Introduction to the hybrid steel frame with wood floor system

by

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Abstract

The American Institute of Steel Construction (AISC) recently published a new design guide, *Design Guide 37: Hybrid Steel Frames with Wood Floors*. This new design guide intends to shed light onto a new form of construction not previously used in building construction. The new system is made up of a steel frame with wood floors rather than the traditional concrete-on-composite deck floors in steel construction that is popular today. The design methodology for this new system is not new. The steel frame design is completed in accordance with the AISC 360 *Specification for Structural Steel Buildings*, and the wood floor system is designed in accordance with the National Design Specification (NDS) for Wood Construction. Although design methods are not new, this new form of construction may lead to new considerations or nuances within the design that will need to be addressed. Considerations on fire protection, vibration, and potential composite action between the wood and steel are introduced and discussed in this report. A key design aspect that is changed is that the CLT slab acts as a continuous lateral brace to the top flange of the secondary beams, eliminating lateral torsional buckling. To show this new form of construction's capabilities, a design example is completed and compared to that of a standard steel framed building. The comparison intends to illustrate this new system and the benefits that can be achieved by utilizing it in building construction.

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Chapter 1: Introduction

Steel and timber are both construction materials used in industry today. To this point, rarely are they utilized together. This report intends to shed light on a new form of construction, hybrid steel-timber buildings.

1.1 Steel Framed Buildings

Steel has been at the forefront of the design industry since the early 1900s. In the mid-1800s, Sir Henry Bessemer and Sidney Thomas made the steel production process much more efficient and steel production significantly cheaper, allowing steel to become more prevalent in the construction industry. Steel is a superior construction material due to its inherent properties. As a construction material, steel has a high strength-to-weight ratio and ductile behavior that improves steel performance. Due to these inherent traits, steel is a very efficient and an economical choice for building design.

The typical steel-framed building includes a floor system comprising a steel floor deck topped with concrete in order to reach the necessary strength. This floor system is designed as composite, with steel anchors, or shear studs, welded to the steel beams designed to handle the shear forces created between the two materials as composite action occurs. This design practice is one of the most common, if not the most common, forms of commercial construction today. That is due to the well-known design process and proven results that can come from this form of construction.

One downside of this form of construction is that having a composite concrete floor may greatly increase the overall weight of the building. This is due to a thick layer of concrete that naturally weighs more than that of other materials. This can cause problems in areas with high seismicity, as the force that a building may endure during a seismic event is directly related to the overall weight of the building. The steel frame's inherent ductile behavior allows this form of construction to still be widely popular even in high seismic areas. Reducing the overall weight of the building would allow for a more efficient design.

1.2 Mass Timber Buildings

Timber as a construction material has been popular since early human history. This use of timber is due to the natural abundance of timber in the world. The use of timber was also sustainable, since when one tree was harvested another was planted in its place. As the drive for sustainable design becomes more prevalent, the desire to use timber in large construction projects increases. This interest in sustainability led to the development of mass timber design and construction. Mass timber design was developed in Austria and Germany in the early 1990s and has become more popular. The sustainability benefits of mass timber construction are very evident, but the issue to this date is not the performance of timber as a material but rather the cost. The structural properties of wood are like that of steel in many regards, with an even greater strength-to-weight ratio. However, the issue with timber is that the density is very low, so to achieve the required load resistance, a larger member is needed than a steel member. These larger members lead to taller buildings, resulting in higher operational costs and more finishes, which immediately increase the overall cost of the building. To go with this, the material cost of timber at this moment is higher than that of steel or concrete in most parts of the world, so the overall cost of the building is quickly determined to be greater than that of a steel-framed building. Due to this, mass timber construction is typically not the cost-efficient design choice for construction projects.

1.3 Hybrid Steel-Timber Buildings

Hybrid steel-timber buildings are a new, innovative form of construction that combines mass timber and steel construction. The frame of the building, consisting of beams and columns, is constructed of steel, while the floor system is of mass timber. This form of construction takes the benefits of both forms of design. The cost of the building decreases with the use of the steel frame, while the mass timber flooring reduces the overall weight of the building and provides sustainability benefits to the project. A very desirable aspect of this new form of construction is the wide range of projects that can utilize this, as well as the inherent aesthetic that it creates. The mass timber floor panels and the exposed steel frame provide an aesthetically pleasing ceiling finish. This exposed material finish will be presented in the following section in a case study.

A few benefits are observed when comparing the hybrid steel-timber building to a mass timber building. The first is the ability to have long-span beams without utilizing deep timber beams. These long-span beams are due to the steel's ability to span much farther than an equivalent timber beam. These steel beams allow for a broader range of uses for this system than a mass timber building. Because of the much higher strength of steel columns compared to mass timber columns, implementing the hybrid steel-timber system allows for taller buildings than mass timber. This is another key benefit of this system. Mass timber has limitations in design that are much more restrictive than steel design, so utilizing the steel frame avoids these restrictions while still getting some benefit from a mass timber building. One more benefit that comes from utilizing the hybrid system rather than an actual mass timber building has to do with vibration. Steel is stiffer than structural timber; therefore, providing the steel frame reduces some of the vibration endured in a mass timber building (Barber et al., 2022).

There are also benefits of utilizing the hybrid system when compared to steel-framed buildings. The largest of them all being the overall weight of the building. Even with a concrete topping, the timber floor is lighter than that of a composite concrete floor system. This weight reduction will decrease the load on members, allowing lighter members to be used in design. The speed of construction will also increase due to the prefabrication of all aspects of the design, allowing for quick installation once the materials reach the job site (Barber et al., 2022). Hybrid steel-timber structures may become more prevalent in the design industry as their use and the drive for sustainability increase.

1.4 Case Study – Houston Endowment Headquarters

This hybrid steel-timber design is new, but some buildings have already utilized this structural system. One is the new Houston Endowment Headquarters located in Houston, Texas. Arup designed this hybrid steel-timber structural system for the new 40,000-square-foot, two-story office building. The original design plan for this system was to be a concrete structural system. This concrete proposal imposed many structural issues, so a new design was necessary, leading to the hybrid steel-timber design. A key issue with the concrete structural plan was that the site and soil conditions were not good, so the weight of the concrete structure resulted in expensive and large foundations that were not practical. The hybrid system resulted in simple shallow foundations that were much more practical than the previous design. The final design for the new Houston Endowment Headquarters consisted of a steel frame and 3-ply cross-laminated timber (CLT) floor panels that spanned approximately 10 feet. The typical structural bays for this project were 30 ft by 30 ft. As for the lateral system, steel moment frames were used for the vertical system, and the CLT slab was able to be used for the diaphragm or the horizontal system (Barber et al., 2022).

A few of the benefits noticed by the design team for this project included the project schedule, design flexibility, and sustainability. The project schedule was reduced drastically due to the prefabrication of both the steel members and the CLT floor panels. Prefabrication resulted in a much quicker installation on site, which led to this building being completed well before the original design timeline. The steel framing could span much longer than the original concrete frame, allowing for more open atriums and other architectural features that the owner desired. The sustainability effect was drastic for this project when the decision was made to switch to the hybrid system. According to Barber et al., the structure's carbon footprint was reduced by 50% compared to the concrete design. This carbon footprint reduction is significant and highly beneficial for the environment.

The project's results proved to be very aesthetically pleasing and a very adequate structural design. Below is an image of the rendering created for this project.

Figure 1.1. Houston Endowment Headquarters Rendering (Barber et al., 2022)

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As shown in Figure 1.1 above, the steel members and the underside of the CLT panel are fully exposed, creating an aesthetically pleasing space. The exposed frame eliminates the need for expensive ceiling finishes and creates a new look that is not common in the design industry today. This building is an excellent example of why this new form of construction may become much more prevalent soon.

1.5 Report Structure

This report aims to discuss a few of the design aspects of this new structural system and present the design process as well. First, design considerations are discussed. This report includes fire protection, vibration, and composite action for the system. After that, the design process and equations are presented. A comparison is then carried out between a hybrid steel-timber system and a steel-framed building with a concrete composite floor system. The calculations for this comparison are included in this report as an appendix. This report aims to present the potential benefits of a new innovative structural system.

Chapter 2: Design Considerations

This section of the report discusses the design considerations for hybrid systems. Design considerations include topics that affect the system's design and are crucial to address before the design is complete. Topics not included in this report that may also need to be addressed include but are not limited to, sustainability, constructability, and the economics of the system. Design considerations included in this report and detailed in the following sections include the fire protection of the steel and timber elements, vibration considerations, and the possibility of composite action between the steel frame and the CLT floor system.

2.1 Fire Protection

The first design consideration discussed is the fire protection of the members. Fire protection for a hybrid system includes three different elements to consider, steel framing members, CLT, and the interaction between the two materials. There are two types of fire protection: passive and active. Active fire protection systems include fire sprinkler systems and mitigation devices. Active systems will not be discussed in this report. Passive fire protection is an approach to delay the rate of temperature increase to the material to provide time for evacuation of the building and time for the fire to either burn out or be extinguished (American Institute of Steel Construction, n.d.). Passive fire protection will be the focus of this report as it directly relates to this system. Fire protection of buildings is determined based on the occupancy of the building and the importance of the member under consideration. For example, an exterior load-bearing wall will require a higher level of fire resistance rating than an interior partition wall due to the importance of that element being able to resist the fire until the fire is extinguished. The Fire Resistance Rating (FRR) is the measurement system that defines different levels of fire protection. The rating system is measured in hours. A building could require anywhere from 0 to 4 hours, depending on the building. The Fire Resistance Rating was determined through testing outlined in ASTM E119 (Barber et al., 2022), in which different fire protection methods on different materials were tested to determine how long the fire resistance rating can be in certain situations. The following sections will discuss the different considerations for the steel framing members and the CLT floors, the methods to achieve adequate fire protection, and the inherent ability of each material for fire resistance.

2.1.1 Steel Fire Protection

Fire protection of the steel members is crucial to ensure the safety of the building in the event of a fire. According to Design Guide 19 – Fire Resistance of Structural Steel Framing, published by the American Institute of Steel Construction, steel offers the benefit of not being a combustible material. However, steel does experience reductions in material properties when exposed to high temperatures. The yield strength and the modulus of elasticity are reduced when exposed to high temperatures. The yield strength of the steel is approximately 85% of the original value when exposed to temperatures up to 800 degrees Fahrenheit and further reduces to about 20% of the original value at temperatures up to 1300 degrees Fahrenheit. These reductions in yield strength and the modulus of elasticity show that both the strength and stiffness decrease when the temperature increases. Knowing this, even though safety factors are included in the design of steel members and members are typically not fully loaded, it is crucial to avoid exposure to extreme temperatures or else failure could occur during a fire.

The most common fire protection for steel framing systems is spray-applied fire-resistive materials (SFRM). This form of fire protection is applied in the field and is used when the steel framing members are not exposed. SRFM is typically not used on architecturally exposed steel because the spray-on material is not aesthetically pleasing. The SRFM material creates an unsmooth gray coating on the outside of the steel members which is not desirable for exposed members. Therefore, the architects prefer not to expose beams and columns with SRFM applied.

The spray-on fire protection works by applying the material to the entire member. When exposed to higher temperatures, it will expand and insulate the structural steel to protect it from rising temperatures. The amount of spray-applied fire-resistive material applied to the member depends on the fire resistance rating required for the project. This method's important aspect is to ensure the adhesion between the steel member and the spray-applied material. If there is an abundance of dirt, oil, or other materials on the steel when this coating is applied, the adhesion may not form, and the total fire protection rating of the steel member will not be reached (American Institute of Steel Construction, n.d.).

Additionally, there are two types of SFRM: fibrous or cementitious. Fibrous SFRM is from iron slag or melting rocks, forming these materials into wool, which produces a "filamentous mass with lightweight and noncombustible properties." (Ruddy et al., 2003). This filamentous mass is mixed with a binder and water and sprayed onto the steel members. Cementitious SFRM is what it appears to be: SFRM with cement or a gypsum material added in. This form of fire protection provides resistance by releasing the water naturally in the cement/gypsum material as steam. This SFRM form can be applied manually or via a high-pressure spray nozzle (Ruddy et al., 2003). SRFM is just one form of fire protection that could be used on steel in a hybrid steel-timber system.

The second form of fire protection would be using fire-rated gypsum wall boards to encase the steel members. This method is not to be used if the steel is exposed for aesthetic purposes, as the member must be fully boxed in and will not be visible at all.

