymbol{ft}}$$ **Ultimate Factored Load**

$L_{ring} := 35\ \boldsymbol{ft} + 5\ \boldsymbol{in}$ *Worst case scenario*

Bolt Shear Failure Check $s_{bolt} := 3\ \boldsymbol{in}$ $s_{OC} := 42\ \boldsymbol{in}$ $d_b := \dfrac{1}{2}\ \boldsymbol{in} = 0.5\ \boldsymbol{in}$ $A_b := \dfrac{\pi \cdot d_b^{\ 2}}{4}$

$$P_u := \frac{(w_u \cdot s_{OC})}{2 \cdot 2} = 2.359\ \boldsymbol{kip}$$ Shear in one bolt (if 2 bolts per 48 in)

$\phi V_n := 2630\ \boldsymbol{lbf}$

KB-TZ2 - Hilti Anchor Fastening Design Manual
1/2" diameter, 2" embedment, 4000 psi concrete

$$\frac{P_u}{\phi V_n} = 0.897 \qquad \textit{Good}$$

Nominal anchor diameter in.	Effective embedment in. (mm)	Nominal embedment in. (mm)	Tension (lesser of concrete breakout / pullout) - ΦN_n				Shear (lesser of concrete breakout or pryout) - ΦV_n			
			f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.1 MPa) lb (kN)	f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.1 MPa) lb (kN)
1/4	1-1/2 (38)	1 3/4 (44)	280 (1.2)	300 (1.3)	340 (1.5)	395 (1.8)	1,095 (4.9)	1,195 (5.3)	1,385 (6.2)	1,695 (7.5)
3/8	1-1/2 (38)	1 7/8 (48)	1,255 (5.6)	1,375 (6.1)	1,585 (7.1)	1,940 (8.6)	1,350 (6.0)	1,480 (6.6)	1,710 (7.6)	2,090 (9.3)
	2 (51)	2 1/2 (64)	1,930 (8.6)	2,115 (9.4)	2,440 (10.9)	2,990 (13.3)	2,080 (9.3)	2,275 (10.1)	2,630 (11.7)	3,220 (14.3)
	2-1/2 (64)	3 (76)	2,185 (9.7)	2,390 (10.6)	2,765 (12.3)	3,385 (15.1)	4,705 (20.9)	5,155 (22.9)	5,950 (26.5)	7,285 (32.4)
1/2	1-1/2 (38)	2 (51)	1,435 (6.4)	1,570 (7.0)	1,815 (8.1)	2,220 (9.9)	1,545 (6.9)	1,690 (7.5)	1,950 (8.7)	2,390 (10.6)
	2 (51)	2 1/2 (64)	1,930 (8.6)	2,115 (9.4)	2,440 (10.9)	2,990 (13.3)	2,080 (9.3)	2,275 (10.1)	2,630 (11.7)	3,220 (14.3)
	2-1/2 (64)	3 (76)	2,700 (12.0)	2,955 (13.1)	3,415 (15.2)	4,180 (18.6)	5,810 (25.8)	6,365 (28.3)	7,350 (32.7)	9,000 (40.0)

(83)	(95)	(14.4)	(15.8)	(18.2)	(22.3)	(31.0)	(34.0)	(39.2)	(48.0)

Connected Member Shear Failure Check

$\phi V_n := 211\ kip$ AISC Table 3-8

$$V_n := \frac{(w_u \cdot s_{OC})}{2} = 4.719\ kip \qquad \frac{V_n}{\phi V_n} = 0.022 \qquad Good$$

Flange Local Bending Check From Beam Point Load

$t_f := 0.650\ in$ $\quad F_{yf} := 50\ ksi$

$R_n := 6.25 \cdot F_{yf} \cdot t_f^{\,2} = 132.031\ kip$ $\quad P_u := V_{max} = 53.91\ kip$ *Ultimate Load From Beam*

$P_{fb} := 0.9 \cdot R_n = 118.828\ kip$ *Design Strength*

$$\frac{P_u}{P_{fb}} = 0.454 \qquad Good$$

Bolt Shear Failure From Beam Point Load

$P_u := V_{max} = 53.91\ kip$ three bolts $\quad P_u := \frac{P_u}{3} = 17.97\ kip$

3/4	3-1/4 (83)	4 (102)	4,000 (17.8)	4,380 (19.5)	5,060 (22.5)	6,195 (27.6)	8,615 (38.3)	9,435 (42.0)	10,895 (48.5)	13,345 (59.4)
	3-3/4 (95)	4 1/2 (114)	4,955 (22.0)	5,430 (24.2)	6,270 (27.9)	7,680 (34.2)	10,675 (47.5)	11,695 (52.0)	13,505 (60.1)	16,540 (73.6)
	4-3/4 (121)	5 1/2 (140)	5,745 (25.6)	6,055 (26.9)	6,580 (29.3)	7,405 (32.9)	15,220 (67.7)	16,670 (74.2)	19,250 (85.6)	23,575 (104.9)

$\phi V_n := 19250\ lbf$

$$\frac{P_u}{\phi V_n} = 0.933 \qquad Good$$

KB-TZ2 - Hilti Anchor Fastening Design Manual
3/4" diameter, 4.75" embedment, 4000 psi concrete

Deflection Check

$$\Delta_{max} := \frac{w_u \cdot s_{OC}^{\,4}}{384 \cdot E \cdot I} = \left(1.993 \cdot 10^{-4}\right)\ in \qquad \Delta_{maxAll} := \frac{s_{OC}}{360} = 0.117\ in$$

$$\frac{\Delta_{max}}{\Delta_{maxAll}} = 1.708 \cdot 10^{-3} \qquad Good$$

Moment Check

$$M_{max} := \frac{w_u \cdot s_{OC}^{2}}{12} = 2.752\ \boldsymbol{kip \cdot ft}$$

$$\phi M_p := 0.9 \cdot F_y \cdot Z_x = 190.5\ \boldsymbol{kip \cdot ft}$$

$$\frac{M_{max}}{\phi M_p} = 0.014 \qquad Good$$

Design of Concrete Slab

Moment Check

$$\phi M_{no} := 10.95\ \boldsymbol{kip} \cdot \frac{\boldsymbol{ft}}{\boldsymbol{ft}}$$

Vulcraft 3VLI-36 Composite Deck Slab
19 Gage, 7.5in slab depth

$$w_{conc\&deck} := 74.9\ \boldsymbol{psf}$$

$$LL := 150\ \boldsymbol{psf}$$
$$DL_{MEP} := 4\ \boldsymbol{psf}$$

Bakery & Commercial Kitchen Live Load
- ASCE 7 (Worst case scenario)

$$w_u := \left(1.2\ \left(DL_{MEP} + w_{conc\&deck}\right) + 1.6\ LL\right) \cdot 1\ \boldsymbol{ft} = 334.68\ \boldsymbol{plf}$$

$$l := 7\ \boldsymbol{ft} + 8\ \boldsymbol{in}$$

$$M_u := \frac{\left(w_u \cdot l^2\right)}{8} = 2.459\ \boldsymbol{kip \cdot ft}$$

Deflection Check

$$I := 26.87\ \boldsymbol{in}^4$$

$$E := 33 \cdot 145^{1.5} \cdot \sqrt[2]{4000} \cdot \boldsymbol{psi} = \left(3.644 \cdot 10^3\right)\ \boldsymbol{ksi}$$

$$\Delta_{max} := \frac{5\ w_u \cdot l^4}{384 \cdot E \cdot I} = 0.266\ \boldsymbol{in}$$

$$\Delta_{maxAll} := \frac{l}{360} = 0.256\ \boldsymbol{in}$$

$$\frac{\Delta_{max}}{\Delta_{maxAll}} = 1.04 \qquad Good$$

Shear Check

$$\phi V_{no} := 9.46\ \frac{\boldsymbol{kip}}{\boldsymbol{ft}}$$

$$V_u := \frac{w_u \cdot l}{2} \cdot \frac{1}{\boldsymbol{ft}} = 1.283\ \frac{\boldsymbol{kip}}{\boldsymbol{ft}}$$

$$\frac{V_u}{} = 0.136 \qquad Good$$

ϕV_{no}

Welded Wire Reinforcement

Recommended for Vulcraft 3VLI-36 Composite Deck Slab
19 Gage, 7.5in slab depth

Mini Office Structural Calcs

W12x40

***3 beams, spaced at 9 feet O.C. covered by composite metal corrugated deck**

$E := 29000\ ksi$ $\quad F_y := 50\ ksi$ $\quad I := 307\ in^4$ $\quad Z_x := 57\ in^3$

Tributary Width $w_T := 9\ ft$

$SW_{beam} := 40\ \dfrac{lbf}{ft}$ $\quad LL := 50\ psf$ Office Live Load - ASCE 7

$DL_{MEP} := 4\ psf$

$w_{conc\&deck} := 63.1\ psf$ Vulcraft 3VLI-36 Composite Deck Slab
18 Gage, 6.5in slab depth

Uniform Member Loads

$$w_L := w_T \cdot LL = 0.45\ \frac{kip}{ft}$$

$$w_D := w_T \cdot \left(DL_{MEP} + w_{conc\&deck}\right) = 0.604\ \frac{kip}{ft}$$

$$w_u := 1.2\ w_D + 1.2 \cdot \left(SW_{beam}\right) + 1.6\ w_L = 1.493\ \frac{kip}{ft}$$ **Ultimate Factored Load**

$L_{girder} := 20\ ft$ *Worst case scenario*

Deflection Check

$$\Delta_{max} := \frac{5\ w_u \cdot L_{girder}^{\ 4}}{384 \cdot E \cdot I} = 0.604\ in \qquad \Delta_{maxAll} := \frac{L_{girder}}{360} = 0.667\ in$$

$$\frac{\Delta_{max}}{\Delta_{maxAll}} = 0.905$$ *Good*

Moment Check

Design Moment $\phi M_n := 0.9 \cdot F_y \cdot Z_x = 213.75\ kip \cdot ft$

$M_{max} := \frac{w_u \cdot L_{girder}^2}{8} = 74.634\ kip \cdot ft$ $\frac{M_{max}}{\phi M_n} = 0.349$ *Good*

Shear Check

$\phi V_n := 487\ kip$ ASIC Table 3-6

$V_{max} := \frac{w_u \cdot L_{girder}}{2} = 14.927\ kip$ $\frac{V_{max}}{\phi V_n} = 0.031$ *Good*

C-Channel Calculations

C12x25

Tributary Width $w_T := 9\ ft$ $I := 144\ in^4$ $Z_x := 29.4\ in^3$

$SW_{beam} := 25\ \frac{lbf}{ft}$ $LL := 50\ psf$ $DL_{MEP} := 4\ psf$ Kitchen&Bakery Live Load - ASCE 7

$w_{conc\&deck} := 63.1\ psf$ Vulcraft 3VLI-36 Composite Deck Slab 18 Gage, 6.5in slab depth

Uniform Member Loads

$w_L := w_T \cdot LL = 0.45\ \frac{kip}{ft}$

$w_D := w_T \cdot (DL_{MEP} + w_{conc\&deck}) = 0.604\ \frac{kip}{ft}$

$w_u := 1.2\ w_D + 1.2 \cdot (SW_{beam}) + 1.6\ w_L = 1.475\ \frac{kip}{ft}$ **Ultimate Factored Load**

$L_{ring} := 6\ ft$ *Worst case scenario*

Connected Member Shear Failure Check

$\phi V_n := 125\ kip$ AISC Table 3-8

$V_n := \frac{V_{max}}{2} = 7.463\ kip$ $\frac{V_n}{\phi V_n} = 0.06$ *Good*

Flange Local Bending Check From Beam Point Load

$t_f := 0.501\ in$ $F_{yf} := 50\ ksi$

$R_n := 6.25 \cdot F_{yf} \cdot t_f^2 = 78.438\ kip$ $P_u := V_{max} = 14.927\ kip$ *Ultimate Load From Beam*