The gypsum board can provide fire resistance to the steel as gypsum board is typically about 21% water, which will be released as steam when the board is exposed to fire, which slows down the heat transfer to the steel column (What is…, 2019). The time the gypsum board takes to burn through, allowing for the steel temperature to increase, is related to the thickness of the gypsum board. The thicker the gypsum board, the longer the column/beam will be fire-rated. Gypsum board and SFRM are both effective ways to provide fire resistance for steel members in a hybrid system.

The third and final form of passive fire resistance systems is intumescent paint. Intumescent paint is a form of fire protection considered aesthetically pleasing to architects as it allows for the only visual difference on the steel to be a layer of paint. Just because this form of fire resistance is only a layer of paint does not mean it is any less effective than the others. Intumescent paint provides fire resistivity by reacting to high temperatures (approximately 480 degrees Fahrenheit) and expanding in thickness by up to 100 times the original thickness of the coating, which provides insulation to the steel member inside. The thickness of the intumescent paint is dependent on the hourly fire rating that is required and the size of the steel member to which it is applied. Another aspect of intumescent paint is that it can be applied in the field and the fabrication shop. Shipping the members fully fireproofed can expedite construction and save money.

There is a stark difference between the appearance of the fire coating before and after a fire. Before, it was a sleek paint job that was aesthetically pleasing. After the fire, it is a thicker, charred material that prevents the steel member from reaching extreme temperatures and losing material properties. The appearance after a fire is like that of SRFM, but this only occurs once the paint has been heated in a fire and will be retrofitted after a fire and will not have this appearance when the building is in use.

2.1.2 CLT Fire Protection

The CLT slab is another aspect of the hybrid system that must be considered for fire protection. Compared to the steel member, the CLT slab has a much-improved innate ability to resist fire and extreme temperatures. One reason is the difficulty of igniting the slab when exposed to fire. This difficulty is due to the sizeable volume-to-surface area ratio, making the ignition of the slab difficult (Okutu, 2019). Additionally, when the CLT does ignite, the member will steadily burn until it reaches a certain point that it will not burn any longer. This point where it no longer burns is due to the charring of the face of the CLT. This charring ability is CLT members' innate form of fire protection. The ability to "char" allows the CLT slab's inner portion to retain its full ability to resist the loading. The charred part of the slab no longer has any load-bearing capacity, but it allows the member to remain load-bearing and prevent failure.

An aspect of the CLT fire protection that must be considered within the design is the species of wood included in the slab. As is known, a CLT slab comprises a different number of wood laminations that can be of the same or different species. Every lamination that will be exposed to the fire must be considered, and this is where the differing species could allow for a different response to the fire at different levels within the slab. So CLT’s fire resistance is also dependent on the species of timber that is used within the slab. The National Design Specification for Wood Construction (NDS) manual has published equations accounting for the char rate and depth to determine the amount of slab that will remain intact during a fire. The char rate is the rate at which the char forms on the CLT slab's outer face(s), and a nominal value is given as 1.5 in/hr. The equation for cross-laminated timber is different from that of other forms of timber due to the properties of the CLT. The equation to calculate the depth the char reaches within the CLT member depends on many aspects of the slab, including the lamination thickness, the nominal char rate, and the exposure time. All these factors will be defined below, along with the equation from the NDS.

$$a_{char} = n_{lam}h_{lam} + \beta_t(t - (n_{lam}t_{gi}))^{0.813}$$

Where:

$n_{lam} = \frac{t}{t_{gi}} = Number\ of\ charred\ laminations\ (rounded\ to\ lowest\ integer)$

$t_{gi} = (\frac{h_{lam}}{\beta_t})^{1.23} = Time\ for\ char\ to\ reach\ glued\ interface\ (hr)$

$\beta_t = \beta_n = 1.5$ (in./hr.$^{0.813}$)

$h_{lam} = lamination\ thickness$ (in.)

$t = exposure\ time$ (hr.)

$a_{char} = char\ depth$ (in.)

The calculated depth of the charring, a_{char}, is now used to calculate the section properties, such as the area, section modulus, and moment of inertia of the remaining CLT section, from which the load-bearing capacity can be determined. The effective depth of the remaining slab is computed by subtracting the effective char depth, which is $1.2a_{char}$, from original thickness. One aspect of the above equation to keep in mind is that this equation is only for one side of the CLT. So, if the fire is on both sides of the member, the effective char depth must be doubled. Below is an image of a slab that has been fire-tested to show that while the outer layer is charred and not considered, the rest of the slab is unchanged.

Figure 2.1. CLT Panel After Fire Test (Barber et al., 2022)

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This is the most common method to determine if the section is adequate for the required time rating, as the exposure time can be changed to reflect the required rating.

Another option for the fire protection of the CLT slab is pressure impregnation. With this pressure impregnation, fire-resistant chemicals are implemented into the slab, which can provide a benefit for fire protection. The primary way the impregnation of these chemicals helps is that it slows the spread of fire. However, this comes at the sacrifice of other aspects of the design. The chemicals put into the slab decrease the overall strength of the section, so while more of the slab may remain intact after the fire, the load-bearing capacity may not change as much as initially expected. The other aspect that can cause serious problems with using chemicals is that the fasteners connecting the CLT slab to the top flange of the beam can corrode, making the connection of these members no longer adequate for transferring the load. Due to these reasons, it is not common to utilize pressure impregnation for fire resistance of a CLT slab.

2.1.3 Steel Timber Interface

The last aspect of a hybrid steel and timber framing system that needs to be fireproofed is the interface between the steel framing member and the CLT slab. This interface includes the bearing of the CLT onto the steel member and the fasteners that connect the two members. If the steel member is not required to be fire-protected, the steel could impose higher temperatures onto the slab that bears on it. These higher temperatures are due to the steel rapidly gaining heat, which, in turn, will ignite the CLT. A recent study showed that the charring of the wood bearing on steel is consistent with that of the rest of the slab (Malaska et al., 2023). The study also tested CLT on steel with intumescent paint. It showed a significant decrease in charring occurred when steel is fire protected. In all tests performed, the charring never left the first layer of CLT. Additionally, when the steel was protected per a three-hour fire rating, after being exposed to the fire for 60 minutes, no charring occurred to the CLT.

The other aspect of the interface between the steel beam and the CLT slab that must be considered is the fasteners between the elements. Self-tapping screws are driven from the underside of the top flange of the beam into the CLT. With this connection, the screws are then able to transfer heat deeper into the CLT member. The degradation of the CLT around the screw can occur, leading to the strength of the connection reducing significantly (Barber et al., 2023).

Figure 2.2. CLT Degradation Near Fastener (Barber et al., 2022)

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The above figure shows the degradation that can occur near a fastener during a fire. As is seen in the image, there is a significant decrease in the section near the screw compared to the rest of the section that has the usual amount of charring. This decrease in connection strength brings concern for the lateral restraint of the beam top flange. This lateral restraint allows the beam to be continuously braced against lateral-torsional buckling. If the connection is not correctly protected, a fire can cause this lateral restraint to no longer exist, and the beam could buckle and lose strength. Different approaches can be taken to avoid losing this lateral restraint. The first and more practical solution would be to add fire protection to the exposed head of the screw. The fire protection of the screw heads will occur if the fire protection is applied in the field. However, for a shop applied fire protection (intumescent paint), additional fire protection would need to be applied in the field after installation. The other choice would be to increase the amount of fire protection on the entire member to prevent the steel beam and the screws from reaching 325 degrees Fahrenheit, when CLT begins to lose its strength at this temperature. Both options are adequate to prevent this from happening if they meet the fire rating requirements necessary for the project being designed.

2.1.4 Common Solutions

While all these options for fire protection are adequate, as with any system, specific options work best for a steel-timber hybrid system. Because a steel-timber hybrid building normally has exposed CLT ceilings and exposed steel, the most common option for providing fire protection is to use intumescent paint on the steel and a slab that is thick enough to withstand the loads after charring has occurred. The other options can work but will come at the cost of aesthetics and will not expose the slab or steel. In the case study on the Houston Endowment Headquarters, this approach allowed for an aesthetically pleasing result, as can be seen in Figure 1.1 of this report. If the steel beams and columns are not exposed, the more economical and common option is to use an SFRM due to its ease of application. The slab needs to be designed to withstand the loading at the hour rating required. This design will require a few extra calculations defined in the CLT Fire Protection Methods section. After this, the system will be adequately protected for the required fire resistance rating.

2.2 Vibration

The next design consideration for the steel-timber hybrid system is vibration. The topics relating to vibration discussed in this report are the codes and standards for design, human excitations, floor frequencies, vibration considerations, design methods, and acoustics.

2.2.1 Codes/Standards

The International Building Code (IBC), which is the dominant building code in the US, does not have prescribed requirements or methods for the vibration control of structures. Within the IBC, serviceability requirements are outlined for deflection of members, but not vibration. A structure designed with only these considerations could result in a floor system being perceived as “low quality and bouncy” by occupants (Breneman et al., 2023).

Other organizations have published design recommendations for vibration, including the American Institute of Steel Construction (AISC) and WoodWorks – Wood Products Council (WoodWorks), for steel and timber members respectively. WoodWorks published the U.S. Mass Timber Floor Vibration Design Guide (Breneman et al., 2023), recently revised in February of 2023, which is the primary reference for mass timber floor vibration in the United States. Within this design guide, recommendations are made to minimize the effects of vibration as well as prescriptive equations and methods of mitigating vibration issues. AISC Design Guide 11 – Vibrations of Steel-Framed Structural Systems Due to Human Activity (Murray et al., 2016) provides information and design considerations for floor vibrations of steel-framed structures due to human activity. This design guide was for steel structures, however, some of the formulas and recommendations can be used with other materials if precautions are taken.

In addition to these two publications, other international organizations have standards and well recognized studies on vibration. Outside the United States, the Eurocode has standards and guidelines for vibration design of lightweight floor systems such as CLT floor systems. Other sources of standards and guidelines for vibration design of CLT floors include the BS 6472-1, SCI P354, JRC-ECCS Joint Report, and CCIP 016 (Zhang et al., 2023).

The BS 6472-1 is a vibration design guide that is published by the British Standards Institute for human exposure to vibration in buildings. The Steel Construction Institute has published the SCI P354 for floor vibration design. This manual was last revised in February 2009. JRC-ECCS Joint Report was a report prepared for the evolution of the Eurocode 3. In this Joint Report vibration standards were discussed. The last additional standard mentioned is the

CCIP 016, a design guide for footfall assessment of floor vibration published by the UK Concrete Centre.

2.2.2 Human Excitations

The design methods and considerations discussed in this report are about vibrations due to the movement of humans within the structure. Although typically not a safety concern, these human-induced vibrations could lead to bouncy floors, which is an occupant satisfaction problem.

Depending on the speed of the human's movement, whether walking, jogging, or running, the frequency of the excitations will be different. The typical vibration frequency range induced by humans walking within a building are in the table below.

Walking Speed	Walking Frequency (Hz)	Steps Per Minute (SPM)	Potential Occupancies
Very Slow (Uncommon)	1.25	75	Laboratories, surgical theaters
Slow	1.6	95	Bedrooms, hotel rooms
Moderate	1.85	110	Residential living areas, office work areas
Fast	2.1	125	Corridors, shopping malls, airports

Table 2.1. Walking Frequency Table (Breneman et al., 2023)

As seen in the table above, the typical range for walking frequency is between 1.6 and 2.1 Hertz. The speeds are related to the occupancy in which the building is meant to be. From the AISC Design Guide 11, "Very slow walking applies to areas with one or two walkers and limited walking paths; examples are laboratories with fewer than three workers and medical imaging rooms. Slow walking applies to areas with three of four potential walkers and limited walking paths. Moderated walking applies to busy areas with clear walking paths. Fast walking applies to areas with clear walking paths, such as corridors" (Murray et al., 2016). Additionally, the table below shows the additional harmonics of the original walking frequency.

Walking Forcing Frequencies and Dynamic Coefficients		
	Person Walking	
Harmonic i	if_{step}, Hz	α_i
1	1.6-2.2	0.5
2	3.2-4.4	0.2
3	4.8-6.6	0.1
4	6.4-8.8	0.05

Table 2.2. Walking Forcing Frequencies to the Fourth Harmonic (Murray et al., 2016)

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In the table above, the frequency is extrapolated to the fourth harmonic. This extrapolation shows that all these frequencies could result in a resonant case if they align with the floor system frequency, which creates a much larger vibration response on the floor.

An additional scenario that may need to be considered is human running. When necessary, a frequency of up to 4 hertz should be considered (Breneman et al., 2023). Examples for which this may need to be considered include gym floors, emergency rooms, or any occupancy in which running may frequently occur.

Additional considerations that should be considered are the average weight of the humans walking and the lengths of the walking strides. The average weight of a walker is 168 pounds according to the U.S. Mass Timber Vibration Design Guide, however, there is not a universally agreed upon value. The force on the floor is proportional to the static weight of the walker. Stride length must be considered to understand how long the loading may be occurring. The stride length directly affects the number of steps taken over a floor length, which in turn influences whether the floor reaches a fully resonant steady-state response with maximum vibration amplitude (Breneman et al., 2023).

The last topic to consider for human excitation is that of the resonant loading function compared to that of the transient loading function. The resonant loading function and its response is much more difficult to predict, as this accounts for multiple steps. With the resonant loading function, the function is formed based on a sharp increase when the heel contacts the ground, followed by a relaxed period until the foot pushes off the floor where a slight increase occurs. This forcing function is shown in the figure below.

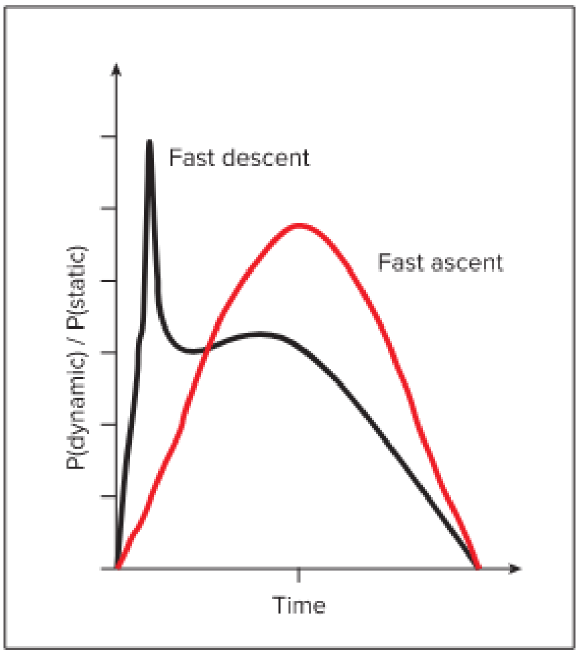

Figure 2.3. Dynamic Loading Function – Walking (Breneman et al., 2023)

The function shown in black refers to the previously described resonant function. This information is outlined in the U.S. Mass Timber Vibration Design Guide and references a study completed by Kerr in 1998. The red line is a simplified approach to this, with the total area of the sinusoidal function equivalent to that of the former. The transient vibration is the vibration that occurs due to the force of a single step. This is much easier to calculate as it is a singular impulse that excites the floor. This allows for the dynamic response of the floor system to be calculated directly from the impulse. (Breneman et al., 2023).

All the above information can be considered for a singular system to optimize the design. The optimization is done based on design methods as well as the frequency of the floor system, which will be discussed next.

2.2.3 Floor Frequencies

The fundamental frequency of the floor is crucial when designing a floor system for vibration. This frequency will determine whether resonant or transient vibration occurs. The goal of the design is to avoid resonance if possible. Resonance is the phenomenon that occurs when the fundamental frequency of the floor system is close to or equal to the excitation frequency (walking frequency). The floor systems are divided into two categories: low-frequency and high-frequency floors. The cutoff point between a low and high-frequency floor system is typically around 8-10 hertz due to the typical values for the walking frequency. Assuming a standard walking frequency of about 2 hertz and extrapolating to the fourth harmonic at 8 hertz, a resonant case may occur with these vibrations if the floor system is around 8-10 hertz. Beyond the fourth harmonic, the vibration will be damped out between excitations (Breneman et al., 2023).

A low-frequency floor has a frequency equal to or less than 8-10 hertz. Low-frequency floors tend to result in resonance, with the vibration building up more severely as the floor frequency decreases. The magnitude of resonance is directly proportional to the damping that is present, therefore the vibration response of low-frequency floors is highly dependent on the amount of damping (Breneman et al., 2023).

High-frequency floors have a fundamental frequency greater than the fourth harmonic of the excitation frequency resulting in the transient vibration being the governing form of vibration. This is due to the vibration having dissipated or damped out between steps. Compared to a low-frequency floor, the magnitude of the vibration response is related to the damping, but to a smaller level (Breneman et al., 2023). High-frequency floor systems are desirable for the design of floor vibration.

Since the fundamental frequency of a floor system is an important parameter, it is crucial to understand what factors can affect the fundamental frequency of the floor system. In a study conducted by Karampour et al., an equation for the fundamental frequency of the floor system is introduced below.

$$f_n = \frac{k_n}{2\pi}\sqrt{\frac{EI}{mL^4}} \quad ---- \quad (1)$$

In Equation 1, "EI" represents the flexural stiffness, "m" represents the distribution of mass, "L" is the length, and k_n is the boundary condition factor. The boundary condition factor will vary depending on the system and the connections of the floor system. The two most common boundary conditions are simply supported and fixed conditions having a boundary condition factor of 9.87 and 22.4, respectively (Karampour et al., 2023). The fundamental frequency that is of interest for vibration design is that of the first mode, which can also be related to the mid-span deflection (δ) of the floor, shown in equation 2 below.

$$f_n = \frac{17.8}{\sqrt{\delta}} \quad ---- \quad (2)$$

The relation shown in Equation 2 is dependent on the mid-span deflection being calculated based on the self-weight of the floor system only and uses a beam analogy for the floor system (Karampour et al., 2023).

The fundamental frequency of the floor system is a crucial part of vibration design. Several factors will affect the frequency of the floor system as well as the maximum amplitude of the vibrations, which are important to consider within the design.

2.2.4 Vibration Considerations

The design for vibration relies heavily on the frequency of the floor system compared to the excitation frequency, but there are other factors as well. These factors either affect the amplitude of vibration or the fundamental frequency. The factors that will be discussed in this section are the mass, damping, and stiffness of the system.

The addition of mass to a system is considered to reduce the amplitude of vibrations, but this must be done cautiously. Adding mass can reduce the vibration amplitude as the system's acceleration is reduced. Newton's Second Law states that a force is equal to mass times acceleration, or acceleration is equal to force divided by mass. Therefore, as the mass increases while the force remains constant, the acceleration decreases resulting in a decreased amplitude of vibration. While adding mass can reduce the amplitude of a singular vibration or a transient vibration, it will also decrease the natural frequency of the floor system. Therefore, adding mass can be beneficial if the natural frequency of the floor system remains at a value that is not going to result in a resonant case with the excitation force.

Not all loads in a system should be considered as mass in vibration design. In addition to the self-weight of structural members, AISC Design Guide 11 recommends an addition of 4 pounds per square foot may be added to account for normal finishes and mechanical equipment (Murray et al., 2016). This recommendation is based on the vibration design for a steel structural system, so for a timber floor, a project-specific addition of dead weight may be considered (Breneman et al., 2023). Additionally, a portion of the live load in certain occupancies can be added. The table below provided in the U.S. Mass Timber Vibration Design Guide shows approximated live loads based on occupancy.

Occupancy	Approximate Expected Live Load (PSF)
Paper Office	11
Electronic Office	8
Residence	6
Assembly Area	0
Shopping Mall	0

Table 2.3. Live Load Recommendations for Vibration Design (Breneman et al., 2023)

The above table does not account for all occupancies, so in those cases, it is acceptable to take 10 percent of the live load for vibration analysis in these cases (Murray et al., 2016).

The next factor is damping, which affects the amplitude of the vibrations. Damping is the amount of energy dissipation occurring in a system and is directly related to the magnitude of the vibrations. As the amount of damping increases, more energy is dissipated resulting in a smaller magnitude of vibration. The amount of damping depends on construction materials, finishes, furniture, and other factors. In the table below, typical damping ranges are provided based on experimental data and research (Breneman et al., 2023).

Category	Range of Damping ζ (% critical)	Discussion
Lightly Damped	1-2%	The lower end includes bare floors without topping and with minimal furnishing. The higher ne dincludes floors with concrete topping and furnishings.
Moderately Damped	2-4%	Lower values include bare timber-concrete composite floors, or timber floors with a floating concrete layer and full furnishings. The higher calues include floors with floating floor layers, raised floors, full furnishings nad mechancial systems. Floors with both furnishings and permanent partitions, not otherwise accounted for, could also be represented at the higher end of the damping range.
Heavily Damped	4-5%	Floors in this range represnte the upper limit of inherent damping. These floors likely include floating toppings, raised floors, suspended ceilings, furnishings, fixtures and/or permanent partitions not otherwise taken into account.
Explicit Damping Control	5%+	Generally, mass timber floors do not have more than 5% damping unless explicit damping control (e.g. a tuned mass damper) is added. These systems are beyond the scope of this guide.

Table 2.4. Typical Damping Values (Breneman et al., 2023)

These values are approximated based on past research, and judgment should be used based on each specific project to determine the appropriate amount of damping present in a building. In the study conducted by Karampour et al., a conservative suggestion for damping in a building should be taken as 1 percent (Karampour et al., 2023), which agrees with the values in the above table as 1 percent is the lower limit of the lightly damped category. The effect of damping in a system that is at resonance is displayed in the figure below.

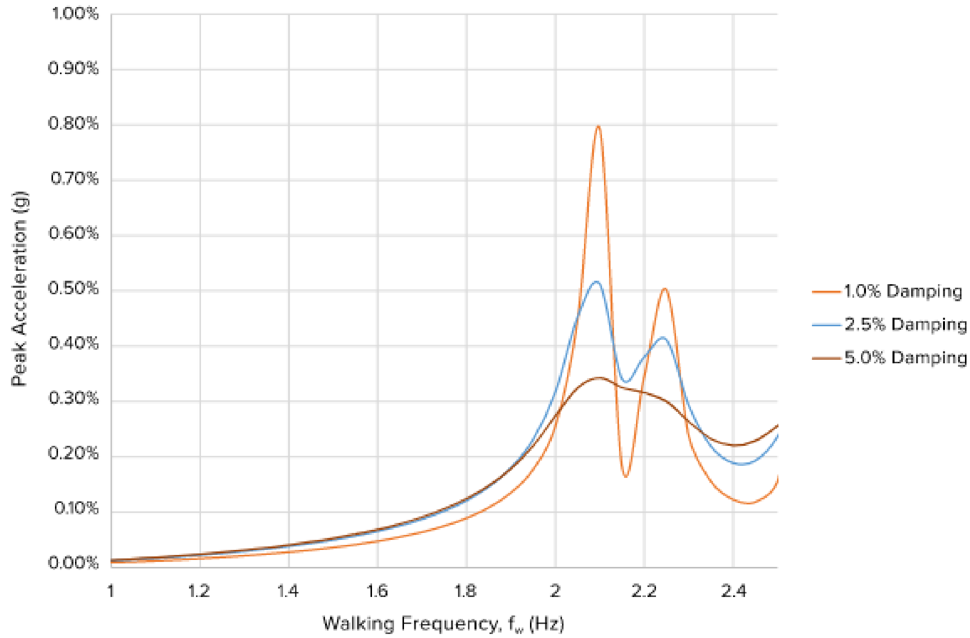

Figure 2.4. Resonant Acceleration at Different Damping Values (Breneman et al., 2023)

As is noticeable in the above figure, as damping increases the value of acceleration decreases. Damping plays a very critical role in vibration design and must be carefully considered in design.

Stiffness is the last element of vibration discussed in this report. When the stiffness increases, so does the vibration frequency. The orientation of the cross-laminated timber (CLT) panel is crucial to consider when calculating the axial stiffness of the floor panel. This is due to timber being an anisotropic material, which means that the material properties are different in different directions. However, these effects can be neglected for bending stiffness, and a general stiffness equation is given for the CLT panel in the National Design Specification for Wood Construction with Commentary (NDS) section 10.4.

$$EI_{app} = \frac{(EI)_{eff}}{1 + \frac{K_s (EI)_{eff}}{GA_{eff} L^2}} \qquad ----(3)$$

Where:

E = Elastic modulus of elasticity

I = Moment of inertia

G = Shear Modulus

A = Cross-sectional area

L = Length

K_s = Shear deformation adjustment factor (NDS Table 10.4.1.1)

This value can then be used in calculations for vibration and can also be an accurate estimate of the amount of vibration that may be prevalent in the structure based on previous experience.