$P_{fb} := 0.9 \cdot R_n = 70.594\ kip$ *Design Strength*

$$\frac{P_u}{P_{fb}} = 0.211 \qquad Good$$

Deflection Check

$$\Delta_{max} := \frac{w_u \cdot s_{OC}^4}{384 \cdot E \cdot I} = \left(2.385 \cdot 10^{-4}\right) \, in \qquad \Delta_{maxAll} := \frac{s_{OC}}{360} = 0.117 \, in$$

$$\frac{\Delta_{max}}{\Delta_{maxAll}} = 2.044 \cdot 10^{-3} \qquad Good$$

Moment Check

$$M_{max} := \frac{w_u \cdot s_{OC}^2}{12} = 1.505 \, kip \cdot ft$$

$$\phi M_p := 0.9 \cdot F_y \cdot Z_x = 110.25 \, kip \cdot ft$$

$$\frac{M_{max}}{\phi M_p} = 0.014 \qquad Good$$

Design of Concrete Slab

Moment Check

$$\phi M_{no} := 10.41 \, kip \cdot \frac{ft}{ft}$$

Vulcraft 3VLI-36 Composite Deck Slab
18 Gage, 6.5in slab depth

$$w_{conc\&deck} := 63.1 \, psf$$

$LL := 50 \, psf$
$DL_{MEP} := 4 \, psf$

Bakery & Commercial Kitchen Live Load
- ASCE 7 (Worst case scenario)

$$w_u := \left(1.2 \left(DL_{MEP} + w_{conc\&deck}\right) + 1.6 \, LL\right) \cdot 1 \, ft = 160.52 \, plf \qquad l := 7.5 \, ft$$

$$M_u := \frac{\left(w_u \cdot l^2\right)}{8} = 1.129 \, kip \cdot ft \qquad \frac{M_u}{\phi M_{no}} \cdot \frac{1}{ft} = 0.108$$

Deflection Check

$$I := 26.87 \, in^4 \qquad E := 33 \cdot 145^{1.5} \cdot \sqrt[2]{4000 \cdot psi} = \left(3.644 \cdot 10^3\right) \, ksi$$

$$\Delta_{max} := \frac{5 \, w_u \cdot l^4}{384 \cdot E \cdot I} = 0.117 \, in \qquad \Delta_{maxAll} := \frac{l}{360} = 0.25 \, in$$

$$\frac{\Delta_{max}}{\Delta_{maxAll}} = 0.467 \qquad Good$$

Shear Check

$\boxed{\phi V_{no}} := 9.46 \ \dfrac{kip}{ft}$

$\boxed{V_u} := \dfrac{w_u \cdot l}{2} \cdot \dfrac{1}{ft} = 0.602 \ \dfrac{kip}{ft}$

$\dfrac{V_u}{\phi V_{no}} = 0.064$ $\quad$ *Good*

Welded Wire Reinforcement

6x6-W2.1xW2.1 Recommended for Vulcraft 3VLI-36 Composite Deck Slab
18 Gage, 6.5in slab depth

Level 2 (Conference Floor)

Floor Framing Calculations **W24x76**
***6 beams, spaced at 7.5 feet O.C. covered by composite metal corrugated deck**

$E := 29000\ ksi$ $F_y := 50\ ksi$ $I := 2100\ in^4$ $Z_x := 200\ in^3$

Tributary Width $w_T := 7\ ft + 8\ in$

$LL := 50\ psf$ Office Live Load - ASCE 7

$SW_{beam} := 76\ \frac{lbf}{ft}$ $DL_{MEP} := 4\ psf$

$w_{conc\&deck} := 63.1\ psf$ Vulcraft - Design Manual - 18 Gage 3VLI Composite Deck
6.5" Slab Thickness

Uniform Member Loads

$$w_L := w_T \cdot LL = 0.383\ \frac{kip}{ft}$$

$$w_D := w_T \cdot \left(DL_{MEP} + w_{conc\&deck}\right) = 0.514\ \frac{kip}{ft}$$

$$w_u := 1.2\ w_D + 1.2 \cdot \left(SW_{beam}\right) + 1.6\ w_L = 1.322\ \frac{kip}{ft}$$ **Ultimate Factored Load**

$L_{girder} := 40\ ft$ *Worst case scenario*

Deflection Check

$$\Delta_{max} := \frac{5\ w_u \cdot L_{girder}^{\ 4}}{384 \cdot E \cdot I} = 1.25\ in \qquad \Delta_{maxAll} := \frac{L_{girder}}{360} = 1.333\ in$$

$$\frac{\Delta_{max}}{\Delta_{maxAll}} = 0.938$$ *Good*

Moment Check

Design Moment $\phi M_n := 0.9 \cdot F_y \cdot Z_x = 750\ kip \cdot ft$

$$M_{max} := \frac{w_u \cdot L_{girder}^{\ 2}}{8} = 264.371\ kip \cdot ft \qquad \frac{M_{max}}{\phi M_n} = 0.352$$ *Good*

Shear Check

$t_w := 0.270\ \mathbf{in}$ $\quad d := 13.8\ \mathbf{in}$ $\quad t_f := 0.385\ \mathbf{in}$

$$A_w := t_w \cdot (d - 2 \cdot t_f)$$

$$\phi V_n := 0.6 \cdot F_y \cdot A_w = 105.543\ \mathbf{kip}$$

$$V_{max} := \frac{w_u \cdot L_{girder}}{2} = 26.437\ \mathbf{kip} \qquad \frac{V_{max}}{\phi V_n} = 0.25 \qquad \textit{Good}$$

Perimeter C-Channel Ring Calculations **C12x30**

Tributary Width $w_T := 7.5\ \mathbf{ft}$ $\quad I := 162\ \mathbf{in}^4$ $\quad Z_x := 33.8\ \mathbf{in}^3$

$LL := 50\ \mathbf{psf}$ Office Live Load - ASCE 7

$$SW_{beam} := 30\ \frac{\mathbf{lbf}}{\mathbf{ft}}$$

$DL_{MEP} := 4\ \mathbf{psf}$

$w_{conc\&deck} := 63.1\ \mathbf{psf}$ Vulcraft - Design Manual - 18 Gage 3VLI Composite Deck

Uniform Member Loads

$$w_L := w_T \cdot LL = 0.375\ \frac{\mathbf{kip}}{\mathbf{ft}}$$

$$w_D := w_T \cdot (DL_{MEP} + w_{conc\&deck}) = 0.503\ \frac{\mathbf{kip}}{\mathbf{ft}}$$

$$w_u := 1.2\ w_D + 1.2 \cdot (SW_{beam}) + 1.6\ w_L = 1.24\ \frac{\mathbf{kip}}{\mathbf{ft}}$$

Ultimate Factored Load

$L_{ring} := 35\ \mathbf{ft} + 5\ \mathbf{in}$ *Worst case scenario*

Bolt Shear Failure Check $s_{bolt} := 3\ \mathbf{in}$ $\quad s_{OC} := 48\ \mathbf{in}$ $\quad F_u := 50\ \mathbf{ksi}$

$$P_u := \frac{(w_u \cdot s_{OC})}{2 \cdot 2} = 1.24\ \mathbf{kip}$$

Shear in one bolt (if 2 bolts per 36in)

$\phi V_n := 1385\ \mathbf{lbf}$

KB-TZ2 - Hilti Anchor Fastening Design Manual
1/4" diameter, 1.5" embedment, 4000 psi concrete

Nominal anchor diameter in.	Effective embedment in. (mm)	Nominal embedment in. (mm)	Tension (lesser of concrete breakout / pullout) - ΦN_n				Shear (lesser of concrete breakout or pryout) - ΦV_n			
			f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.1 MPa) lb (kN)	f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.1 MPa) lb (kN)
1/4	1-1/2 (38)	1 3/4 (44)	280 (1.2)	300 (1.3)	340 (1.5)	395 (1.8)	1,095 (4.9)	1,195 (5.3)	1,385 (6.2)	1,695 (7.5)
3/8	1-1/2 (38)	1 7/8 (48)	1,255 (5.6)	1,375 (6.1)	1,585 (7.1)	1,940 (8.6)	1,350 (6.0)	1,480 (6.6)	1,710 (7.6)	2,090 (9.3)
	2 (51)	2 1/2 (64)	1,930 (8.6)	2,115 (9.4)	2,440 (10.9)	2,990 (13.3)	2,080 (9.3)	2,275 (10.1)	2,630 (11.7)	3,220 (14.3)
	2-1/2 (64)	3 (76)	2,185 (9.7)	2,390 (10.6)	2,765 (12.3)	3,385 (15.1)	4,705 (20.9)	5,155 (22.9)	5,950 (26.5)	7,285 (32.4)
1/2	1-1/2 (38)	2 (51)	1,435 (6.4)	1,570 (7.0)	1,815 (8.1)	2,220 (9.9)	1,545 (6.9)	1,690 (7.5)	1,950 (8.7)	2,390 (10.6)
	2 (51)	2 1/2 (64)	1,930 (8.6)	2,115 (9.4)	2,440 (10.9)	2,990 (13.3)	2,080 (9.3)	2,275 (10.1)	2,630 (11.7)	3,220 (14.3)
	2-1/2 (64)	3 (76)	2,700 (12.0)	2,955 (13.1)	3,415 (15.2)	4,180 (18.6)	5,810 (25.8)	6,365 (28.3)	7,350 (32.7)	9,000 (40.0)
	3-1/4 (83)	3 3/4 (95)	3,235 (14.4)	3,545 (15.8)	4,095 (18.2)	5,015 (22.3)	6,970 (31.0)	7,640 (34.0)	8,820 (39.2)	10,800 (48.0)

$$\frac{P_u}{\phi V_n} = 0.895 \qquad \textit{Good}$$

Bolt Shear Failure From Beam Point Load

$P_u := V_{max} = 26.437\ kip$ three bolts $P_u := \frac{P_u}{3} = 8.812\ kip$

5/8	2-3/4 (70)	3 1/4 (83)	3,110 (13.8)	3,410 (15.2)	3,935 (17.5)	4,820 (21.4)	6,705 (29.8)	7,345 (32.7)	8,480 (37.7)	10,385 (46.2)
	3-1/4 (83)	3 3/4 (95)	4,000 (17.8)	4,380 (19.5)	5,060 (22.5)	6,195 (27.6)	8,615 (38.3)	9,435 (42.0)	10,895 (48.5)	13,345 (59.4)
	4 (102)	4 1/2 (114)	4,420 (19.7)	4,840 (21.5)	5,590 (24.9)	6,845 (30.4)	9,520 (42.3)	10,430 (46.4)	12,040 (53.6)	14,750 (65.6)

KB-TZ2 - Hilti Anchor Fastening Design Manual
5/8" diameter, 3.25" embedment, 4000 psi concrete

$\phi V_n := 10895\ lbf$ $\frac{P_u}{\phi V_n} = 0.809$ *Good*

Connected Member Shear Failure Check

$\phi V_n := 165\ kip$ AISC Table 3-8

$V_n := \frac{(w_u \cdot s_{OC})}{2} = 2.48\ kip$ $\frac{V_n}{\phi V_n} = 0.015$ *Good*

Flange Local Bending Check From Beam Point Load

$t_f := 0.501\ in$ $F_{yf} := 50\ ksi$

$R_n := 6.25 \cdot F_{yf} \cdot t_f^{\ 2} = 78.438\ kip$ $P_u := V_{max} = 26.437\ kip$ *Ultimate Load From Beam*

$P_{fb} := 0.9 \cdot R_n = 70.594\ kip$ *Design Strength*

$\frac{P_u}{P_{fb}} = 0.374$ *Good*

Deflection Check

$\Delta_{max} := \frac{w_u \cdot s_{OC}^{\ 4}}{384 \cdot E \cdot I} = \left(3.04 \cdot 10^{-4}\right)\ in$ $\Delta_{maxAll} := \frac{s_{OC}}{360} = 0.133\ in$

$\frac{\Delta_{max}}{\Delta_{maxAll}} = 2.28 \cdot 10^{-3}$ *Good*

Moment Check

$M_{max} := \frac{w_u \cdot s_{OC}^{\ 2}}{12} = 1.653\ kip \cdot ft$

$\phi M_p := 0.9 \cdot F_y \cdot Z_x = 126.75\ \boldsymbol{kip \cdot ft}$ $\frac{M_{max}}{\phi M_p} = 0.013$ *Good*

Design of Concrete Slab

Moment Check

$\phi M_{no} := 10.41\ \boldsymbol{kip} \cdot \frac{\boldsymbol{ft}}{\boldsymbol{ft}}$ Vulcraft 3VLI-36 Composite Deck Slab
18 Gage, 6.5in slab depth $w_{conc\&deck} := 63.1\ \boldsymbol{psf}$

$LL := 150\ \boldsymbol{psf}$ Bakery & Commercial Kitchen Live Load
$DL_{MEP} := 4\ \boldsymbol{psf}$ - ASCE 7 (Worst case scenario)

$w_u := \left(1.2\ \left(DL_{MEP} + w_{conc\&deck}\right) + 1.6\ LL\right) \cdot 1\ \boldsymbol{ft} = 320.52\ \boldsymbol{plf}$ $l := 7.5\ \boldsymbol{ft}$

$M_u := \frac{\left(w_u \cdot l^2\right)}{8} = 2.254\ \boldsymbol{kip \cdot ft}$

Deflection Check $I := 18.62\ \boldsymbol{in}^4$ $E := 33 \cdot 145^{1.5} \cdot \sqrt[2]{4000} \cdot \boldsymbol{psi} = \left(3.644 \cdot 10^3\right)\ \boldsymbol{ks}$