These factors all influence the design of a steel-timber hybrid system for vibration design. The focus is on the CLT floor panel as the vibration will be occurring within the floor spans, therefore the CLT is what will be analyzed, and these factors are from the CLT panel for calculations of vibration.

2.2.5 Design Methods

Vibration design for steel-timber hybrid systems may be the governing case for the floor design. Different methods exist to determine whether additional design needs to be done to minimize vibrations. In this section, the most common methods for vibration analysis will be presented.

The first method for vibration analysis is a rule of thumb based on the fundamental frequency of the floor system. According to the AISC Design Guide 11, all floors with a frequency of less than 3 hertz should be avoided. This is due to the unavoidable resonance with a system having a frequency as low as 3 hertz. Additionally, in a report conducted by Dolan et al. (1999) cited in the U.S. Mass Timber Vibration Design Guide (2023), if the fundamental frequency of the floor system is above 14 hertz, vibration will not be a concern in occupancies such as residences and office spaces. This corroborates what is presented in Design Guide 11 stating there will not be serviceability issues related to vibration if the fundamental frequency is greater than 15 hertz (Murray et al., 2016). The area between 3 and 15 hertz is considered a transitional zone for vibration serviceability and additional analysis must be done. Equation 1 in this report can be used to calculate the fundamental frequency. From Equation 1, a generic form of this equation for a simple span is rewritten below.

$$f_n = \frac{\pi}{2L^2}\sqrt{\frac{EI}{m}} \quad ---- \quad (4)$$

The fundamental frequency of the floor system can be compared to the boundary limits to determine if additional analysis needs to be conducted.

A simplified method of analysis for vibration is to follow the span limits published in the U.S. CLT Handbook and the Canadian CLT Handbook. These span limits were developed based on the CLT floor panels resting on bearing walls. With this, these limits are not directly applicable to a steel-timber hybrid system presented in this report. This is due to these limits not accounting for any flexibility that may be present in the steel frame compared to a rigid bearing wall in a mass timber design. Due to this, these span limits can be used cautiously when designing a hybrid system, and judgment is required by the engineer of record on if the result will be adequate. The span limit presented relies on factors such as the bending stiffness, specific gravity, and cross-sectional area of a 1-foot-wide strip of the CLT panel in question.

$$L_{lim} \leq \frac{1}{12.05}\frac{(EI_{eff})^{0.293}}{(\bar{\rho}A)^{0.122}} \quad (ft) \quad ---- \quad (5)$$

Equation 5 (Karacabeyli and Douglas, 2013) can be used to get an approximate span length based on the inherent properties of the CLT panel used in the design. These span limits were determined based on 5-ply and 7-ply CLT panels, which corresponds well to a steel-timber hybrid system as these depths are commonly used for these designs. For 9-ply and 3-ply CLT panels, the span limits may be overly conservative and unconservative, respectively. Information is provided for CLT floor systems that have a concrete topping. If the topping is less than double the weight of the CLT panel, Equation 5 may be directly used. However, if the concrete topping weight is greater than double the weight, then the span limit must be reduced by 10 percent (Breneman et al., 2023).

The modal response analysis method depicts the vibration of a steel-timber hybrid system more accurately. This method requires assumptions to be made, much more details on the design, and a finite element analysis model. The method uses the principle of superposition of the different modal shapes of the floor. With this approach, the design is much more adaptable to unique designs rather than an empirical equation. The finite element model needs to be defined accurately, otherwise the results are not reliable. More information on modal vibration analysis for specific cases and the post-processing methods are in chapters 4 and 6, respectively, of the U.S. Mass Timber Vibration Design Guide (2023).

The last method mentioned in this report for vibration design is response time history analysis. Response time history analysis is the most accurate form of vibration analysis but is also the most difficult to perform correctly. This difficulty is due to the number of distinct factors considered and the judgment that must be employed by the engineer. This method relies on human excitations being modeled along the length of the floor so that floor vibration can be analyzed at different points of interest. This modeling is difficult to do as estimations must be made for walking paths, forcing functions, and response points. The forcing functions shown earlier in this report are not simple and can create a significant margin for error if an equation does not accurately depict the force the floor will endure. Response points that are implemented must be carefully chosen, and engineering judgment is very crucial. The locations of these points will determine how accurate the information is and whether this is the worst case that occurs within a specific design. Due to these estimations and difficulties, response time history analysis is typically not the first method of analysis chosen if another method can produce accurate results.

2.2.6 Acoustics

Acoustics is another form of vibration that can lead to dissatisfaction from occupants. There is no correlation between acoustics and occupants' safety, but most occupants will not be pleased if there is no sound separation between adjacent areas in a building. A difficult aspect of acoustics with any design is that it is very subjective. Being subjective, no design will be perfect, but there are limits to try and minimize acoustical issues within buildings. These limits are in the International Building Code sections 1206.2 and 1206.3, stating that walls, partitions, and floor-ceiling assemblies should have a sound transmission class and impact insulation class rating of at least fifty (ICC, 2021). These requirements are for walls, partitions, and floor-ceiling assemblies separating dwelling units from each other or other public areas. With this, these requirements may not be completely sufficient for every design, but they are a good starting point and applicable to many occupancies. Sound transmission and impact insulation classes will be discussed in the following paragraphs, as well as design-specific acoustical issues related to steel-timber hybrid systems and potential remedies to these issues.

To understand the two rating classes, sound transmission and impact insulation (isolation), it is crucial to understand sound separation. There are two forms of sound separation, each relating to a singular rating class. The first form of sound separation is airborne sound separation. Sound separation measures the amount of sound transmitted through the air into adjacent spaces through walls, floors, or other partitions. Airborne sound separation issues correlate to the sound transmission class, and when the partitions are not adequately designed, sound may travel from adjacent rooms, such as other occupants talking. Airborne sound separation is measured by comparing the amount of sound lost within the room to standard sound transmission class curves defined in ASTM E413 (Barber et al., 2022). The measurements for sound loss shall be done per ASTM E90. Impact sound transmission is the sound separation relating to the impact insulation class. This is the ability to hear noises such as people walking in a room above you. Impact sound transmission is from floor to floor, with the issues arising from the floors underneath the noise. The impact insulation class rating is formed by measuring the amount of noise that is created by a "tapping machine" in the room above and comparing to the amount that is perceived in the space below, and then compared to standard curves in ASTM E989 (Barber et al., 2022). The figure below visualizes the differences between these forms of sound separation.

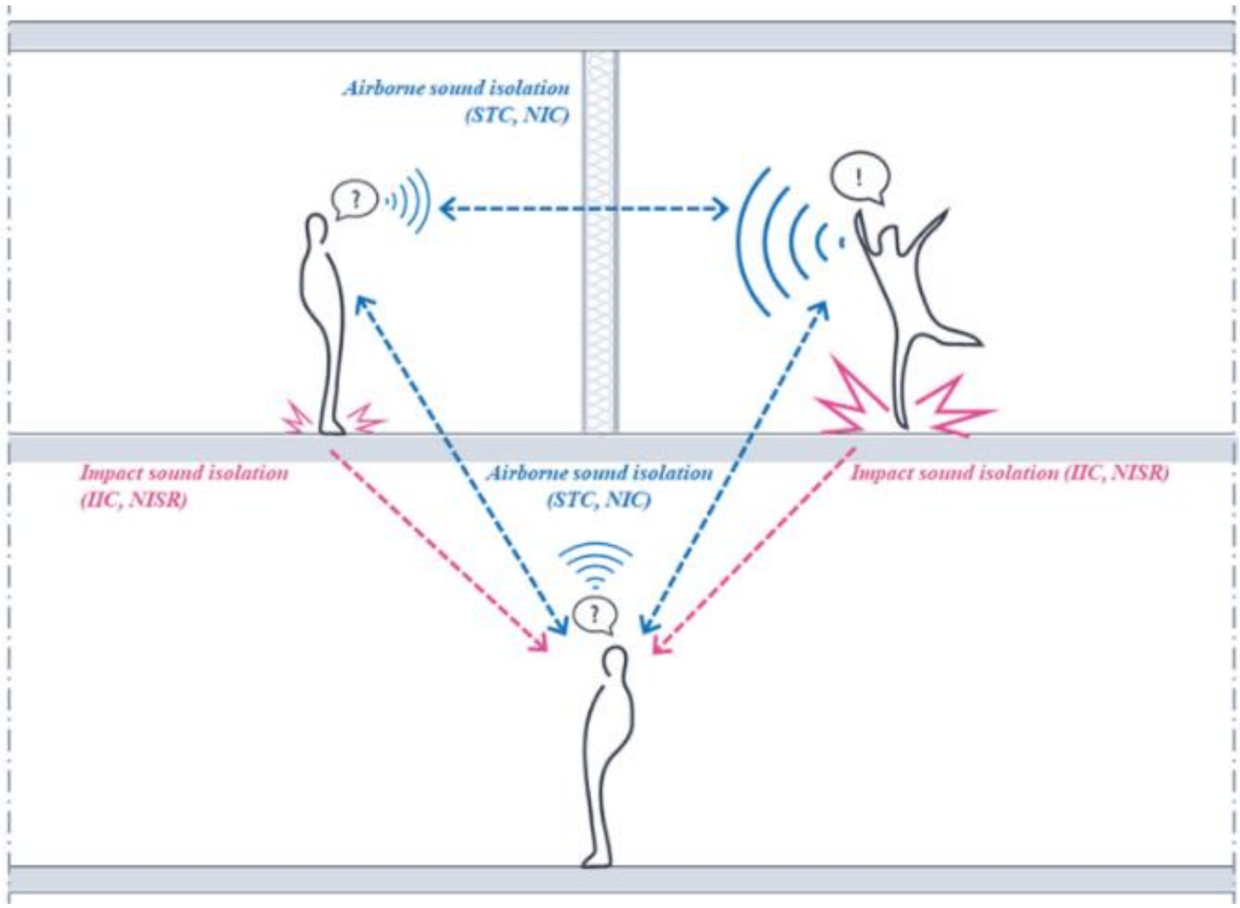

Figure 2.5. Visual Representation of Sound Separation (Barber et al., 2022)

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These rating systems can be useful, but engineers must be mindful that they are generalized measurements that may not be accurate in the field. These ratings only account for sixty percent of audible frequencies, so certain noises are excluded from these ratings. Additionally, they do not account for flanking paths. Flanking paths will be defined in depth below, but it is estimated that actual field ratings for sound transmission class and impact insulation class can be 5-8 points lower than the laboratory ratings.

Flanking paths are areas in a building that can allow for sound transmission from space to space around partitions. The goal of design is to avoid having flanking paths, but there typically are flanking paths, including gaps under doors, penetrations, and junctions between the CLT slab and partitions (Barber et al., 2022). Additionally, flanking paths can form based on the connection made in CLT. These can be screws, angle brackets, hold downs, or other forms of connections, some of which will create a tighter connection resulting in less of a flanking path for sound transmission (Guigou Carter, 2023). Within the design of a steel-timber hybrid system, it is important to identify certain areas that may be at risk for sound transmission and implement specific connection details to minimize issues.

In general, CLT floor systems are not the ideal material for sound isolation, therefore needing additional consideration into the acoustics. The need for additional considerations is due to timber floors being lightweight, and therefore more sound, especially impacts, will transmit to rooms below. The typical acoustical ratings for a 5-ply CLT slab are approximately 40 and 20 for sound transmission and impact insulation classes, respectively (Barber et al., 2022). Comparing these to the required levels from the international building code, which was 50 for both, additional design is necessary. There are three typical options to improve the acoustical performance of a CLT floor in a steel-timber hybrid system: add mass, add airspace, or add resilience. Adding mass is based on the "mass law," which states that for every doubling of the mass, 6 decibels of additional airborne sound isolation is achieved. Most steel-timber hybrid systems already have a concrete topping or other form of topping on the CLT slab; therefore, this is typically already included in the design. The one key factor to consider is that if the topping increases for acoustical reasons, it will induce additional load onto the structure. Adding airspace is another way to improve the acoustical performance of this system. Using "double-leaf" partitions, or mass-air-mass partitions, is a typical method to add airspace. This form of partition will increase the sound transmission class rating. One item to note is that the mass law mentioned previously is for “single leaf” partitions, therefore the effects of both are not completely additive in design. The last design concept to improve acoustical performance is adding resilience. Adding resilience is done by adding acoustical layers into the floor system that will significantly increase the impact insulation class rating. The increase in rating is related to the thickness of the layer, so as the layer gets thicker, the rating gets higher. Floor finishes such as carpets can also be considered a resilient topping that will aid in impact sound transmission (Barber et al., 2022). These concepts can all be applied to improve the acoustical performance of a CLT floor system within a hybrid structure. In the figure below, ratings are provided for some typical floor assemblies.