$\Delta_{max} := \frac{5\ w_u \cdot l^4}{384 \cdot E \cdot I} = 0.336\ \boldsymbol{in}$ $\Delta_{maxAll} := \frac{l}{360} = 0.25\ \boldsymbol{in}$

$\frac{\Delta_{max}}{\Delta_{maxAll}} = 1.345$ *Good*

Shear Check

$\phi V_{no} := 7.97\ \frac{\boldsymbol{kip}}{\boldsymbol{ft}}$

$V_u := \frac{w_u \cdot l}{2} \cdot \frac{1}{\boldsymbol{ft}} = 1.202\ \frac{\boldsymbol{kip}}{\boldsymbol{ft}}$

$\frac{V_u}{\phi V_{no}} = 0.151$ *Good*

Welded Wire Reinforcement

6x6-W2.1xW2.1 Recommended for Vulcraft 3VLI-36 Composite Deck Slab
18 Gage, 6.5in slab depth

Mini Office Structural Calcs

W12x40

***3 beams, spaced at 9 feet O.C. covered by composite metal corrugated deck**

$E := 29000\ ksi$ $F_y := 50\ ksi$ $I := 307\ in^4$ $Z_x := 57\ in^3$

Tributary Width $w_T := 9\ ft$

$SW_{beam} := 40\ \frac{lbf}{ft}$

$LL := 50\ psf$ Office Live Load - ASCE 7

$DL_{MEP} := 4\ psf$

$w_{conc\&deck} := 63.1\ psf$ Vulcraft 3VLI-36 Composite Deck Slab
18 Gage, 6.5in slab depth

Uniform Member Loads

$w_L := w_T \cdot LL = 0.45\ \frac{kip}{ft}$

$w_D := w_T \cdot (DL_{MEP} + w_{conc\&deck}) = 0.604\ \frac{kip}{ft}$

$w_u := 1.2\ w_D + 1.2 \cdot (SW_{beam}) + 1.6\ w_L = 1.493\ \frac{kip}{ft}$ **Ultimate Factored Load**

$L_{girder} := 20\ ft$ *Worst case scenario*

Deflection Check

$\Delta_{max} := \frac{5\ w_u \cdot L_{girder}^4}{384 \cdot E \cdot I} = 0.604\ in$ $\Delta_{maxAll} := \frac{L_{girder}}{360} = 0.667\ in$

$\frac{\Delta_{max}}{\Delta_{maxAll}} = 0.905$ *Good*

Moment Check

Design Moment $\phi M_n := 0.9 \cdot F_y \cdot Z_x = 213.75\ kip \cdot ft$

$M_{max} := \frac{w_u \cdot L_{girder}^2}{8} = 74.634\ kip \cdot ft$ $\frac{M_{max}}{\phi M_n} = 0.349$ *Good*

Shear Check

$\phi V_n := 487\ kip$ ASIC Table 3-6

$V_{max} := \frac{w_u \cdot L_{girder}}{2} = 14.927\ kip$ $\frac{V_{max}}{\phi V_n} = 0.031$ *Good*

C-Channel Calculations **C12x25**

Tributary Width $w_T := 9\ ft$ $I := 144\ in^4$ $Z_x := 29.4\ in^3$

$SW_{beam} := 25\ \frac{lbf}{ft}$ $LL := 50\ psf$ Kitchen&Bakery Live Load - ASCE 7

$DL_{MEP} := 4\ psf$

$w_{conc\&deck} := 63.1\ psf$ Vulcraft 3VLI-36 Composite Deck Slab 18 Gage, 6.5in slab depth

Uniform Member Loads

$w_L := w_T \cdot LL = 0.45\ \frac{kip}{ft}$

$w_D := w_T \cdot \left(DL_{MEP} + w_{conc\&deck}\right) = 0.604\ \frac{kip}{ft}$

$w_u := 1.2\ w_D + 1.2 \cdot \left(SW_{beam}\right) + 1.6\ w_L = 1.475\ \frac{kip}{ft}$ **Ultimate Factored Load**

$L_{ring} := 6\ ft$ *Worst case scenario*

Connected Member Shear Failure Check

$\phi V_n := 125\ kip$ AISC Table 3-8

$V_n := \frac{V_{max}}{2} = 7.463\ kip$ $\frac{V_n}{\phi V_n} = 0.06$ *Good*

Flange Local Bending Check From Beam Point Load

$t_f := 0.501\ in$ $F_{yf} := 50\ ksi$

$R_n := 6.25 \cdot F_{yf} \cdot t_f^{\ 2} = 78.438\ kip$ $P_u := V_{max} = 14.927\ kip$ *Ultimate Load From Beam*

$P_{fb} := 0.9 \cdot R_n = 70.594\ kip$ *Design Strength*

$\frac{P_u}{P_{fb}} = 0.211$ *Good*

Deflection Check

$\Delta_{max} := \frac{w_u \cdot s_{OC}^4}{384 \cdot E \cdot I} = \left(4.068 \cdot 10^{-4}\right)\ in$ $\Delta_{maxAll} := \frac{s_{OC}}{360} = 0.133\ in$

$\frac{\Delta_{max}}{\Delta_{maxAll}} = 3.051 \cdot 10^{-3}$ *Good*

Moment Check

$M_{max} := \frac{w_u \cdot s_{OC}^2}{12} = 1.966\ kip \cdot ft$

$\phi M_p := 0.9 \cdot F_y \cdot Z_x = 110.25\ kip \cdot ft$

$\frac{M_{max}}{\phi M_p} = 0.018$ *Good*

Design of Concrete Slab

Moment Check

$\phi M_{no} := 10.41\ kip \cdot \frac{ft}{ft}$ Vulcraft 3VLI-36 Composite Deck Slab 18 Gage, 6.5in slab depth $w_{conc\&deck} := 63.1\ psf$

$LL := 50\ psf$ Bakery & Commercial Kitchen Live Load
$DL_{MEP} := 4\ psf$ - ASCE 7 (Worst case scenario)

$w_u := \left(1.2\ \left(DL_{MEP} + w_{conc\&deck}\right) + 1.6\ LL\right) \cdot 1\ ft = 160.52\ plf$ $l := 7.5\ ft$

$M_u := \frac{\left(w_u \cdot l^2\right)}{8} = 1.129\ kip \cdot ft$ $\frac{M_u}{\phi M_{no}} \cdot \frac{1}{ft} = 0.108$

Deflection Check $I := 26.87\ in^4$ $E := 33 \cdot 145^{1.5} \cdot \sqrt[2]{4000} \cdot psi = \left(3.644 \cdot 10^3\right)\ ksi$

$\Delta_{max} := \frac{5\ w_u \cdot l^4}{384 \cdot E \cdot I} = 0.117\ in$ $\Delta_{maxAll} := \frac{l}{360} = 0.25\ in$

$\frac{\Delta_{max}}{\Delta_{maxAll}} = 0.467$ *Good*

Shear Check

$\boxed{\phi V_{no}} := 9.46\ \frac{kip}{ft}$

$\boxed{V_u} := \frac{w_u \cdot l}{2} \cdot \frac{1}{ft} = 0.602\ \frac{kip}{ft}$

$\frac{V_u}{\phi V_{no}} = 0.064$ *Good*

Welded Wire Reinforcement

6x6-W2.1xW2.1 Recommended for Vulcraft 3VLI-36 Composite Deck Slab
18 Gage, 6.5in slab depth

Level 4 (Penthouse First Floor)

Floor Framing Calculations **W24x76**

Tributary Width $w_T := 7\ ft + 8\ in$

$SW_{beam} := 76\ \frac{lbf}{ft}$ $LL := 40\ psf$
$DL_{MEP} := 4\ psf$
$w_{conc\&deck} := 63.1\ psf$

Residential Live Load - ASCE 7
Vulcraft - Design Manual - 18 Gage 3VLI Composite Deck
6.5" Slab Thickness

$E := 29000\ ksi$ $F_y := 50\ ksi$
$I := 2100\ in^4$
$Z_x := 200\ in^3$

Uniform Member Loads

$$w_L := w_T \cdot LL = 0.307\ \frac{kip}{ft}$$

$$w_D := w_T \cdot \left(DL_{MEP} + w_{conc\&deck}\right) = 0.514\ \frac{kip}{ft}$$

$$w_u := 1.2\ w_D + 1.2 \cdot \left(SW_{beam}\right) + 1.6\ w_L = 1.199\ \frac{kip}{ft}$$

Ultimate Factored Load

Member Lengths & Areas (Worst Case Scenario)

$$L_{girder} := 40\ ft$$

Deflection Check

$$\Delta_{max} := \frac{5\ w_u \cdot L_{girder}^{\ 4}}{384 \cdot E \cdot I} = 1.134\ in$$ $$\Delta_{maxAll} := \frac{L_{girder}}{360} = 1.333\ in$$

$$\frac{\Delta_{max}}{\Delta_{maxAll}} = 0.851$$ *Good*

Moment Check

Design Moment $\phi M_n := 0.9 \cdot F_y \cdot Z_x = 750\ kip \cdot ft$

$$M_{max} := \frac{w_u \cdot L_{girder}^{\ 2}}{8} = 239.837\ kip \cdot ft$$ $$\frac{M_{max}}{\phi M_n} = 0.32$$ *Good*

Shear Check

$$\phi V_n := 315\ kip$$

$$V_{max} := \frac{w_u \cdot L_{girder}}{2} = 23.984\ kip$$ $$\frac{V_{max}}{\phi V_n} = 0.076$$ *Good*

Perimeter C-Channel Ring Calculations   C12x30

Tributary Width $w_T := 7\ ft + 8\ in$ $I := 162\ in^4$ $Z_x := 33.8\ in^3$

$LL := 40\ psf$ Residential Live Load - ASCE 7

$SW_{beam} := 30\ \frac{lbf}{ft}$ $DL_{MEP} := 4\ psf$

$w_{conc\&deck} := 63.1\ psf$ Vulcraft - Design Manual - 18 Gage 3VLI Composite Deck
6.5" Slab Thickness

Uniform Member Loads

$$w_L := w_T \cdot LL = 0.307\ \frac{kip}{ft}$$

$$w_D := w_T \cdot \left(DL_{MEP} + w_{conc\&deck}\right) = 0.514\ \frac{kip}{ft}$$

$$w_u := 1.2\ w_D + 1.2 \cdot \left(SW_{beam}\right) + 1.6\ w_L = 1.144\ \frac{kip}{ft}$$ **Ultimate Factored Load**

$L_{ring} := 35\ ft + 5\ in$ *Worst case scenario*

Bolt Shear Failure Check $s_{bolt} := 3\ in$ $s_{OC} := 48\ in$ $d_b := \frac{1}{2}\ in = 0.5\ in$ $A_b := \frac{\pi \cdot {d_b}^2}{4}$

$$P_u := \frac{\left(w_u \cdot s_{OC}\right)}{2 \cdot 2} = 1.144\ kip$$ Shear in one bolt (if 2 bolts per 36in)

$\phi V_n := 1385\ lbf$ KB-TZ2 - Hilti Anchor Fastening Design Manual
1/4" diameter, 1.5" embedment, 4000 psi concrete

Nominal anchor diameter in.	Effective embedment in. (mm)	Nominal embedment in. (mm)	Tension (lesser of concrete breakout / pullout) - ΦN_n				Shear (lesser of concrete breakout or pryout) - ΦV_n			
			f'c = 2,500 psi (17.2 MPa) lb (kN)	f'c = 3,000 psi (20.7 MPa) lb (kN)	f'c = 4,000 psi (27.6 MPa) lb (kN)	f'c = 6,000 psi (41.1 MPa) lb (kN)	f'c = 2,500 psi (17.2 MPa) lb (kN)	f'c = 3,000 psi (20.7 MPa) lb (kN)	f'c = 4,000 psi (27.6 MPa) lb (kN)	f'c = 6,000 psi (41.1 MPa) lb (kN)
1/4	1-1/2 (38)	1 3/4 (44)	280 (1.2)	300 (1.3)	340 (1.5)	395 (1.8)	1,095 (4.9)	1,195 (5.3)	1,385 (6.2)	1,695 (7.5)
3/8	1-1/2 (38)	1 7/8 (48)	1,255 (5.6)	1,375 (6.1)	1,585 (7.1)	1,940 (8.6)	1,350 (6.0)	1,480 (6.6)	1,710 (7.6)	2,090 (9.3)
	2 (51)	2 1/2 (64)	1,930 (8.6)	2,115 (9.4)	2,440 (10.9)	2,990 (13.3)	2,080 (9.3)	2,275 (10.1)	2,630 (11.7)	3,220 (14.3)
	2-1/2 (64)	3 (76)	2,185 (9.7)	2,390 (10.6)	2,765 (12.3)	3,385 (15.1)	4,705 (20.9)	5,155 (22.9)	5,950 (26.5)	7,285 (32.4)
1/2	1-1/2 (38)	2 (51)	1,435 (6.4)	1,570 (7.0)	1,815 (8.1)	2,220 (9.9)	1,545 (6.9)	1,690 (7.5)	1,950 (8.7)	2,390 (10.6)
	2 (51)	2 1/2 (64)	1,930 (8.6)	2,115 (9.4)	2,440 (10.9)	2,990 (13.3)	2,080 (9.3)	2,275 (10.1)	2,630 (11.7)	3,220 (14.3)
	2-1/2 (64)	3 (76)	2,700 (12.0)	2,955 (13.1)	3,415 (15.2)	4,180 (18.6)	5,810 (25.8)	6,365 (28.3)	7,350 (32.7)	9,000 (40.0)
	3-1/4 (83)	3 3/4 (95)	3,235 (14.4)	3,545 (15.8)	4,095 (18.2)	5,015 (22.3)	6,970 (31.0)	7,640 (34.0)	8,820 (39.2)	10,800 (48.0)

$$\frac{P_u}{\phi V_n} = 0.826$$ *Good*

Bolt Shear Failure From Beam Point Load

$P_u := V_{max} = 23.984\ kip$ two bolts $P_u := \frac{P_u}{2} = 11.992\ kip$

KB-TZ2 - Hilti Anchor Fastening Design Manual
5/8" diameter, 4" embedment, 4000 psi concrete

$\phi V_n := 12040\ lbf$ $\frac{P_u}{\phi V_n} = 0.996$ *Good*

Connected Member Shear Failure Check

$\boxed{\phi V_n} := 165\ \mathbf{kip}$ AISC Table 3-8

$$V_n := \frac{(w_u \cdot s_{OC})}{2} = 2.288\ \mathbf{kip} \qquad \frac{V_n}{\phi V_n} = 0.014 \qquad \textit{Good}$$

Flange Local Bending Check From Beam Point Load

$t_f := 0.501\ \mathbf{in}$ $\qquad F_{yf} := 50\ \mathbf{ksi}$

$R_n := 6.25 \cdot F_{yf} \cdot t_f^{\ 2} = 78.438\ \mathbf{kip}$ $\qquad \boxed{P_u} := V_{max} = 23.984\ \mathbf{kip}$ *Ultimate Load From Beam*

$P_{fb} := 0.9 \cdot R_n = 70.594\ \mathbf{kip}$ *Design Strength*

$$\frac{P_u}{P_{fb}} = 0.34 \qquad \textit{Good}$$

Deflection Check

$$\boxed{\Delta_{max}} := \frac{w_u \cdot s_{OC}^{\ 4}}{384 \cdot E \cdot I} = \left(2.805 \cdot 10^{-4}\right)\ \mathbf{in} \qquad \boxed{\Delta_{maxAll}} := \frac{s_{OC}}{360} = 0.133\ \mathbf{in}$$

$$\frac{\Delta_{max}}{\Delta_{maxAll}} = 2.104 \cdot 10^{-3} \qquad \textit{Good}$$

Moment Check

$$\boxed{M_{max}} := \frac{w_u \cdot s_{OC}^{\ 2}}{12} = 1.525\ \mathbf{kip \cdot ft}$$

$$\phi M_p := 0.9 \cdot F_y \cdot Z_x = 126.75\ \mathbf{kip \cdot ft}$$

$$\frac{M_{max}}{\phi M_p} = 0.012 \qquad \textit{Good}$$

Design of Concrete Slab

Moment Check

$\phi M_{no} := 10.41\ kip \cdot \frac{ft}{ft}$ Vulcraft 3VLI-36 Composite Deck Slab
18 Gage, 6.5in slab depth $w_{conc\&deck} := 63.1\ psf$

$LL := 40\ psf$ Bakery & Commercial Kitchen Live Load
$DL_{MEP} := 4\ psf$ - ASCE 7 (Worst case scenario)

$w_u := \left(1.2\ \left(DL_{MEP} + w_{conc\&deck}\right) + 1.6\ LL\right) \cdot 1\ ft = 144.52\ plf$ $l := 7.5\ ft$

$$M_u := \frac{\left(w_u \cdot l^2\right)}{8} = 1.016\ kip \cdot ft$$

Deflection Check

$I := 18.62\ in^4$ $E := 33 \cdot 145^{1.5} \cdot \sqrt[2]{4000 \cdot psi} = \left(3.644 \cdot 10^3\right)\ ksi$

$\Delta_{max} := \frac{5\ w_u \cdot l^4}{384 \cdot E \cdot I} = 0.152\ in$ $\Delta_{maxAll} := \frac{l}{360} = 0.25\ in$

$\frac{\Delta_{max}}{\Delta_{maxAll}} = 0.607$ *Good*

Shear Check

$\phi V_{no} := 7.97\ \frac{kip}{ft}$

$V_u := \frac{w_u \cdot l}{2} \cdot \frac{1}{ft} = 0.542\ \frac{kip}{ft}$

$\frac{V_u}{\phi V_{no}} = 0.068$ *Good*

Welded Wire Reinforcement

6x6-W2.1xW2.1 Recommended for Vulcraft 3VLI-36 Composite Deck Slab
19 Gage, 7.5in slab depth

Level 6 (Sleeping Quarter Wall Framing)

$V_{wind} := 110 mph$ $p_g := 46\ psf$ *ASCE Hazard Tool* Square HSS

HSS2x2x1/4

Velocity Pressure at Height z=145'

$K_z := 1.06$ Exposure Category B - 150 feet above ground level

$K_e := 0.96$ 1000 ft ground elevation above sea level

$K_{zt} := 1.0$ No topographic factor for wind

$q_z := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_e \cdot V_{wind}^2 \cdot psf = 31.521\ psf$ Velocity Pressure at 145'

$s := 16\ in$ Stud Spacing

$w := q_z \cdot s = 42.028\ plf$ Uniform Load on Each Stud

For the column acting as a fixed-fixed beam

Moment Check $l_{stud} := 10\ ft$ $E := 29000\ ksi$ $I := .747\ in^4$

$M_{max} := \frac{w \cdot l_{stud}^2}{12} = 0.35\ kip \cdot ft$ $F_y := 50\ ksi$ Cold Form Steel

$Z_x := 0.964\ in^3$

$\phi M_p := 0.9 \cdot F_y \cdot Z_x = 3.615\ kip \cdot ft$ $\frac{M_{max}}{\phi M_p} = 0.097$ *Good*

Shear Check

$V_{max} := \frac{w \cdot l_{stud}}{2} = 0.21\ kip$ $h := 1.3\ in$ $t := 0.233\ in$

$A_w := 2 \cdot h \cdot t = 0.606\ in^2$ $\phi V_n := 0.6 \cdot F_y \cdot A_w = 18.174\ kip$ $\frac{V_{max}}{\phi V_n} = 0.012$

Deflection Check

$\Delta_{max} := \frac{w \cdot l_{stud}^4}{384 \cdot E \cdot I} = 0.087\ in$ $\Delta_{allow} := \frac{l_{stud}}{500} = 0.24\ in$ $\frac{\Delta_{max}}{\Delta_{allow}} = 0.364$

Sloped Roof Snow Load $Slope := \frac{4}{20} \cdot 100 = 20$

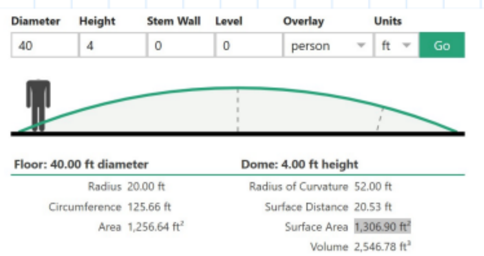
$C_e := 0.9$ *Fully exposed, surface roughness C* $R_{roof} := 50$

$C_s := 1.0$ 20 degree slope $C_t := 1.19$ R_{roof} = 50 & P_g = 50 psf

$p_f := 0.7 \cdot C_e \cdot C_t \cdot p_g = 34.486$ ***psf***

$p_s := C_s \cdot p_f = 34.486$ ***psf***

Glass Dome & Material Load



$\gamma_{glass} := 156$ ***pcf***

$q_{glass} := \gamma_{glass} \cdot 1\ \boldsymbol{in} = 13$ ***psf***

Factored load $w_t := 10\ \boldsymbol{ft}$

$w_u := 1.2\ (q_{glass} \cdot w_t) + 1.0\ (p_s \cdot w_t) = 0.501\ \frac{\boldsymbol{kip}}{\boldsymbol{ft}}$

Point Load on each column

$\frac{h}{t} = 5.579$ $1.40 \cdot \sqrt[2]{\frac{E}{F_y}} = 33.716$ Slenderness Good

$P_u := w_u \cdot s = 0.668$ ***kip***

$\phi P_n := 40$ ***kip*** $\frac{P_u}{\phi P_n} = 0.017$ *Good*

For HSS Subject to Shear, Moment, Torsion, & Axial Force:

$P_r := P_u$ $P_c := \phi P_n$ $M_{rx} := M_{max}$ $M_{ry} := M_{max}$ $M_{cy} := \phi M_p$ $M_{cx} := M_{cy}$

$V_r := V_{max}$ $V_c := \phi V_n$

$$\left(\frac{P_r}{P_c} + \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) + \left(\frac{V_r}{V_c}\right)^2 = 0.211$$

$$\left(\frac{P_r}{P_c} + \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) + \left(\frac{V_r}{V_c}\right)^2 \le 1 = 1$$

Level 5 (Sleeping Quarters)

Floor Framing Calculations **W24x76**

Tributary Width $w_T := 8\ ft$

$SW_{beam} := 76\ \frac{lbf}{ft}$ $LL := 40\ psf$

Residential Live Load - ASCE 7

Vulcraft - Design Manual - 18 Gage 3VLI Composite Deck

$p_s = 34.486\ psf$ *Snow*

$q_z = 31.521\ psf$ *Wind*

$E := 29000\ ksi$ $F_y := 50\ ksi$

$I := 2100\ in^4$

$Z_x := 200\ in^3$

$w_{gyp} := 2\ psf$ $SW_{HSS} := 5.41\ plf$

$w_{conc\&deck} := 63.1\ psf$

Uniform Member Loads

$w_L := w_T \cdot LL$ $w_D := w_T \cdot (w_{conc\&deck} + w_{gyp}) + SW_{beam} + SW_{HSS}$ $w_s := p_s \cdot w_T$ $w_w := q_z \cdot w_T$

$w_{u1} := 1.2\ (w_D) + 1.6\ w_L + 0.3\ w_s = 1.317\ \frac{kip}{ft}$

$w_{u2} := 1.2\ w_D + 1.0\ w_s + \max(w_L, 0.5\ w_w) = 1.319\ \frac{kip}{ft}$ **Ultimate Factored Load**

$w_{u3} := 1.2 \cdot w_D + w_w + w_L + 0.3\ w_s = 1.378\ \frac{kip}{ft}$

$w_u := w_{u3} = 1.378\ \frac{kip}{ft}$

Member Lengths & Areas (Worst Case Scenario)

$L_{girder} := 40\ ft$

Deflection Check

$\Delta_{max} := \frac{5\ w_u \cdot L_{girder}^{\ 4}}{384 \cdot E \cdot I} = 1.303\ in$ $\Delta_{maxAll} := \frac{L_{girder}}{360} = 1.333\ in$

$\frac{\Delta_{max}}{\Delta_{maxAll}} = 0.977$ *Good*

Moment Check

Design Moment $\phi M_n := 0.9 \cdot F_y \cdot Z_x = 750\ kip \cdot ft$

$$\boxed{M_{max}} := \frac{w_u \cdot L_{girder}^{\ 2}}{8} = 275.518\ \boldsymbol{kip \cdot ft} \qquad \frac{M_{max}}{\phi M_n} = 0.367 \qquad \textit{Good}$$

Shear Check

$$\boxed{\phi V_n} := 306\ \boldsymbol{kip}$$

$$\boxed{V_{max}} := \frac{w_u \cdot L_{girder}}{2} = 27.552\ \boldsymbol{kip} \qquad \frac{V_{max}}{\phi V_n} = 0.09 \qquad \textit{Good}$$

Perimeter C-Channel Ring Calculations **C12x30**

Tributary Width $\boxed{w_T} := 8\ \boldsymbol{ft}$ $\boxed{I} := 162\ \boldsymbol{in}^4$ $\boxed{Z_x} := 33.8\ \boldsymbol{in}^3$