Sketch and Short Description	STC Rating	IIC Rating
Bare CLT 5 ply (175 mm)	42	26
38mm (1½") precast concrete slab on 13 mm (½") rubber membrane placed on top of a CLT 5 ply (175mm)	56	48
10 mm floating engineered wood floor placed on a 2mm closed cell foam on 38mm (1½") precast concrete slab on 13 mm (½") rubber membrane placed on top of a CLT 5 ply (175mm)	55	51
38mm (1½") precast concrete slab on 10 mm tar boards placed on top of a CLT 5 ply (175mm)	54	36
10 mm floating engineered wood floor placed on a 2mm closed cell foam on 38mm (1½") precast concrete slab on 10 mm tar boards placed on top of a CLT 5 ply (175mm)	53	47
38mm (1½") precast concrete slab on 9 mm closed cell foam placed on top of a CLT 5 ply (175mm)	54	39
10 mm floating engineered wood floor placed on a 2mm closed cell foam on 38mm (1½") precast concrete slab on 9 mm closed cell foam placed on top of a CLT 5 ply (175mm)	52	48
38mm (1½") precast concrete slab on 9 mm closed cell foam placed on top of a CLT 5 ply (175mm) with one layer of 16 mm Type X gypsum board installed on Z channels	70	56

Figure 2.6. Typical Floor Assemblies Acoustical Performance (Barber et al., 2022)

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The figure above shows floor assemblies evaluated and presented in AISC Design Guide 37 Hybrid Steel Frames with Wood Floors. As is evident in the figure above, very few assemblies reached the required rating of 50 for both ratings. This proves that CLT floor assemblies typically do not have adequate acoustical performance, and additional considerations are necessary.

2.3 Composite Action

In current practice, there is no composite action assumed for the steel-timber hybrid system so that the steel beams are designed as non-composite. Studies are looking into the composite action between the timber and steel. Composite action between the steel frame and CLT slab requires nonslip bonding between the two materials. This nonslip bonding must provide enough shear strength through the fasteners (screws, nails, bolts, etc.) between the two materials. Composite action is defined as binding at least two different materials together as a single section for improved structural performance. The horizontal shear stress that forms between the two materials is displayed in the figure below.

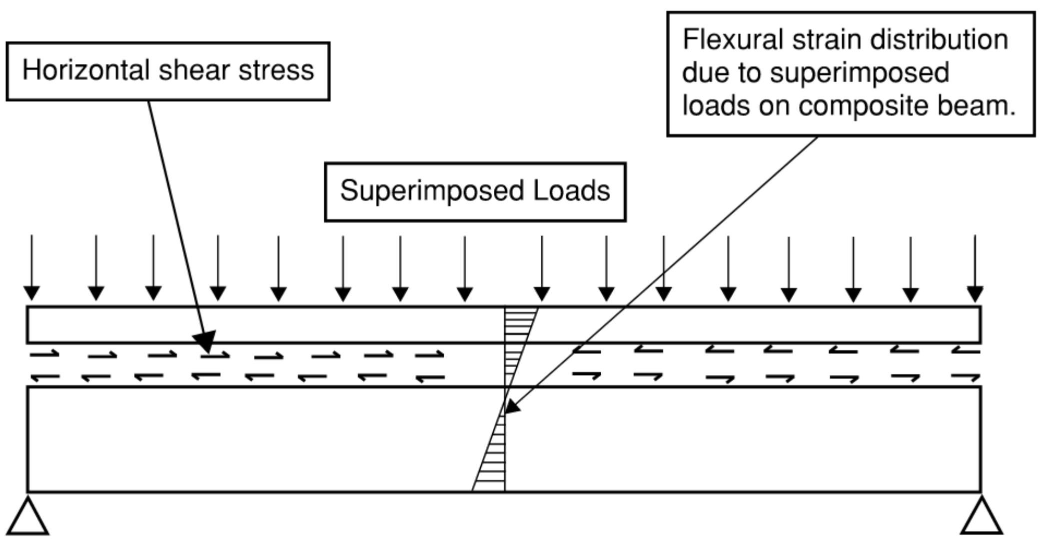

Figure 2.7. Shear Force Development During Composite Action

The shear stresses shown above between the two materials must be managed by the fasteners that create the composite action. By creating this composite action between the steel frame and CLT slab, the improvement in structural performance could bring better material efficiency, increased load-bearing capacity, and reduced weight and cost of the structure (Aspila et al., 2022). How this composite action can be formed for a steel-timber hybrid system, as well as its uses/benefits, will be discussed in this report.

2.3.1 How Composite Action is Formed

Two methods can be used for a steel-timber hybrid system to create the composite action between the CLT floor slab and steel framing members. The first option is already standard in design today but differs slightly. It is common for CLT slabs to have a concrete topping for acoustical and vibration reasons. With the proper detailing and design, it is possible to utilize this concrete topping compositely with the steel member, just as if it were a concrete floor. The modification must be made because there is only a certain amount of concrete that is truly in contact with the framing member, and this must be accounted for in the calculations. The concrete and the CLT slab inherently form a composite bond between the two members due to how the concrete cures so a composite section between the timber, concrete, and steel is formed via the provided shear studs. This form of composite action can be very reliable when designed correctly and an adequate amount of shear studs are provided to handle the required shear, as shown in the figure below.

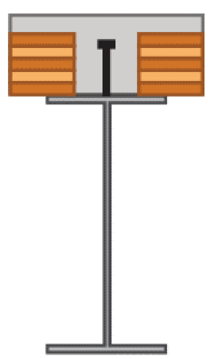

Figure 2.8. Steel-Concrete-Timber Composite Section (Barber et. al, 2022)

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It must be noted that the composite action is not truly formed between the steel and the timber but rather between the concrete topping and the steel. The concrete topping and the timber element do have composite behavior between the two materials due to the curing of the concrete. However, the strength benefit needs to be better understood. The second method to form the composite action between wood and steel is the connection between the CLT slab and the top flange of the beams. The shear strength and stiffness of the connection are not well understood to this date, and there are no design guidelines for the composite action with this type of construction. Therefore, this is not a very commonly applied structural design method. The following figure shows a section of this connection.

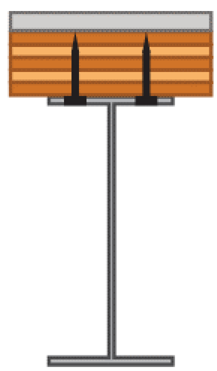

Figure 2.9. Steel-Timber Composite Section (Barber et. al, 2022)

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The connection that is being shown in Figure 2.9 is for if this composite action is being formed between the steel and the timber. The composite action relies on the shear strength of the connections between two materials. The closer the spacing of the connectors, the more composite action would be formed. Another aspect to consider for this form of composite action is the strength deterioration under fire. In a fire, the CLT member chars and loses some of the connection capacity. If there is a drastic decrease in connection capacity between the two materials, it could lose composite action, hence, a large amount of strength.

2.3.2 Potential Benefits

While it is known that the composite action between the floor system and steel framing members can increase strength capacity, it is not commonly used for a steel-timber hybrid system. This lack of use is due to a lack of understanding of the behavior between the two elements and a lack of research into this specific topic for the hybrid steel-timber system. Currently, it is most common to design the two members separately and to design the connection between them strictly for the load it carries. The benefits of the composite action are a strength increase, a reduction in member size, which reduces the building weight, and an increase in material efficiency. As the drive for sustainable design continues within the structural design and construction industry, composite action for this hybrid system may become more desirable, and more research needs to be done so that it can be used in design.

Chapter 3: Structural Design

Chapter 3 of this report is intended to give insight into the design process of the steel-timber hybrid structural system. The differences between this new system and a typical steel-framed building or a mass timber floor building are discussed, in addition to any other nuances with the design of the steel-timber hybrid system. Following the introduction of design procedures, a design example is presented. A typical bay in a building is designed using both a steel-timber hybrid system and a steel frame with a composite concrete floor system. Comparisons between the two designs are made and presented to show the differences.

3.1 Design Process

There is nothing new to structural engineers in designing a steel-timber hybrid structural system. Both materials in the system, namely, structural steel and timber, are designed according to their material specifications: the AISC Specification for Structural Steel Buildings for the steel frame and NDS Chapter 10 for the CLT slab. However, the steel frame and the CLT slab affect each other within the design, and there can be nuances specific to this form of structures, which will be discussed below, along with limit states and equations being presented.

3.1.1 Steel Frame Design

The steel framing consists of columns and beams supporting the CLT floor system. While it is known that steel can span long distances, it is also essential to consider a layout conducive to the CLT floor system. As mentioned above, the steel framing members are designed per the AISC Specification for Structural Steel Buildings. Generally, as outlined in Chapter 2, composite action is neglected. Steel beams are designed as being simply supported and checked for shear, flexure, and deflection with no composite action.

The shear strength of the beam is determined based on the member's section properties and is calculated using equation G2-1 from the AISC Specification.

$$V_n = 0.6F_y A_w C_{v1} \quad ---- \quad \text{AISC Spec. Eq. G2} - 1$$

The design shear strength is adequate when:

$$\emptyset_v V_n \geq V_u$$

Where:

$\emptyset_v = 1$ and $C_{v1} = 1$ When $\frac{h}{t_w} \leq 2.24\sqrt{\frac{E}{F_y}}$

$\emptyset_v = 0.9$ for all other cases

Most W-shapes with F_y = 50 ksi meet the criterion above, hence, $\emptyset_v = 1$ and $C_{v1} = 1$

Flexural strength is typically more critical than shear strength for steel beams. The CLT slab bearing on the beams can provide bracing for the top flange that is in compression. Therefore, the limit state of lateral-torsional buckling does not need to be considered. This bracing is possible due to the significant in plane lateral stiffness and strength of the CLT slab and the fact that the connection between the slab and the steel framing members can transfer the bracing force. Self-tapping screws are typically used for this connection and are driven from the bottom of the top flange of the beam into the CLT slab. Without lateral-torsional buckling, the only flexural limit states that must be checked are yielding and local buckling (Sections F2 and F3 in AISC Specification). Flexural strength is deemed adequate if

$$\emptyset_b M_n \geq M_u$$

Where:

$\emptyset_b = 0.9$

The flexural design of the steel beams in this design is very straightforward when considering the top flange braced by the CLT slab. Engineering judgment should be used on whether the top flange is fully braced, but for this report, it will be considered fully braced.

Deflection is another limit state that must be checked for the design of the steel beams. This serviceability check is normally done per Table 1604.3 of the International Building Code (ICC, 2021). The deflection limits that are checked for the steel beams are L/360 for live load and L/240 for the total load, where L is the length of the beam. Maximum deflection of a simply supported beam occurs at the midspan of the beam; and the equations for deflections are presented below.

$$\Delta_{LL} = \frac{5w_L l^4}{384EI} < \frac{l}{360}$$

$$\Delta_{TL} = \frac{5(w_D + w_L)l^4}{384EI} < \frac{l}{240}$$

Where:

w_D = Dead Load per Lineal Foot

w_L = Live Load per Lineal Foot

l = Length of Beam

E = Modulus of elasticity of steel = 29,000 KSI

I = Moment of Inertia about X-Axis

The calculated maximum deflections should be within the limits stated above. One other consideration within deflection is the use of camber. In typical steel framed buildings with composite slab, a camber can be added to a steel beam to compensate dead load deflection. but camber should be avoided for a hybrid steel-timber system. When camber is utilized, construction issues can occur when CLT slabs are not able to be installed on a leveled support.

The columns are designed the same way as those in a typical steel framed building, and per the AISC Specification Chapter E. The hybrid steel-timber system does not affect the design of the columns.

3.1.2 CLT Floor Design

CLT is the most used wood product for floor slabs due to its higher in-plane shear strength and stiffness from the cross laminations (Barber et al., 2022). Other options for floor slabs include nail laminated timber (NLT) and dowel laminated timber (DLT), which are not focused on in this report. With the higher strength and stiffness, the CLT panel can typically be used as the diaphragm for the building, which is more efficient than adding plywood sheathing or increased depth of a concrete topping for the diaphragm. As mentioned above, it is important to consider the CLT panel sizes when laying out the structural system to minimize waste. CLT can come in various sizes, but the standard sizing is typically between 2 and 10 feet wide and up to 60 feet long (Cross Laminated Timber (CLT), n.d.). CLT panels are typically designed as one-way spanning elements due to the minimal weak axis strength and the difficulty of providing moment continuity at the splices between the panels. CLT panel slabs are designed according to the National Design Specification for Wood Construction (NDS) Chapter 10. The CLT slab's fire and vibration design considerations have already been presented in Chapter 2 and will not be repeated here.

For the design of the CLT slab, many factors are used in different limit states. These factors account for different adjustments for service conditions of the CLT slab. Below is the list of these factors and their typical value. All the values are obtained from NDS.

Load Duration Factor, $C_D = 0.9$ (Building contains dead load, NDS Table 2.3.2)

Wet Service Factor, C_M = 1.0 (Assumed moisture content below 16%, NDS Sect. 10.3.3)

Temperature Factor, C_t = 1.0 (Assumed temperature of less than 100ºF, NDS Table 2.3.3)

Beam Stability Factor, C_L = 1.0 (Depth does not exceed the breadth, NDS Sect. 3.3.3)

The next list of factors are specific to load resistance factor design, or LRFD.

Format Conversion Factor, K_F = 2.54 for Bending (NDS Table 10.3-1)

K_F = 2.0 for Shear (NDS Table 10.3-1)

Resistance Factor, $\emptyset_b$ = 0.85 for Bending (NDS Table N2)

$\emptyset_v$ = 0.75 for Shear (NDS Table N2)

Time Effect Factor, λ = 0.8 (Live load is from occupancy, NDS Table N3)

The CLT slab's flexural strength is determined by the equation below from NDS Table 10.3-1.

$$F_b\left(S_{eff}\right)' = F_b\left(S_{eff}\right)C_D C_M C_t C_L K_F \emptyset \lambda$$

Where:

$F_b(S_{eff})$ is the nominal bending strength of the section (ANSI/APA PRG 320 Table A2)

The design strength is then compared to maximum stress under the loads on the slab to determine adequacy of the slab.

The equation for the shear strength, as a value of shear strength per unit length of width, of the CLT slab is presented below and is from Table 10.3-1 in the NDS.

$$V'_{s,0} = V_{s,0} C_M C_t K_F \emptyset$$

Where:

$V_{s,0}$ is the nominal shear strength of the section (ANSI/APA PRG 320 Table A2)

The design shear strength is compared to the shear force per unit length of width calculated under loads on the slab to determine the adequacy of the section.

Deflection will be checked based on Section 3.5 from the NDS. For deflection calculations, both shear deformation and flexural deformation need to be considered. The apparent modulus of elasticity, which accounts for both flexural and shear deformations, will be used and obtained from the NDS Supplement. The total deflection is calculated by the equations shown below (NDS Section 3.5).

$$\Delta_T = K_{cr}\Delta_{LT} + \Delta_{ST} \quad ----\text{NDS Eqn. } 3.5-1$$

Where:

K_{cr} = Creep Factor = 2.0 for CLT in dry conditions (NDS Sect. 3.5.2)

Δ_{LT} = Long Term Deflection (Dead Load Deflection) = $\frac{w_d l^4}{145 EI_{app} C_M C_t}$

Δ_{ST} = Short Term Deflection (Live Load Deflection) = $\frac{w_l l^4}{145 EI_{app} C_M C_t}$

$$(EI)_{app} = \text{Adjusted Stiffness for Shear Deformation} = \frac{(EI)_{eff}}{1 + \frac{K_s (EI)_{eff}}{GA_{eff} L^2}}$$

An important note for these deflection calculations is that equations for Δ_{LT} and Δ_{ST} are based on a 3-span continuous condition. For other conditions, the coefficients of these equations will be different. The bending and shear stiffness values used in the deflection equations will be obtained from Table A2 in the ANSI/APA PRG 320 (APA, 2018). The calculated deflections will be compared to the limits as recommended by the International Building Code. For typical floors, the limits are L/360 for live load only, and L/240 for dead and live load.

The fire and vibration designs have already been discussed in Chapter 2 and will not be repeated here.

3.2 Design Comparison

A design example is presented here for both traditional steel design and the hybrid design. The two designs are then compared to illustrate the differences. The design was done based on a few assumptions. The steel-timber hybrid system is designed as non-composite, whereas the steel framed building is designed with composite action between the steel beams and concrete topping. The dead load for the floor system (besides member self-weight) was taken to be 12 PSF (10 PSF for MEP/Misc., 2 PSF for an acoustical mat). The live load was taken to be uniform load of 100 PSF over the entire floor plate. The secondary beams were designed as continuously laterally braced members that are simply supported. The girders were designed as a simply supported beam with lateral bracing at third points from the secondary members framing into the girder. Additionally, the load on the secondary members will be uniformly distributed from the slab above, containing dead and live load, whereas the girder will be loaded via point loads at the third points from the secondary members. The CLT slab thickness was designed based on the bending and shear strengths of the sections from ANSI/APA PRG-320 (APA, 2018). A typical bay in a model building, shown in the figure below, is designed in two ways: a hybrid steel-timber system and a steel-framed building with a composite concrete floor system.

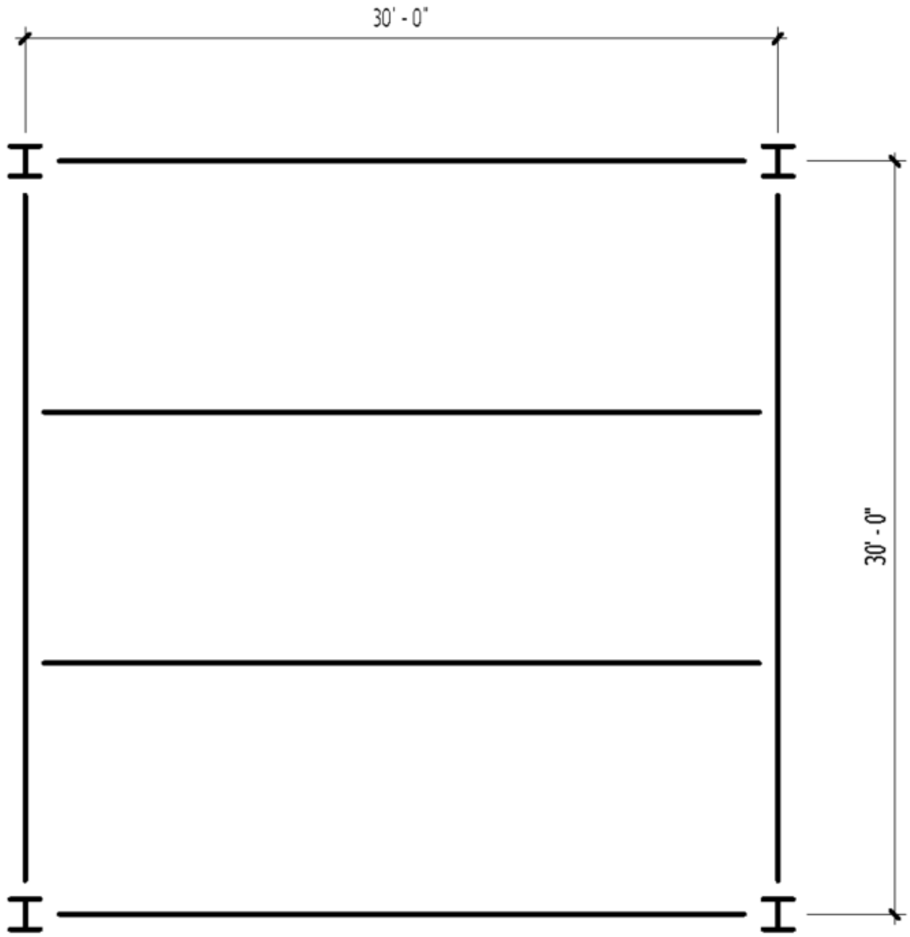

Figure 3.1. Structural Bay for Design Comparison

Once the design has been completed, a comparison will be made, outlining the overall differences between the two designs. Points of interest within the comparison will include steel weight, overall weight, and floor system depth. Both designs will have the same loading present, except for the self-weight of the structural members (beams, slab, etc.). The two designs are compared in the tables below, and the calculations are presented in Appendix B.

Design Comparison		
Parameter	Non-Composite Hybrid System	Steel Composite System
Slab Thickness	5 Ply - 6-7/8" CLT Slab, 3" NWC Topping	3VLI20 Deck with 4" NWC
Slab Weight w/ Topping	57.5 PSF	69 PSF
Total Dead Load	70 PSF	86 PSF
Secondary Beam Size	W21x44	W14x30
Girder Size	W21x55	W21x55
Column Load	219 kips	237 kips
Column Size	W12x40	W12x40

Table 3.1. Design Comparison Results

Material Usage Comparison (Per Typical Bay)		
Parameter	Hybrid System	Steel Composite System
Steel Weight (lbs) - Secondary Members	3960	2700
Steel Weight (lbs) - Girders	3300	3300
Overall Steel Weight (lbs)	7260	6000
Concrete Weight (lbs)	33750	61875
Timber Weight (lbs)	18000	0
Overall Slab Weight (lbs)	51750	61875
Overall Material Weight (lbs)	59010	67875

Table 3.2. Material Usage Comparison

As displayed in Table 3.1, the two designs have similarities. The main difference is the slab weight/total dead load that the floor system creates, illustrated in Table 3.2 for a singular structural bay. The difference of 16 PSF is not an incredible amount of load, but when this is applied over multiple floors and the entire floor plan in a building, the overall weight of the building may significantly change. As shown in Table 3-2, the overall material weight for the steel composite system is about 9 kips more than that of the hybrid system. Considering a perfect square building, 120’ x 120’, the steel composite system would have 144 kips more of load than the hybrid system per floor from the material self-weight. In the design of the column, the same column could be used for both designs due to the tributary area being only one floor. If this building became a 5-story office building, the difference between the column loading at the first floor increases to almost 100 kips, which would require an increased column size. This weight reduction is a significant difference that could be very beneficial to reduce seismic loading on a building, as well as foundation sizes. As for the design of the beams, the steel composite system utilized secondary members that were both lighter and shallower. While this is an advantage, adding steel anchors onto the top flange of the steel beam adds cost. The additional steel weight of the hybrid system is almost negligible. In terms of pounds per square foot, the hybrid system sees about 8 PSF while the steel composite system has 6.7 PSF. This is not a large amount of load at all and is not an issue when comparing the designs. For the floor system selection, similar depths were chosen to best portray how the systems' weights compare. The steel-timber hybrid system results in a larger slab thickness due to adding a 3" normal-weight concrete topping, which may be a drawback to this structural system. However, the ceiling is normally exposed in these hybrid steel-timber systems so the floor-to-floor heights may not change at all or be greater than the steel system.

After reviewing the two designs, the differences in a typical bay are insignificant. However, these differences can be increased over the entire building regarding the overall weight of the building. This weight reduction may provide significant benefits for the seismic and foundation design of the building. The design comparison was intended to display the difference between one of the most common design solutions, a steel composite system, and the new hybrid steel-timber system. It shows that the difference between them is not significant besides the overall weight of the building, which is a benefit for the hybrid steel-timber system.

Chapter 4: Conclusion

Steel and timber are two of the industry's most common and abundant construction materials. Up to this point, the two materials are rarely utilized together. The most common forms of each use of material are steel composite systems and mass timber systems. Both common structural systems have their advantages and disadvantages. The steel composite system is typically a heavier system than mass timber. A mass timber system typically has shorter spans than a steel-framed structure. However, the steel-timber hybrid structural system introduces the possibility of using the two materials together to achieve the benefits of both systems.