Uniform Member Loads

$$w_u = 1.378\ \frac{\boldsymbol{kip}}{\boldsymbol{ft}}$$

$$L_{ring} := 35\ \boldsymbol{ft} + 5\ \boldsymbol{in}$$ *Worst case scenario*

Bolt Shear Failure Check $s_{bolt} := 3\ \boldsymbol{in}$ $s_{OC} := 42\ \boldsymbol{in}$

$$\boxed{P_u} := \frac{\left(w_u \cdot s_{OC}\right)}{2 \cdot 2} = 1.205\ \boldsymbol{kip}$$ Shear in one bolt (if 2 bolts per 36in)

$$\boxed{\phi V_n} := 1385\ \boldsymbol{lbf}$$

KB-TZ2 - Hilti Anchor Fastening Design Manual
1/4" diameter, 1.5" embedment, 4000 psi concrete

Nominal anchor diameter in.	Effective embedment in. (mm)	Nominal embedment in. (mm)	Tension (lesser of concrete breakout / pullout) - ΦN_n				Shear (lesser of concrete breakout or pryout) - ΦV_n			
			f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.1 MPa) lb (kN)	f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.1 MPa) lb (kN)
1/4	1-1/2 (38)	1 3/4 (44)	280 (1.2)	300 (1.3)	340 (1.5)	395 (1.8)	1,095 (4.9)	1,195 (5.3)	1,385 (6.2)	1,695 (7.5)
3/8	1-1/2 (38)	1 7/8 (48)	1,255 (5.6)	1,375 (6.1)	1,585 (7.1)	1,940 (8.6)	1,350 (6.0)	1,480 (6.6)	1,710 (7.6)	2,090 (9.3)
	2 (51)	2 1/2 (64)	1,930 (8.6)	2,115 (9.4)	2,440 (10.9)	2,990 (13.3)	2,080 (9.3)	2,275 (10.1)	2,630 (11.7)	3,220 (14.3)
	2-1/2 (64)	3 (76)	2,185 (9.7)	2,390 (10.6)	2,765 (12.3)	3,385 (15.1)	4,705 (20.9)	5,155 (22.9)	5,950 (26.5)	7,285 (32.4)
1/2	1-1/2 (38)	2 (51)	1,435 (6.4)	1,570 (7.0)	1,815 (8.1)	2,220 (9.9)	1,545 (6.9)	1,690 (7.5)	1,950 (8.7)	2,390 (10.6)
	2 (51)	2 1/2 (64)	1,930 (8.6)	2,115 (9.4)	2,440 (10.9)	2,990 (13.3)	2,080 (9.3)	2,275 (10.1)	2,630 (11.7)	3,220 (14.3)
	2-1/2 (64)	3 (76)	2,700 (12.0)	2,955 (13.1)	3,415 (15.2)	4,180 (18.6)	5,810 (25.8)	6,365 (28.3)	7,350 (32.7)	9,000 (40.0)
	3-1/4 (83)	3 3/4 (95)	3,235 (14.4)	3,545 (15.8)	4,095 (18.2)	5,015 (22.3)	6,970 (31.0)	7,640 (34.0)	8,820 (39.2)	10,800 (48.0)

$$\frac{P_u}{\phi V_n} = 0.87 \qquad \textit{Good}$$

Bolt Shear Failure From Beam Point Load

$$\boxed{P_u} := V_{max} = 27.552\ \boldsymbol{kip}$$ three bolts $$\boxed{P_u} := \frac{P_u}{3} = 9.184\ \boldsymbol{kip}$$

KB-TZ2 - Hilti Anchor Fastening Design Manual
5/8" diameter, 3.25" embedment, 4000 psi concrete

$\phi V_n := 10895\ \mathbf{lbf}$ $\qquad \frac{P_u}{\phi V_n} = 0.843$ $\qquad$ *Good*

Connected Member Shear Failure Check

$\phi V_n := 165\ \mathbf{kip}$ $\qquad$ AISC Table 3-8

$V_n := \frac{(w_u \cdot s_{OC})}{2} = 2.411\ \mathbf{kip}$ $\qquad \frac{V_n}{\phi V_n} = 0.015$ $\qquad$ *Good*

Flange Local Bending Check From Beam Point Load

$t_f := 0.510\ \mathbf{in}$ $\qquad F_{yf} := 50\ \mathbf{ksi}$

$R_n := 6.25 \cdot F_{yf} \cdot t_f^{\ 2} = 81.281\ \mathbf{kip}$ $\qquad P_u := V_{max} = 27.552\ \mathbf{kip}$ $\qquad$ *Ultimate Load From Beam*

$P_{fb} := 0.9 \cdot R_n = 73.153\ \mathbf{kip}$ $\qquad$ *Design Strength*

$\frac{P_u}{P_{fb}} = 0.377$ $\qquad$ *Good*

Deflection Check

$\Delta_{max} := \frac{w_u \cdot s_{OC}^{\ 4}}{384 \cdot E \cdot I} = (1.98 \cdot 10^{-4})\ \mathbf{in}$ $\qquad \Delta_{maxAll} := \frac{s_{OC}}{360} = 0.117\ \mathbf{in}$

$\frac{\Delta_{max}}{\Delta_{maxAll}} = 1.697 \cdot 10^{-3}$ $\qquad$ *Good*

Moment Check

$M_{max} := \frac{w_u \cdot s_{OC}^{\ 2}}{12} = 1.406\ \mathbf{kip} \cdot \mathbf{ft}$

$\phi M_p := 0.9 \cdot F_y \cdot Z_x = 126.75\ \mathbf{kip} \cdot \mathbf{ft}$

$\frac{M_{max}}{\phi M_p} = 0.011$ $\qquad$ *Good*

Design of Concrete Slab for 1' Slab

Moment Check

$\phi M_{no} := 10.95\ \mathbf{kip} \cdot \dfrac{\mathbf{ft}}{\mathbf{ft}}$

Vulcraft 3VLI-36 Composite Deck Slab
18 Gage, 6.5 in slab depth

$w_u := \dfrac{\left(1.2 \cdot w_D + w_w + w_L + 0.3\ w_s\right)}{w_T} \cdot 1\ \mathbf{ft} = 0.172\ \dfrac{\mathbf{kip}}{\mathbf{ft}}$

Use 1 ft strip width as trib

$l := 8\ \mathbf{ft}$

$M_u := \dfrac{\left(w_u \cdot l^2\right)}{8} = 1.378\ \mathbf{kip} \cdot \mathbf{ft}$

Deflection Check

$I := 18.62\ \mathbf{in}^4$

$E := 33 \cdot 145^{1.5} \cdot \sqrt[2]{4000 \cdot \mathbf{psi}} = \left(3.644 \cdot 10^3\right)\ \mathbf{ksi}$

$\Delta_{max} := \dfrac{5\ w_u \cdot l^4}{384 \cdot E \cdot I} = 0.234\ \mathbf{in}$

$\Delta_{maxAll} := \dfrac{l}{360} = 0.267\ \mathbf{in}$

$\dfrac{\Delta_{max}}{\Delta_{maxAll}} = 0.877$ *Good*

Shear Check

$\phi V_{no} := 7.97\ \dfrac{\mathbf{kip}}{\mathbf{ft}}$

$V_u := \dfrac{w_u \cdot l}{2} \cdot \dfrac{1}{\mathbf{ft}} = 0.689\ \dfrac{\mathbf{kip}}{\mathbf{ft}}$

$\dfrac{V_u}{\phi V_{no}} = 0.086$ *Good*

Welded Wire Reinforcement

6x6-W2.1xW2.1

Recommended for Vulcraft 3VLI-36 Composite Deck Slab
19 Gage, 7.5in slab depth

Plate & Weld Design

Weld available strength For W12x40 Beam

$D := 1\ in$ Weld size in sixteenths of an inch $d := 11.9\ in$

$b_f := 8.01\ in$

$l := b_f$ Flange Weld

$F_{EXX} := 70\ ksi$ E70 Electrode Fill

$$\phi R_{nW12} := 0.75 \cdot 0.60 \cdot F_{EXX} \cdot \left(\frac{\sqrt[2]{2}}{2}\right) \cdot \left(\frac{D}{16}\right) \cdot l = 11.151\ kip$$ Required Strength

Weld available strength For C12x30 Beam

$D := 2\ in$ Weld size in sixteenths of an inch $d := 12\ in$

$b_f := 3.17\ in$

$l := b_f$ Flange Weld

$F_{EXX} := 70\ ksi$ E70 Electrode Fill

$$\phi R_{nC12} := 0.75 \cdot 0.60 \cdot F_{EXX} \cdot \left(\frac{\sqrt[2]{2}}{2}\right) \cdot \left(\frac{D}{16}\right) \cdot l = 8.826\ kip$$ Required Strength

Weld available strength For HSS2X2X1/4 Column

$D := 2\ in$ Weld size in sixteenths of an inch $d := 1.3\ in$

$b_f := 1.3\ in$

$l := b_f$ Flange Weld

$F_{EXX} := 70\ ksi$ E70 Electrode Fill

$$\phi R_{nHSS2x2x.25} := 0.75 \cdot 0.60 \cdot F_{EXX} \cdot \left(\frac{\sqrt[2]{2}}{2}\right) \cdot \left(\frac{D}{16}\right) \cdot l = 3.62\ kip$$ Required Strength

L1 Shear at W12x40 Weld

$SW_{beam} := 40\ \frac{lbf}{ft}$ $LL := 50\ psf$ $w_T := 4.5\ ft$ $l := 8\ ft$

$DL_{MEP} := 4\ psf$

$w_{conc\&deck} := 63.1\ psf$

$w_L := w_T \cdot LL = 0.225\ \frac{kip}{ft}$

$w_D := w_T \cdot (DL_{MEP} + w_{conc\&deck}) = 0.302\ \frac{kip}{ft}$

$w_u := 1.2\ w_D + 1.2 \cdot (SW_{beam}) + 1.6\ w_L = 0.77\ \frac{kip}{ft}$

$V_{uW12} := \frac{(w_u \cdot l)}{2} = 3.081\ kip$

$\frac{V_{uW12}}{\phi R_{nC12}} = 0.349$ *Good*

L1 Shear at C12x30 Weld

$SW_{beam} := 40\ \frac{lbf}{ft}$ $LL := 50\ psf$ $w_T := 9\ ft$ $l := 20\ ft$

$DL_{MEP} := 4\ psf$

$w_{conc\&deck} := 63.1\ psf$

$w_L := w_T \cdot LL = 0.45\ \frac{kip}{ft}$

$w_D := w_T \cdot (DL_{MEP} + w_{conc\&deck}) = 0.604\ \frac{kip}{ft}$

$w_u := 1.2\ w_D + 1.2 \cdot (SW_{beam}) + 1.6\ w_L = 1.493\ \frac{kip}{ft}$

$P_u := \frac{(w_u \cdot l)}{2} = 14.927\ kip$ Point Load from Beam to C Channel

$V_{uC12} := \frac{P_u}{2} = 7.463\ kip$ Shear at C-channel weld

$\frac{V_{uC12}}{\phi R_{nC12}} = 0.846$ *Good*

L1 Shear at Hss2x2x1/4 Weld

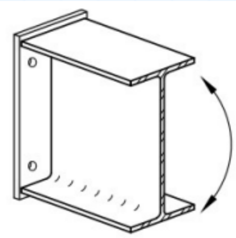
$w := 42.028\ \boldsymbol{plf}$ Wind Load Causing Shear

$l_{stud} := 10\ \boldsymbol{ft}$ Stud length

$$V_{HSS} := \frac{w \cdot l_{stud}}{2} = 0.21\ \boldsymbol{kip}$$ Shear at C-channel weld

$$\frac{V_{HSS}}{\phi R_{nC12}} = 0.024 \qquad Good$$

Bolt Check for 2B Flush W12x40



4 Total Bolts

$V_{uW12} = 3.081\ \boldsymbol{kip}$

$$R_{bolt} := \frac{V_{uW12}}{4} = 0.77\ \boldsymbol{kip}$$

For 1/4" Diameter Bolt, 1 3/4" embedment

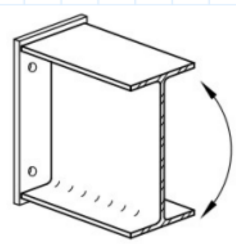
(d) *Two-bolt flush (2B Flush)*

Nominal anchor diameter in.	Effective embedment in. (mm)	Nominal embedment in. (mm)	Tension (lesser of concrete breakout / pullout) - ΦN_n f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.1 MPa) lb (kN)	Shear (lesser of concrete breakout or pr f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)
1/4	1-1/2 (38)	1 3/4 (44)	280 (1.2)	300 (1.3)	340 (1.5)	395 (1.8)	1,095 (4.9)	1,195 (5.3)	1,385 (6.2)
3/8	1-1/2 (38)	1 7/8 (48)	1,255 (5.6)	1,375 (6.1)	1,585 (7.1)	1,940 (8.6)	1,350 (6.0)	1,480 (6.6)	1,710 (7.6)
	2 (51)	2 1/2 (64)	1,930 (8.6)	2,115 (9.4)	2,440 (10.9)	2,990 (13.3)	2,080 (9.3)	2,275 (10.1)	2,630 (11.7)
	2-1/2 (64)	3 (76)	2,185 (9.7)	2,390 (10.6)	2,765 (12.3)	3,385 (15.1)	4,705 (20.9)	5,155 (22.9)	5,950 (26.5)

$\phi V_n := 1385\ \boldsymbol{lbf}$

$$\frac{R_{bolt}}{\phi V_n} = 0.556 \qquad Good$$

Bolt Check for 2B Flush C12x30



4 Total Bolts

$V_{uC12} = 7.463\ \boldsymbol{kip}$

$$R_{bolt} := \frac{V_{uC12}}{4} = 1.866\ \boldsymbol{kip}$$

For 3/8" Diameter Bolt, 2.5" embedment

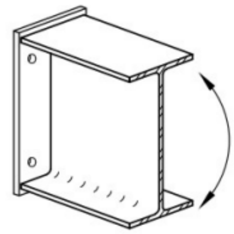
(d) *Two-bolt flush (2B Flush)*

Nominal anchor diameter in.	Effective embedment in. (mm)	Nominal embedment in. (mm)	Tension (lesser of concrete breakout / pullout) - ΦN_n f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.1 MPa) lb (kN)	Shear (lesser of concrete breakout or pr f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)
1/4	1-1/2 (38)	1 3/4 (44)	280 (1.2)	300 (1.3)	340 (1.5)	395 (1.8)	1,095 (4.9)	1,195 (5.3)	1,385 (6.2)
3/8	1-1/2 (38)	1 7/8 (48)	1,255 (5.6)	1,375 (6.1)	1,585 (7.1)	1,940 (8.6)	1,350 (6.0)	1,480 (6.6)	1,710 (7.6)
	2 (51)	2 1/2 (64)	1,930 (8.6)	2,115 (9.4)	2,440 (10.9)	2,990 (13.3)	2,080 (9.3)	2,275 (10.1)	2,630 (11.7)
	2-1/2 (64)	3 (76)	2,185 (9.7)	2,390 (10.6)	2,765 (12.3)	3,385 (15.1)	4,705 (20.9)	5,155 (22.9)	5,950 (26.5)

$\phi V_n := 2630\ \boldsymbol{lbf}$

$$\frac{R_{bolt}}{\phi V_n} = 0.709 \qquad Good$$

Bolt Check for 2B Flush HSS2X2X1/4



(d) *Two-bolt flush (2B Flush)*

4 Total Bolts

$$V_{HSS} = 0.21\ kip$$

$$R_{bolt} := \frac{V_{HSS}}{4} = 0.053\ kip$$

For 1/4" Diameter Bolt, 1.75" embedment

$$\phi V_n := 1195\ lbf$$

$$\frac{R_{bolt}}{\phi V_n} = 0.044 \qquad Good$$

Nominal anchor diameter in.	Effective embedment in. (mm)	Nominal embedment in. (mm)	Tension (lesser of concrete breakout / pullout) - ΦN_n				Shear (lesser of concrete breakout or pr		
			f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.1 MPa) lb (kN)	f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)
1/4	1-1/2 (38)	1 3/4 (44)	280 (1.2)	300 (1.3)	340 (1.5)	395 (1.8)	1,095 (4.9)	1,195 (5.3)	1,385 (6.2)
3/8	1-1/2 (38)	1 7/8 (48)	1,255 (5.6)	1,375 (6.1)	1,585 (7.1)	1,940 (8.6)	1,350 (6.0)	1,480 (6.6)	1,710 (7.6)
	2 (51)	2 1/2 (64)	1,930 (8.6)	2,115 (9.4)	2,440 (10.9)	2,990 (13.3)	2,080 (9.3)	2,275 (10.1)	2,630 (11.7)
	2-1/2 (64)	3 (76)	2,185 (9.7)	2,390 (10.6)	2,765 (12.3)	3,385 (15.1)	4,705 (20.9)	5,155 (22.9)	5,950 (26.5)

Plate Thickness for W12x40

Try a 14x14 in plate

Force Transfer

$d := 11.9\ in \qquad N := 14\ in \qquad B := 14\ in \qquad b_f := 8.01\ in$

$$P_u := V_{uW12} = 3.081\ kip$$

$$m := \frac{N - 0.95\ d}{2} = 1.348\ in$$

$$n := \frac{B - 0.8\ b_f}{2} = 3.796\ in$$

Take largest out of three

$$n' := \frac{\sqrt[2]{d \cdot b_f}}{4} = 2.441\ in$$

Available Strength $\qquad F_y := 36\ ksi \qquad$ A36 Steel $\qquad l := \max(m, n, n') = 3.796\ in$

$$t_{min} := l \cdot \sqrt[2]{\frac{2 \cdot P_u}{0.9\ F_y \cdot B \cdot N}} = 0.118\ in$$

$$\frac{1}{8}\ in = 0.125\ in$$

$$t_{W12} := \frac{1}{8}\ in$$

Plate Thickness for C12x40 Try a 14x14 in plate

Force Transfer

$d := 12\ in$ $N := 14\ in$ $B := 14\ in$ $b_f := 3.17\ in$

$P_u := V_{uC12} = 7.463\ kip$

$m := \dfrac{N - 0.95\ d}{2} = 1.3\ in$

$n := \dfrac{B - 0.8\ b_f}{2} = 5.732\ in$ Take largest out of three

$n' := \dfrac{\sqrt[2]{d \cdot b_f}}{4} = 1.542\ in$

Available Strength $F_y := 36\ ksi$ A36 Steel $l := \max(m, n, n') = 5.732\ in$

$t_{min} := l \cdot \sqrt[2]{\dfrac{2 \cdot P_u}{0.9\ F_y \cdot B \cdot N}} = 0.278\ in$ $\dfrac{2}{8}\ in = 0.25\ in$ $\dfrac{3}{8}\ in = 0.375\ in$

$t_{C12} := \dfrac{3}{8}\ in$

Plate Thickness for HSS2X2X1/4 Try a 6X6 in plate

Force Transfer

$d := 1.3\ in$ $N := 6\ in$ $B := 6\ in$ $b_f := 1.3\ in$

$P_u := V_{HSS} = 0.21\ kip$

$m := \dfrac{N - 0.95\ d}{2} = 2.383\ in$

$n := \dfrac{B - 0.8\ b_f}{2} = 2.48\ in$ Take largest out of three

$n' := \dfrac{\sqrt[2]{d \cdot b_f}}{4} = 0.325\ in$

Available Strength $F_y := 36\ ksi$ A36 Steel $l := \max(m, n, n') = 2.48\ in$

$$t_{min} := l \cdot \sqrt[2]{\frac{2 \cdot P_u}{0.9\ F_y \cdot B \cdot N}} = 0.047\ in$$

$$\frac{1}{8}\ in = 0.125\ in$$

$$t_{HSS} := \frac{1}{8}\ in$$

Interior Ceiling Framing

Conference Level Ceiling — Vulcraft 3NA-24 Acoustical Roof Deck
16 Gage

4 Beams, 2 end beams, 2 spaced 12'-8" O.C — **W14x43**

$w_T := 12\ ft + 8\ in$ $\quad SW_{beam} := 43\ plf$

$w_{deck} := 3.9\ psf$ $\quad MEP := 4\ psf$ $\quad w_{sheat} := 2\ psf$ $\quad w_{insul} := 0.8\ psf$

$w_u := 1.4\ (w_{deck} + MEP + w_{sheat} + w_{insul}) \cdot w_T + 1.4 \cdot SW_{beam} = 249.947\ plf$

$F_y := 50\ ksi$

$L_{girder} := 40\ ft$ $\quad E := 29000\ ksi$

$I := 428\ in^4$

$Z_x := 69.6\ in^3$

Deflection Check

$\Delta_{max} := \dfrac{5\ w_u \cdot L_{girder}^{\ 4}}{384 \cdot E \cdot I} = 1.16\ in$ $\quad \Delta_{maxAll} := \dfrac{L_{girder}}{360} = 1.333\ in$

$\dfrac{\Delta_{max}}{\Delta_{maxAll}} = 0.87$ *Good*

Moment Check

Design Moment $\quad \phi M_n := 0.9 \cdot F_y \cdot Z_x = 261\ kip \cdot ft$

$M_{max} := \dfrac{w_u \cdot L_{girder}^{\ 2}}{8} = 49.989\ kip \cdot ft$ $\quad \dfrac{M_{max}}{\phi M_n} = 0.192$ *Good*

Shear Check

$\phi V_n := 306\ kip$

$V_{max} := \dfrac{w_u \cdot L_{girder}}{2} = 4.999\ kip$ $\quad \dfrac{V_{max}}{\phi V_n} = 0.016$ *Good*

Perimeter C-Channel Ring Calculations — C12x30

Tributary Width $\quad w_T := 12\ ft + 8\ in$ $\quad I := 162\ in^4$ $\quad Z_x := 33.8\ in^3$

Uniform Member Loads

$w_u = 0.25\ \dfrac{kip}{ft}$

$L_{ring} := 35\ ft + 5\ in$ *Worst case scenario*

Bolt Shear Failure Check $s_{bolt} := 3\ in$ $s_{OC} := 48\ in$

$P_u := \frac{(w_u \cdot s_{OC})}{2 \cdot 2} = 0.25\ kip$ Shear in one bolt (if 2 bolts per 48in)

$\boxed{\phi V_n} := 1385\ lbf$ KB-TZ2 - Hilti Anchor Fastening Design Manual
1/4" diameter, 1.5" embedment, 4000 psi concrete

$\frac{P_u}{\phi V_n} = 0.18$ *Good*

Bolt Shear Failure From Beam Point Load

$\boxed{P_u} := V_{max} = 4.999\ kip$ two bolts $\boxed{P_u} := \frac{P_u}{2} = 2.499\ kip$

KB-TZ2 - Hilti Anchor Fastening Design Manual
1/2" diameter, 2" embedment, 4000 psi concrete

$\boxed{\phi V_n} := 2630\ lbf$ $\frac{P_u}{\phi V_n} = 0.95$ *Good*

Connected Member Shear Failure Check

$\boxed{\phi V_n} := 165\ kip$ AISC Table 3-8

$V_n := \frac{(w_u \cdot s_{OC})}{2} = 0.5\ kip$ $\frac{V_n}{\phi V_n} = 3.03 \cdot 10^{-3}$ *Good*

Flange Local Bending Check From Beam Point Load

$t_f := 0.510\ in$ $F_{yf} := 50\ ksi$

$R_n := 6.25 \cdot F_{yf} \cdot t_f^{\ 2} = 81.281\ kip$ $\boxed{P_u} := V_{max} = 4.999\ kip$ *Ultimate Load From Beam*

$P_{fb} := 0.9 \cdot R_n = 73.153\ kip$ *Design Strength*

$\frac{P_u}{P_{fb}} = 0.068$ *Good*

Deflection Check

$$\boxed{\Delta_{max}} := \frac{w_u \cdot s_{OC}^{\;4}}{384 \cdot E \cdot I} = \left(6.129 \cdot 10^{-5}\right)\ \textit{in} \qquad \boxed{\Delta_{maxAll}} := \frac{s_{OC}}{360} = 0.133\ \textit{in}$$

$$\frac{\Delta_{max}}{\Delta_{maxAll}} = 4.597 \cdot 10^{-4} \qquad \textit{Good}$$

Moment Check

$$\boxed{M_{max}} := \frac{w_u \cdot s_{OC}^{\;2}}{12} = 0.333\ \textit{kip} \cdot \textit{ft} \qquad \phi M_p := 0.9 \cdot F_y \cdot Z_x = 126.75\ \textit{kip} \cdot \textit{ft}$$

$$\frac{M_{max}}{\phi M_p} = 2.629 \cdot 10^{-3}$$

Design of Non Composite Roof for 1' Strip

$$l := 12\ \textit{ft} + 8\ \textit{in}$$

$$\boxed{w_u} := \frac{w_u}{w_T} \cdot 1\ \textit{ft} = 19.733\ \textit{plf}$$

$$\phi M_{no} := 2496\ \textit{lbf} \cdot \frac{\textit{ft}}{\textit{ft}}$$

$$M_u := \frac{\left(w_u \cdot l^2\right)}{8} = 395.749\ \textit{lbf} \cdot \textit{ft}$$

$$\frac{M_u}{\phi M_{no}} \cdot \frac{1}{\textit{ft}} = 0.159 \qquad \textit{Good}$$