The same structural design considerations for steel and timber must be considered with this new system, but some aspects may be different. One key consideration for this hybrid steel-timber system is fire protection. The interface between the steel member and the CLT slab can create different behavior in these locations. The steel member increases temperature faster than that of the CLT slab, which at the interface results in expedited charring in the CLT if not designed correctly. With that, it is crucial to design the steel member for the required fire rating to avoid this situation, which could cause premature failure. The CLT slab will lose capacity after a fire, but the member retains some of its strength due to the charring action. Vibration can also be an issue with CLT floor panels, but with the addition of the steel frame and a potential concrete topping, these issues can be mitigated relatively easily. Span limits are recommended for vibration control by timber industry for mass timber construction, which will also be acceptable for the hybrid steel-timber systems. Advanced vibration analysis methods are available if span exceeds the limits. Composite action between timber and steel is an area of a potential increase in the efficiency of this system in the future as more research needs to be done to understand the behavior of the hybrid system.

Steel-timber hybrid system has many characteristics that make it a desirable structural system. Replacing the typical concrete floor used in a steel composite system with a CLT slab has many benefits, including lowering the overall weight of the building and creating a more sustainable design. CLT slab has the aesthetic appealing with exposed ceiling, which is typical for this system. Wood is a sustainable construction material, increasing the structural system's sustainability while not sacrificing much performance.

To this date, hybrid steel-timber buildings have not been a common design choice. A crucial factor contributing to this is that only a little research has been done on this system until recently. With the publication of the Design Guide 37 from the AISC, this new innovative system may rise in popularity due to the increased awareness of what this type of system can achieve. An essential step in using this structural system will be lowering the cost of timber, but as it becomes more and more common, the price will drop and make it an economical choice. Hybrid steel-timber structural systems are an excellent choice for a structural design project and can perform as well as other common systems.

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Appendix B - Design Calculations

The following pages will present the calculations that were mentioned in the report for the hybrid system and the typical steel framed building.

Steel-Timber Hybrid System Example	Master's Report	Cole Herpich

Design the gravity system for a steel-timber hybrid system.

Assume CLT slab provides continuous lateral bracing to the top flange of the steel beams, therefore no lateral torsional buckling

Step Description	Calculations	References
General Floor Information	l = 10 ft (Span between supports/secondary beams) 6-7/8" Depth CLT Panel, E2 CLT Layup	
Floor Loading	Floor Live Load - 100 PSF Floor Dead Load - 70 PSF CLT Slab Weight - 20 PSF (6 7/8" Thick, 5 Ply CLT) Concrete Topping - 37.5 PSF (3" NWC Topping) Acoustic Mat - 2 PSF MEP/Misc - 10 PSF	
	CLT Floor Slab Design	
Linear Loading	Linear Live Load per foot - 100 PLF Linear Dead Load per foot - 70 PLF	
Factored Load	$w_u = 1.2D + 1.6L =$ 243.4 PLF	
Determine Critical Loads	Assume that the slab is simply supported, continuous over 3 spans Shear Forces Exterior Condition - $V = \frac{2wl}{5} =$ 973.6 lb/ft Max. Interior Condition - $V = \frac{3wl}{5} =$ 1460.4 lb/ft	 AISC Manual Table 3-22c AISC Manual Table 3-22c

Steel-Timber Hybrid System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
	Moment Forces	
	Exterior Midspan Moment -	
	$M = .08wl^2 =$ 1947.2 lb*ft/ft	AISC Manual Table 3-22c
	Interior Midspan Moment -	
	$M = .025wl^2 =$ 608.5 lb*ft/ft	AISC Manual Table 3-22c
	Moment at Supports -	
	$M = .10wl^2 =$ 2434 lb*ft/ft	AISC Manual Table 3-22c
	Critical Shear and Moment,	
	$V_{max} =$ 1460.4 lb/ft	
	$M_{Max} =$ 2434 lb*ft/ft	
Determine Relevant Adj. Factors	$C_D =$ 1 (Live Load)	NDS Table 2.3.2
	$C_M =$ 1 (Dry Conditions)	NDS Sect. 10.3.3
	$C_t =$ 1 (Temperature less than 100 degrees)	NDS Table 2.3.3
	$C_L =$ 1 (Depth does not exceed breadth)	NDS Sect. 3.3.3
	$K_{F,Bending} =$ 2.54 (Format Conversion Factor)	NDS Table 10.3-1
	$K_{F,Shear} =$ 2	NDS Table 10.3-1
	$\emptyset_{Bending} =$ 0.85 (Resistance Factor)	NDS Table 10.3-1
	$\emptyset_{Shear} =$ 0.75	NDS Table 10.3-1
	$\lambda =$ 0.8 (Based on Live Load)	NDS Table N3

Step Description	Calculations	References
Flexural Design	$F_b(S_{eff})' = F_b(S_{eff})C_D C_M C_t C_L K_F \emptyset \lambda =$	NDS Table 10.3-1
	$F_b(S_{eff}) =$ 8,825 lb*ft/ft	ANSI/APA PRG 320 Table A2
	$F_b(S_{eff})' =$ 15242.54 lb*ft/ft	
	$F_b(S_{eff})' > M_{Max} =$ 2434 lb*ft/ft	
	CLT Slab is adequate for Flexure	
Shear Design	$V_{s,0}' = V_{s,0} C_M C_t K_F \emptyset \lambda =$	NDS Table 10.3-1
	$V_{s,0} =$ 2625 lb/ft	ANSI/APA PRG 320 Table A2
	$V_{s,0}' =$ 3150 lb/ft	
	$V'_{s,0} > V_{Max} =$ 1460.4 lb/ft	
	CLT Slab is adequate for Shear	
Deflection Check	$(EI)_{app} = \dfrac{(EI)_{eff}}{1 + \dfrac{K_s (EI)_{eff}}{GA_{eff} L^2}} =$ 303333251 lb*in^2/ft	NDS Eqn. 10.4-1
	$(EI)_{eff} =$ 389000000 lb*in^2/ft	
	$GA_{eff} =$ 1100000 lb/ft	
	$K_s =$ 11.5 (Pinned Condition)	NDS Table 10.4.1.1

Step Description	Calculations	References
	$\Delta_{Max,DL} = \frac{w_D l^4}{145(EI)_{app} C_M C_t} =$ 0.0273049 in	
	$\Delta_{Max,LL} = \frac{w_L l^4}{145(EI)_{app} C_M C_t} =$ 0.03928762 in	
	$\Delta_{max} = K_{cr}(\Delta_{Max,DL}) + \Delta_{Max,LL} =$ 0.0939 in	
	$K_{cr} =$ 2 (Cross-Laminated Timber)	NDS Sect. 3.5.2
	Deflection Limit -	
	$\Delta_{all} = \frac{L}{360} =$ 0.33333 in	
	$\Delta_{Max} < \Delta_{All}$	
	CLT Slab is adequate for Deflection	
Vibration Design	Based on span limits presented in the report from CLT Handbook	
	$l_{lim} \leq \frac{1}{12.05} \frac{(EI)_{app}^{.293}}{(\rho A)^{.122}} =$ 16.83 ft	CLT Handbook Chapter 7, Eqn. 4
	ρ = 0.35 lb/in^3 A = 82.5 in^2	
	Actual Span = 10 ft < l_{lim} = 16.83 ft	
	CLT Slab is adequate for Vibration	

Steel-Timber Hybrid System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
CLT Fire Design	For this design, a 1-hour rating is going to be assumed.	

Known Information:

$h =$	6.875	in	(Depth of CLT Member)
$h_{lam} =$	1.375	in	(Depth of Each Lamination)
$n_{lam} =$	1		(Laminations to be charred)
$t =$	1	hr	(Fire Rating)
$\beta_\eta =$	1.5	in/hr	(Char Rate)

Time for Char to reach glued interface, t_{gi},

$$t_{gi} = \left(\frac{h_{lam}}{\beta_\eta}\right)^{1.23} = 0.8985 \text{ hr}$$

Char Depth,

$$a_{char} = 1.2\left[n_{lam}h_{lam} + \beta_\eta\left(t - n_{lam}t_{gi}\right)^{0.813}\right]$$

$$a_{char} = 1.93023 \text{ in}$$

Effective Cross Section Depth, h_{fire},

$$h_{fire} = h - 1.2a_{char} = 4.56 \text{ in}$$

Effective Section Modulus After Fire, $S_{eff,fire}$,

$$S_{eff,fire} = \frac{1' * h_{fire}^2}{6} = 3.46 \text{ in}^3$$

Bending Capacity after Fire,

$$F_b S_{eff,fire} = \frac{F_b S_{eff} S_{eff,fire}}{S_{eff}} = 3880.22 \text{ lb*ft/ft}$$

$$F_b S_{eff,fire}' = 6701.91 \text{ lb*ft/ft}$$

$$F_b\left(S_{eff}\right)' > M_{Max} = 2434 \text{ lb*ft/ft}$$ **OK**

Steel-Timber Hybrid System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
	Steel Beam Design - Secondary Beams	
Beam Info	Length - 30 ft $F_y =$ 50 ksi	
Loading	Tributary Width = 10 ft Dead Load - 70 PSF Live Load - 100 PSF	
	Linear Load Along Beam,	
	Beam Self Weight - 44 plf (W18x40)	
	DL, Linear - 739 plf (Unfactored)	
	LL, Linear - 1000 plf (Unfactored)	
	Linear load, Factored -	
	$w_u = 1.2D + 1.6L =$ 2486.8 plf	
	Design Moment and Shear Forces,	
	$V_{max} = \frac{wl}{2} =$ 37.302 kips	AISC Manual Table 3-23
	$M_{max} = \frac{wl^2}{8} =$ 279.765 kip*ft	
Preliminary Design	Design Based on Deflection Limits,	
	$I_{x,re\ 'd} > \frac{5wl^4}{384E\Delta_{All}} =$ 728.58 in^4	
	$\Delta_{all} = \frac{l}{240} =$ 1.5 in (Total Load Deflection)	

Steel-Timber Hybrid System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
	Preliminary Section Size,	AISC Manual Table 1-1
	Try a W21x44 with I_x = 843 in^4	
Beam Properties	I_x = 843 in^4 Z_x = 95.4 in^3 d = 20.7 in t_w = 0.35 in h/t_w = 53.6	AISC Manual Table 1-1
Moment Capacity	Check Moment Capacity,	
	$\emptyset_b M_n = 0.9F_y Z_x =$ 357.75 k*ft	AISC Spec. Eqn. F2-1
	$\emptyset_b M_n > M_{max} =$ 279.765 k*ft	
	Lateral Torsional Buckling does not apply due to CLT Slab	
	Secondary Steel Beam is adequate for Flexure	
Shear Capacity	Check Shear Capacity,	
	$\emptyset_v V_n = \emptyset_v * 0.6F_y A_w C_{v1} =$ 217.35 kips	AISC Spec. Eqn. G2-1
	$A_w = dt_w =$ 7.245 in^2	
	h/t_w = 53.6 < 53.9463 $= 2.24\sqrt{\frac{E}{F_y}}$	
	Therefore, $\emptyset_v =$ 1 $C_{v1} =$ 1	
	$\emptyset_v V_n > V_{max} =$ 37.302 kips	
	Secondary Steel Beam is adequate for Shear	

Steel-Timber Hybrid System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
Confirm Deflection Adequacy	Deflection Limits: Total Load - $\Delta_{all,TL} = \frac{l}{240} =$ 1.5 in Live Load - $\Delta_{all,LL} = \frac{l}{360} =$ 1 in Actual Deflections Total Unfactored Load Deflection, $\Delta_{Act,TL} = \frac{5wl^4}{384EI} =$ 1.30 in < 1.5 in $= \Delta_{all,TL}$ Unfactored Live Load Deflection, $\Delta_{Act,LL} = \frac{5wl^4}{384EI} =$ 0.745 in < 1 in $= \Delta_{all,LL}$ **Secondary Steel Beam is adequate for Deflection**	IBC 2021 Table 1604.3
Final Design	**Utilize a W21x44 for the secondary steel members in the hybrid steel timber design**	

Steel-Timber Hybrid System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
	Steel Beam Design - Girders	
Loading	Point Loads at 10' Increments, Length = 30 ft	
	P_{DL} = 11.085 kips (Unfactored)	
	P_{LL} = 15 kips (Unfactored)	
	Factored Point Load Value,	
	$P_u = 1.2P_{DL} + 1.6P_{LL} =$ 37.302 kips	
	Dead Load from Girder Self Weight,	
	$w_{sw} =$ 55 plf (W27x84)	
	$w_{sw,u} = 1.2w_{sw} =$ 66 plf	
	Design Moment and Shear Forces,	
	$V_{max} = P_u + \frac{w_{sw,u}l}{2} =$ 38.292 kips	AISC Manual Table 3-23
	$M_{max} = P_u a + \frac{w_{sw,u}l^2}{8} =$ 380.445 k*ft	
	a = point load spacing = 10 ft	
Preliminary Design	Design Based on Deflection Limits, neglect self weight for now,	
	$I_{x,req'd} > \frac{P_{TL}a}{24E\Delta_{all}}(3l^2 - 4a^2) =$ 993.029 in^4	AISC Manual Table 3-23
	$\Delta_{all} = \frac{l}{240} =$ 1.5 in (Total Load Deflection)	