Deflection Check $\boxed{I} := 1.598\ \textit{in}^4$ $\boxed{E} := 29000\ \textit{ksi} = \left(2.9 \cdot 10^4\right)\ \textit{ksi}$

$$\boxed{\Delta_{max}} := \frac{5\ w_u \cdot l^4}{384 \cdot E \cdot I} = 0.247\ \textit{in} \qquad \boxed{\Delta_{maxAll}} := \frac{l}{240} = 0.633\ \textit{in}$$

$$\frac{\Delta_{max}}{\Delta_{maxAll}} = 0.389 \qquad \textit{Good}$$

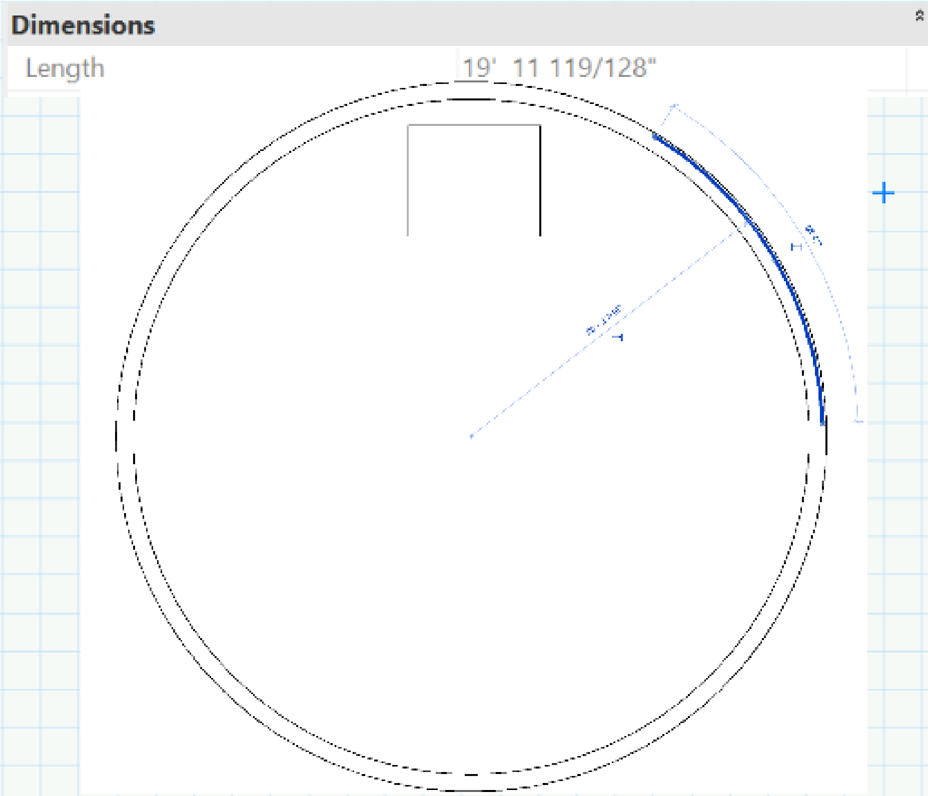
Shear Check

$$\phi V_{no} := 8.126\ \frac{\textit{kip}}{\textit{ft}}$$

$$V_u := \frac{w_u \cdot l}{2} \cdot \frac{1}{\textit{ft}} = 0.125\ \frac{\textit{kip}}{\textit{ft}}$$

$$\frac{V_u}{\phi V_{no}} = 0.015 \qquad \textit{Good}$$

Shear Wall Design



Shear Wall Properties

$t := 3.5\ \mathbf{in}$ $\quad L_{eff} := 20\ \mathbf{ft}$ $\quad L_{wall} := 40\ \mathbf{ft}$ $\quad h := 10\ \mathbf{ft}$

$A := t \cdot L_{eff} = 840\ \mathbf{in}^2$

$E := 29000\ \mathbf{ksi}$ *Steel* $\quad G := 11200\ \mathbf{ksi}$ *Steel*

Wind Pressure Values

$q_{wind} := 31.521\ \mathbf{psf}$ $\quad w_{wind} := q_{wind} \cdot h = 315.21\ \mathbf{plf}$

$$V_{wind} := \frac{\left(L_{wall} \cdot w_{wind}\right)}{2} = 6.304\ \mathbf{kip}$$

Drift Calcs

$$\Delta := \frac{1.2 \cdot V_{wind} \cdot h}{A \cdot G} = \left(9.649 \cdot 10^{-5}\right)\ \mathbf{in} \qquad \Delta_{all} := \frac{h}{500} = 0.24\ \mathbf{in}$$

$\Delta < \Delta_{all} = 1$ *Good*

Choose SSW12x10

Model No.	Dimensions (in.)			Qty. of Top of Wall Screws	Anchor Bolt Dia.	Factored Shear Resistance, V_r (lb.)	Anchor Bolt[6] Tension T_r (lb.)	Drift at V_r (in.)	Specified[4] Load at H/500 (lb.)	Total Wall Weight (lb.)
	W	H	T							
SSW12x10	12	117 ¼	3 ½	4	¾"	645	8720	0.588	265	104

Shear Calcs

$V := V_{wind} = 6.304\ kip$ $\quad V_{boltdes} := 645\ lbf$

$n_{bolt} := \dfrac{V}{V_{boltdes}} = 9.774$ $\quad \dfrac{120}{n_{bolt}} = 12.278$ $\quad$ 12 in O.C. (10 bolt total)

$V_{des} := 10 \cdot V_{boltdes} = 6.45\ kip$

$\dfrac{V}{V_{des}} = 0.977$ *Good*

Moment & Tension Calcs

$M_{OT} := V_{wind} \cdot \dfrac{h}{2} = 31.521\ kip \cdot ft$

$T_{req} := \dfrac{M_{OT}}{L_{eff}} = 1.576\ kip$

$T_{des} := 8720\ lbf$

$\dfrac{T_{req}}{T_{des}} = 0.181$ *Good*

Footing Design

1. Footing depth, D—Based on architectural requirements (primarily elevation of lowest floor) and soil conditions (see Section 9.1).
2. Footing width, B—Determined by geotechnical design (see Section 9.1).
3. Footing thickness, T—Determined by shear demand. May also be necessary to revise D in order to provide adequate cover for the floor slab.
4. Flexural reinforcement, size and spacing of bars—Determined by flexural demand.
5. Connection details—Determined by type, shape, and design of columns or walls.

FOOTING SIZE TABLE								
40 psf Live Load	Maximum Tributary Area (Sq. Ft.)							
	10	20	30	40	50	60	70	80
	Required minimum Diameter of a Round Footing (inches)							
	8	10	12	14	16	18	18	20
Frost Depth is 42". Minimum thickness is 12" for all footings.								
Tributary area figured as X or Y in diagram above. Assumed 2000 psf soil bearing. Non-frost-protected, free standing deck footing sizes shall be determined by this table.								

$TRIB := 8\ ft$

Loads on Continuous Footing

Guess

$H := 1\ ft$

$w_c := 145\ pcf$ $\quad t_{wall} := 8\ in$ $\quad h_{wall} := 43.5\ ft$

$w_{wall} := w_c \cdot t_{wall} \cdot h_{wall} = 4.205\ \frac{kip}{ft}$ $\quad p_g := 46\ psf$

$w_{snow} := p_g \cdot TRIB = 368\ plf$

1. Footing Depth

$P_s := w_{wall} + w_{snow} = 4.573\ \frac{kip}{ft}$

$D := 12\ in$

Load P/b (k/ft)	Minimum D (in)
0–10	12

2. Footing Width

- **ULS requirement**—The bearing capacity design criterion (Equation 7.37 or 7.39) must be satisfied. The techniques for doing so are discussed in Chapter 7.
- **SLS requirements**—The settlement and differential settlement criteria (Equations 5.20 and 5.21) must be satisfied. The techniques for doing so are described in Chapter 8.

$P_s := 4.573\ \ klf$ $\quad q_{allow} := 2\ \ ksf\ \ Assume$ $\quad w_{footing} := 0.145\ B$

$F.S. := 3.5$

$$P_s := P_s + w_{footing} \xrightarrow{simplify,\, float,\, 4} 0.145 \cdot B + 4.573$$

$$q_s := \frac{P_s}{B} \xrightarrow{simplify} \frac{4.573}{B} + 0.145$$

$$q_s = q_{allow} \xrightarrow{solve,\, B,\, float,\, 4} 2.465 \qquad B := 2.5\ ft$$

Check Service Condition $H := 1\ ft$

$w_f := 145\ pcf \cdot B \cdot H = 362.5\ plf$

$P_s := 4.573\ klf + w_f = \left(4.936 \cdot 10^3\right)\ plf$

$q_s := \dfrac{P_s}{B} = \left(1.974 \cdot 10^3\right)\ psf$ $q_{allow} := 2\ ksf$

$\dfrac{q_s}{q_{allow}} = 0.987$

Factored Load for Structural Design $D := w_{wall} + w_f = \left(4.568 \cdot 10^3\right)\ plf$

$S := w_{snow} = 368\ plf$

$P_u := 1.2\ D + 1.6\ S = 6.07\ klf$

$q_u := \dfrac{P_u}{B} = 2.428\ ksf$

One-way Shear Check $c := 8$

$P_u := 6070\ \dfrac{lb}{ft}$ $\phi := 0.75$

$f'_c := 4000\ psi$ $B := 30\ in$

$b_w := 12\ in$

$d := \dfrac{P_u \cdot (B - c)}{48 \cdot \phi \cdot b_w \cdot \sqrt[2]{f'_c} + 2\ P_u} = 3.384\ in$

$V_{uc} := P_u \cdot \left(\dfrac{B - c - 2\ d}{B}\right) = 3.082 \cdot 10^3\ lbf$ **Required Shear Strength**

$b_o := c + d = 11.384$

$V_n := 4\ b_o \cdot d \cdot \sqrt[2]{f'_c} = 9.746 \cdot 10^3$

$\phi V_n := 0.75\ V_n = 7.309 \cdot 10^3$ **Design Shear Strength**

$\dfrac{V_{uc}}{\phi V_n} = 0.422$ *Good*

Thickness, T $d_b := \frac{6}{8}$

$T := d + d_b + 3 = 7.134$ $T := 8\ in$

Flexural Design $spacingGUESS := \frac{(30\ in - 6\ in)}{4} = 6\ in$

$l := \frac{(B - c)}{2}\ in = 11\ in$ $q_{wind} := 31.521\ psf$ $B := 2.5\ ft$

$P_u := P_u \cdot lbf = 6.07\ kip$ $M_{wall} := 31.521\ kip \cdot ft$

$M_{uc} := \frac{P_u \cdot l^2}{2\ B} + \frac{2\ M_{wall} \cdot l}{B} = 24.135\ kip \cdot ft$ $f_y := 6 \cdot 10^4$ psi

$B := 30$ in $d = 3.384$ in $M_{uc} := 2.896 \cdot 10^5\ lbf \cdot in$

$f'_c = 4 \cdot 10^3\ psi$

$A_s := \left(\frac{f'_c \cdot B}{1.176\ f_y}\right)\left(d - \sqrt[2]{d^2 - \frac{2.353\ M_{uc}}{\phi \cdot f'_c \cdot B}}\right) = 2.405\ in^2$ $A_s := 2.405\ in^2$ $f_y := 60\ ksi$

$f'_c := 4000\ psi$ $B := 2.5\ ft$ $d := 3.384\ in$

$a := \frac{A_s \cdot f_y}{0.85\ f'_c \cdot B} = 1.415\ in$

$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 32.187\ kip \cdot ft$

$\phi M_n := 0.9\ M_n = 28.968\ kip \cdot ft$ *Design* $M_{uc} := 24.135\ (kip \cdot ft)$ *Required*

$\frac{M_{uc}}{\phi M_n} = 0.833$ *Good*

Large Opening Calculations

Bolt Shear Failure Check
15.5' Opening

6 total bolts

Table 2 — Hilti Carbon Steel KB-TZ2 design strength based on concrete failure modes in uncracked concrete per ACI 318 Ch. 17, applicable for both hammer and core drilled installations [1,2,3,4]

Nominal anchor diameter in.	Effective embedment in. (mm)	Nominal embedment in. (mm)	Tension (lesser of concrete breakout / pullout) - ΦN_n				Shear (lesser of concrete breakout or pryout) - ΦV_n			
			f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.1 MPa) lb (kN)	f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.1 MPa) lb (kN)
1/4	1-1/2 (38)	1 3/4 (44)	945 (4.2)	980 (4.4)	1,040 (4.6)	1,125 (5.0)	1,545 (6.9)	1,690 (7.5)	1,950 (8.7)	2,390 (10.6)
3/8	1-1/2 (38)	1 7/8 (48)	1,435 (6.4)	1,570 (7.0)	1,815 (8.1)	2,220 (9.9)	1,545 (6.9)	1,690 (7.5)	1,950 (8.7)	2,390 (10.6)
	2 (51)	2 1/2 (64)	2,205 (9.8)	2,415 (10.7)	2,790 (12.4)	3,420 (15.2)	2,375 (10.6)	2,605 (11.6)	3,005 (13.4)	3,680 (16.4)
	2-1/2 (64)	3 (76)	2,715 (12.1)	2,895 (12.9)	3,205 (14.3)	3,690 (16.4)	6,640 (29.5)	7,275 (32.4)	8,400 (37.4)	10,290 (45.8)
1/2	1-1/2 (38)	2 (51)	1,610 (7.2)	1,765 (7.9)	2,040 (9.1)	2,495 (11.1)	1,735 (7.7)	1,900 (8.5)	2,195 (9.8)	2,690 (12.0)
	2 (51)	2 1/2 (64)	2,480 (11.0)	2,720 (12.1)	3,140 (14.0)	3,845 (17.1)	2,675 (11.9)	2,930 (13.0)	3,380 (15.0)	4,140 (18.4)
	2-1/2 (64)	3 (76)	3,085 (13.7)	3,375 (15.0)	3,900 (17.3)	4,775 (21.2)	6,640 (29.5)	7,275 (32.4)	8,400 (37.4)	10,290 (45.8)
	3-1/4 (83)	3 3/4 (95)	4,570 (20.3)	5,005 (22.3)	5,780 (25.7)	7,080 (31.5)	9,845 (43.8)	10,785 (48.0)	12,450 (55.4)	15,250 (67.8)

$V_{u1} := 10.75\ \boldsymbol{kip}$

$$R_{bolt} := \frac{V_{u1}}{6} = 1.792\ \boldsymbol{kip}$$

3/8" diameter, 2"
embedment, 4000 psi

$\phi V_n := 3005\ \boldsymbol{lbf}$

$$\frac{R_{bolt}}{\phi V_n} = 0.596 \qquad \textit{good}$$

Bolt Shear Failure Check
10' Opening

6 total bolts

Table 2 — Hilti Carbon Steel KB-TZ2 design strength based on concrete failure modes in uncracked concrete per ACI 318 Ch. 17, applicable for both hammer and core drilled installations [1,2,3,4]

Nominal anchor diameter in.	Effective embedment in. (mm)	Nominal embedment in. (mm)	Tension (lesser of concrete breakout / pullout) - ΦN_n				Shear (lesser of concrete breakout or pryout) - ΦV_n			
			f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.1 MPa) lb (kN)	f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.1 MPa) lb (kN)
1/4	1-1/2 (38)	1 3/4 (44)	945 (4.2)	980 (4.4)	1,040 (4.6)	1,125 (5.0)	1,545 (6.9)	1,690 (7.5)	1,950 (8.7)	2,390 (10.6)
3/8	1-1/2 (38)	1 7/8 (48)	1,435 (6.4)	1,570 (7.0)	1,815 (8.1)	2,220 (9.9)	1,545 (6.9)	1,690 (7.5)	1,950 (8.7)	2,390 (10.6)
	2 (51)	2 1/2 (64)	2,205 (9.8)	2,415 (10.7)	2,790 (12.4)	3,420 (15.2)	2,375 (10.6)	2,605 (11.6)	3,005 (13.4)	3,680 (16.4)
	2-1/2 (64)	3 (76)	2,715 (12.1)	2,895 (12.9)	3,205 (14.3)	3,690 (16.4)	6,640 (29.5)	7,275 (32.4)	8,400 (37.4)	10,290 (45.8)
1/2	1-1/2 (38)	2 (51)	1,610 (7.2)	1,765 (7.9)	2,040 (9.1)	2,495 (11.1)	1,735 (7.7)	1,900 (8.5)	2,195 (9.8)	2,690 (12.0)
	2 (51)	2 1/2 (64)	2,480 (11.0)	2,720 (12.1)	3,140 (14.0)	3,845 (17.1)	2,675 (11.9)	2,930 (13.0)	3,380 (15.0)	4,140 (18.4)
	2-1/2 (64)	3 (76)	3,085 (13.7)	3,375 (15.0)	3,900 (17.3)	4,775 (21.2)	6,640 (29.5)	7,275 (32.4)	8,400 (37.4)	10,290 (45.8)
	3-1/4 (83)	3 3/4 (95)	4,570 (20.3)	5,005 (22.3)	5,780 (25.7)	7,080 (31.5)	9,845 (43.8)	10,785 (48.0)	12,450 (55.4)	15,250 (67.8)

$V_{u2} := 6.23\ \boldsymbol{kip}$

$$R_{bolt} := \frac{V_{u2}}{6} = 1.038\ \boldsymbol{kip}$$

3/8" diameter, 1.5"
embedment, 4000 psi

$\phi V_n := 1950\ \boldsymbol{lbf}$

$$\frac{R_{bolt}}{\phi V_n} = 0.532 \qquad \textit{good}$$

Bolt Shear Failure Check
16' Opening

6 total bolts

Table 2 — Hilti Carbon Steel KB-TZ2 design strength based on concrete failure modes in uncracked concrete per ACI 318 Ch. 17, applicable for both hammer and core drilled installations [1,2,3,4]

Nominal anchor diameter in.	Effective embedment in. (mm)	Nominal embedment in. (mm)	Tension (lesser of concrete breakout / pullout) - ΦN_n				Shear (lesser of concrete breakout or pryout) - ΦV_n			
			f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.1 MPa) lb (kN)	f'_c = 2,500 psi (17.2 MPa) lb (kN)	f'_c = 3,000 psi (20.7 MPa) lb (kN)	f'_c = 4,000 psi (27.6 MPa) lb (kN)	f'_c = 6,000 psi (41.1 MPa) lb (kN)
1/4	1-1/2 (38)	1 3/4 (44)	945 (4.2)	980 (4.4)	1,040 (4.6)	1,125 (5.0)	1,545 (6.9)	1,690 (7.5)	1,960 (8.7)	2,390 (10.6)
3/8	1-1/2 (38)	1 7/8 (48)	1,435 (6.4)	1,570 (7.0)	1,815 (8.1)	2,220 (9.9)	1,545 (6.9)	1,690 (7.5)	1,950 (8.7)	2,390 (10.6)
	2 (51)	2 1/2 (64)	2,205 (9.8)	2,415 (10.7)	2,790 (12.4)	3,420 (15.2)	2,375 (10.6)	2,605 (11.6)	3,005 (13.4)	3,680 (16.4)
	2-1/2 (64)	3 (76)	2,715 (12.1)	2,895 (12.9)	3,205 (14.3)	3,690 (16.4)	6,640 (29.5)	7,275 (32.4)	8,400 (37.4)	10,290 (45.8)
1/2	1-1/2 (38)	2 (51)	1,610 (7.2)	1,765 (7.9)	2,040 (9.1)	2,495 (11.1)	1,735 (7.7)	1,900 (8.5)	2,195 (9.8)	2,690 (12.0)
	2 (51)	2 1/2 (64)	2,480 (11.0)	2,720 (12.1)	3,140 (14.0)	3,845 (17.1)	2,675 (11.9)	2,930 (13.0)	3,380 (15.0)	4,140 (18.4)
	2-1/2 (64)	3 (76)	3,085 (13.7)	3,375 (15.0)	3,900 (17.3)	4,775 (21.2)	6,640 (29.5)	7,275 (32.4)	8,400 (37.4)	10,290 (45.8)
	3-1/4 (83)	3 3/4 (95)	4,570 (20.3)	5,005 (22.3)	5,780 (25.7)	7,080 (31.5)	9,845 (43.8)	10,785 (48.0)	12,450 (55.4)	15,250 (67.8)

$$V_{u3} := 11.34\ \boldsymbol{kip}$$

$$\boxed{R_{bolt}} := \frac{V_{u3}}{6} = 1.89\ \boldsymbol{kip}$$

3/8" diameter, 2" embedment, 4000 psi

$$\boxed{\phi V_n} := 3005\ \boldsymbol{lbf}$$

$$\frac{R_{bolt}}{\phi V_n} = 0.629 \qquad \textit{good}$$

Plate thickness for 15.5' opening

Try a 6x7 in plate

Force transfer

HSS6x4x5/16

$d := 4.625\ \boldsymbol{in} \qquad N := 6\ \boldsymbol{in} \qquad B := 7\ \boldsymbol{in} \qquad b_f := 2.625\ \boldsymbol{in}$

$$P_u := V_{u1} = 10.75\ \boldsymbol{kip}$$

$$m := \frac{N - 0.95\ d}{2} = 0.803\ \boldsymbol{in}$$

$$n := \frac{B - 0.8\ b_f}{2} = 2.45\ \boldsymbol{in}$$

Take the largest out of the three

$$n' := \frac{\sqrt[2]{d \cdot b_f}}{4} = 0.871\ \boldsymbol{in}$$

Available Strength $\qquad F_y := 36\ \boldsymbol{ksi} \qquad$ A36 Steel $\qquad l := \max\left(m, n, n'\right) = 2.45\ \boldsymbol{in}$

$$t_{min} := l \cdot \sqrt[2]{\frac{2\ P_u}{0.9\ F_y \cdot B \cdot N}} = 0.308\ in$$

$$\frac{3}{8}\ in = 0.375\ in$$

$$t_{15.5} := \frac{3}{8}\ in$$

Plate thickness for 10' opening

Try a 6x7 in plate

Force transfer

HSS6x4x1/4

$d := 4.875\ in$ $N := 6\ in$ $B := 7\ in$ $b_f := 2.875\ in$

$P_u := V_{u2} = 6.23\ kip$

$$m := \frac{N - 0.95\ d}{2} = 0.684\ in$$

$$n := \frac{B - 0.8\ b_f}{2} = 2.35\ in$$

Take the largest out of the three

$$n' := \frac{\sqrt[2]{d \cdot b_f}}{4} = 0.936\ in$$

Available Strength $F_y := 36\ ksi$ A36 Steel $l := \max(m, n, n') = 2.35\ in$

$$t_{min} := l \cdot \sqrt[2]{\frac{2\ P_u}{0.9\ F_y \cdot B \cdot N}} = 0.225\ in$$

$$\frac{1}{4}\ in = 0.25\ in$$

$$t_{10} := \frac{1}{4}\ in$$

Plate thickness for 16' opening

Try a 6x7 in plate

Force transfer

HSS6x4x5/16

$d := 4.625\ in$ $\quad N := 6\ in$ $\quad B := 7\ in$ $\quad b_f := 2.625\ in$

$P_u := V_{u3} = 11.34\ kip$

$$m := \frac{N - 0.95\ d}{2} = 0.803\ in$$

$$n := \frac{B - 0.8\ b_f}{2} = 2.45\ in$$

Take the largest out of the three

$$n' := \frac{\sqrt[2]{d \cdot b_f}}{4} = 0.871\ in$$

Available Strength $\quad F_y := 36\ ksi$ $\quad$ A36 Steel $\quad l := \max(m, n, n') = 2.45\ in$

$$t_{min} := l \cdot \sqrt[2]{\frac{2\ P_u}{0.9\ F_y \cdot B \cdot N}} = 0.316\ in$$

$$\frac{3}{8}\ in = 0.375\ in$$

$$t_{16} := \frac{3}{8}\ in$$

Weld Strength

Weld available strength For HSS6x4x5/16

$D := 5\ in$ Weld size in sixteenths of an inch

$d := 4.625\ in$

$b_f := 2.625\ in$

$l := b_f$ Flange weld

$F_{EXX} := 70\ ksi$ E70 Electrode Fill

$$\phi R_{nHSS6x4x.3125} := 0.75 \cdot 0.6\ F_{EXX} \cdot \left(\frac{\sqrt[2]{2}}{2}\right) \cdot \left(\frac{D}{16}\right) \cdot l = 18.272\ kip$$

Require strength

Weld available strength For HSS6x4x1/4

$D := 3\ in$ Weld size in sixteenths of an inch

$d := 4.875\ in$

$b_f := 2.875\ in$

$l := b_f$ Flange weld

$F_{EXX} := 70\ ksi$ E70 Electrode Fill

$$\phi R_{nHSS6x4x.25} := 0.75 \cdot 0.6\ F_{EXX} \cdot \left(\frac{\sqrt[2]{2}}{2}\right) \cdot \left(\frac{D}{16}\right) \cdot l = 12.007\ kip$$

Require strength

Shear at 15.5' Opening Weld

$V_{u1} = 10.75\ kip$ shear at weld

$$\frac{V_{u1}}{\phi R_{nHSS6x4x.3125}} = 0.588$$

good

Shear at 10' Opening Weld

$V_{u2} = 6.23\ \textbf{kip}$ shear at weld

$$\frac{V_{u2}}{\phi R_{nHSS6x4x.25}} = 0.519$$ *good*

Shear at 15.5' Opening Weld

$V_{u3} = 11.34\ \textbf{kip}$ shear at weld

$$\frac{V_{u3}}{\phi R_{nHSS6x4x.3125}} = 0.621$$ *good*