Steel-Timber Hybrid System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
	Preliminary Section Size,	
	Try a W21x55 with I_x = 1140 in^4	
Beam Properties	I_x = 1140 in^4 S_x = 110 in^3 Z_x = 126 in^3 C_b = 1 d = 20.8 in t_w = 0.375 in h/t_w = 50	AISC Manual Table 1-1
Moment Capacity	Check Moment Capacity for Yielding,	
	$M_n = M_p = F_y Z_x =$ 525 k*ft	AISC Spec. Eqn. F2-1
	$\emptyset_b M_n =$ 472.5 k*ft	
	$\emptyset_b M_n > M_{max} =$ 380.445 k*ft	
	Lateral Torsional Buckling does apply for girder,	
	L = 30 ft l_b = 10 ft	AISC Manual Table 3-2
	l_p = 6.1 ft l_r = 17.4 ft	
	$l_p < l_b \leq l_r$, Therefore Eqn. F2-2 Applies	
	$M_n = C_b\left[M_p - \left(M_p - 0.7F_yS_x\right)\left(\frac{l_b - l_p}{l_r - l_p}\right)\right] =$ 454.654 k*ft	
	$\emptyset_b M_n =$ 409.19 k*ft	
	$\emptyset_b M_n > M_{max} =$ 380.445 k*ft	
	Girder is adequate for Flexure	

Steel-Timber Hybrid System Example	Master's Report	Cole Herpich
Step Description	**Calculations**	**References**
Shear Capacity	Check Shear Capacity, $\emptyset_v V_n = \emptyset_v * 0.6F_y A_w C_{v1} =$ 234 kips $A_w = dt_w =$ 7.8 in^2 $h/t_w =$ 50 < 53.9463 $= 2.24\sqrt{\frac{E}{F_y}}$ Therefore, $\emptyset_v =$ 1 $C_{v1} =$ 1 $\emptyset_v V_n > V_{max} =$ 38.292 kips **Girder is adequate for Shear**	AISC Spec. Eqn. G2-1

Steel-Timber Hybrid System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
Confirm Deflection Adequacy	Deflection Limits:	
	Total Load - $\Delta_{all,TL} = \dfrac{l}{240} =$ 1.5 in	IBC 2021 Table 1604.3
	Live Load - $\Delta_{all,LL} = \dfrac{l}{360} =$ 1 in	
	Actual Deflections	
	Total Unfactored Load Deflection,	
	$\Delta_{Act,TL} = \dfrac{5wl^4}{384EI} + \dfrac{P_{TL}a}{24EI}(3l^2 - 4a^2) =$ 1.33694 in	
	$\Delta_{Act,TL} =$ 1.33694 in < 1.5 in $= \Delta_{all,TL}$	
	Unfactored Live Load Deflection,	
	$\Delta_{Act,LL} = \dfrac{P_{LL}a}{24EI}(3l^2 - 4a^2) =$ 0.75136 in	
	$\Delta_{Act,LL} =$ 0.75136 in < 1 in $= \Delta_{all,LL}$	
	Girder is adequate for Deflection	
Final Design	**Utilize a W21x55 for the girders in the hybrid steel timber design**	

Steel-Timber Hybrid System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
	Typical Steel Column Design	
Loading	Loads Determined based on floor loads and tributary area, Floor Dead Load - 70 PSF Floor Live Load - 100 PSF Tributary Area = 900 (30x30) SF (Based on assuming more than one bay, interior column) Column Load, P_D = 62.55 kips P_L = 90 kips $P_u = 1.2P_D + 1.6P_L =$ 219.06 kips Try a W12x40 for column design,	
Column Properties	Column Properties L_b = 16 ft (16 ft unbraced length, given) L_c = 16 ft (K=1, Pinned-Pinned Connection) r_x = 5.13 in r_y = 1.94 in A_g = 11.7 in^2 E = 29000 ksi F_y = 50 ksi	
Column Strength	Compressive Strength Equation: $\emptyset_c P_n = 0.9F_{cr}A_g$	AISC Spec. Eqn. E3-1

Steel-Timber Hybrid System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
Slenderness	Check Slenderness of Column	
	$\frac{L_c}{r_x} = 37.4269$	
	$\frac{L_c}{r_y} = 98.9691$ (Governs)	
	$\frac{L_c}{r_y} = 98.9691 < 113.432 = 4.71\sqrt{\frac{E}{F_y}}$	
	Therefore,	
	$F_{cr} = (0.658)^{\frac{F_y}{F_e}} * F_y = 24.4300697$ ksi	AISC Spec. Eqn. E3-2
	Where:	
	$F_e = \frac{\pi^2 E}{(\frac{L_c}{r})^2} = 29.22$ ksi	AISC Spec. Eqn. E3-4
Strength	Column Strength,	
	$\emptyset_c P_n = 0.9 F_{cr} A_g = 257.25$ kips	AISC Spec. Eqn. E3-1
	$\emptyset_c P_n = 257.25$ kips > 219 kips $= P_u$	
	W12x40 Column is Adequate for this Design	
Final Design	The final design for this hybrid system is as follows: 6 7/8" thick, E2 CLT panel with 3" NWC Topping W21x44 secondary beams, W21x55 Girders W12x40 Columns	

Steel Composite System Example	Master's Report	Cole Herpich

Design the gravity system for a steel composite system - 3/4 Inch Shear Studs

Step Description	Calculations			References
Bay Spacing/ Deck Size/ General Info	**Steel Composite Beam Design**			
	Beam Spacing/Length			
	Tributary width/Beam Spacing =	10	ft	
	Beam Length =	30	ft	
	Deck/Topping Information			
	Deck (Vulcraft) -	3VLI20		Vulcraft Catalog
	NWC Topping Depth -	4	in	
	Topping Self Weight -	150	PCF	
	Deck Total Depth -	7	in	
	Concrete Compressive Strength			
	Compressive Strength, f'_c =	4	ksi	
	Steel Strength			
	Yield Strength, F_y =	50	ksi	
	Ult. Strength, F_u -	65	ksi	
Loading Information	Loading Information			
	Dead Loads			
	Pre-Composite Loads:			
	Slab Weight =	69	PSF	Vulcraft Catalog
	Steel Weight =	5	PSF	Assumed
	Composite Loads:			
	Misc. (MEP,etc) =	12	PSF	
	Live Loads			
	Pre-Composite Loads:			
	Construction Load =	25	PSF	
	Composite Loads:			
	Storage/Office =	100	PSF	

Steel Composite System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
Composite Deck and Anchor Code Checks	1) Concrete Strength: Based on LWC or NWC	AISC Specification
	3 ksi ≤ 4 ksi ≤ 10 **O.K.**	Sect. I1.3
	2) Rib Height: $h_r \leq 3"$	Sect. I3.2c
	h_r = 3 in **O.K.**	
	3) Average Rib Width: $w_r \geq 2"$	Sect. I3.2c
	w_r = 7.25 **O.K.**	
	4) Steel Headed Stud Anchor Diameter ≤ 3/4"	Sect. I8.1
	Shear Stud Diameter = 0.75 in **O.K.**	
	5) Flange Thickness Requirement: $t_f \geq 0.3"$	Sect. I8.1
	Confirm After Pre-Composite Design	
	6) Min. Stud Length Above Concrete: Stud Length ≥ 1.5"	
	Utilize 4.875" Anchors per Anchor Manufacturer	
	7) Minimum Length of Stud Anchors: $4*d_{sa}$	Sect. I8.2
	4.5 in > 3 in = $4*d_{sa}$ **O.K.**	
	8) Concrete Cover on Top of Studs: Cover ≥ 0.5"	Comm. I3.2c
	2.5 in > 0.5 in **O.K.**	
	9) Slab Thickness Above Steel Deck: Cover ≥ 2"	Sect. I3.2c

Steel Composite System Example	Master's Report	Cole Herpich
Step Description	**Calculations**	**References**
Pre-Composite Design	Precomposite Dead Load: $w_D =$ 0.74 klf Precomposite Live Load: $w_L =$ 0.25 klf Factored Loads: $w_u =$ 1.288 klf $\quad w_u = 1.2w_D + 1.6w_L$ Ultimate Pre-Composite Moment: $M_u =$ 144.9 k*ft $\quad M_u = \frac{w_u l^2}{8}$ Required Plastic Section Modulus: $Z_x =$ 38.64 in^3 $\quad Z_x = \frac{M_u}{\emptyset F_y}$ Beam Selection based on Precomposite Loading: $\boxed{\text{Utilize W14x30}}$ with $Z_x =$ 47.3 in^3 Beam Section Properties: $A_g =$ 8.85 in^2 $\quad h/t_w =$ 45.4 $t_f =$ 0.385 in2 $\quad I_x =$ 291 in^4 Deflection Calculation $\Delta_{nc} =$ 1.5981 in $\quad \Delta_{nc} = \frac{5w_D L^4}{384EI}$ $\Delta_{All} =$ 1.0 in $\quad \Delta_{All} = \frac{L}{360}$ Camber Necessary for Design 80% of Calculated Deflection = 1.2785 in $\boxed{\text{Utilize 1.25 in. camber}}$	AISC Manual Table 1-1 IBC Table 1604.3

Steel Composite System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
Composite Design	Composite Dead Load:	
	w_D = 0.86 klf	
	Composite Live Load:	
	w_L = 1 klf	
	Factored Loads:	
	w_u = 2.632 klf	
	Ultimate Composite Moment:	
	M_u = 296.1 k*ft $M_u = \frac{w_u l^2}{8}$	
Effective Width	Determine Effective Width, b:	
	1) 1/8th of the beam span, center to center of supports	
	1/8*L = 7.5 ft	
	2) 1/2 the distance to the centerline of the adjacent beam	
	1/2*Spacing = 10 ft	
	3) Distance to Edge of Slab	
	N/A	
	4) Governing Effective Width,	
	b = 7.5 ft	

Steel Composite System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
Composite Design Cont.	Available Flexural Strength: $h/t_w =$ 45.4 $\leq$ 90.6 Therefore Plastic Stress Distribution Applies Trial Value for Compression Block: $a_{trial} =$ 0.723 in $\quad a_{trial} = \frac{0.5(A_s F_y)}{0.85 f_c' b}$ Compressive Concrete Flange Force to Beam Top Flange, Y2 $Y_{con} =$ 7 in Y2 = 6.6385 in $\quad Y2 = Y_{con} - \frac{a_{trial}}{2}$ AISC Manual Table 3-19 Y2 = 6.6385 in $\sum Q_n =$ 183 kips Selecting PNA Location 5 with $Q_n =$ 183 k $\varphi_b M_n =$ 315 k*ft > 296.1 k*ft Actual Compression Block Depth, a: $a = \frac{\sum Q_n}{0.85 f_c' b}$ a = 0.598 in < 0.723 in = a_{trial} Live Load Deflection Check: $I_{LB} =$ 756 in^4 $\Delta_c =$ 0.8313 in $\quad \Delta_c = \frac{5 w_L L^4}{384 E I_{LB}}$ $\Delta_c =$ 0.8313 in < 1.0 in = Δ_{all}	AISC Manual Table 3-19 AISC Manual Table 3-20

Steel Composite System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
	AISC Design Guide 3 Deflection Limit $\Delta_c =$ 0.4156 in < 1 OK $\Delta_c = 0.5 * \Delta_c < 1"$	AISC Design Guide 3
Final Member Selection	Final Member Selection Section Properties: W14x30 Is Adequate with composite design $A_g =$ 8.85 in^2 h/tw = 45.4 $t_f =$ 0.385 in Ix = 291 in^4 $I_{LB} =$ 756 in4	
Steel Anchor Strength	Steel Anchor Strength: 1 Anchor Per Rib = 17.2 kips 2 Anchors Per Rib = 14.6 kips	AISC Manual Table 3-21
Number and Spacing of Anchors	Number and Spacing of Anchors: Deck Flute Spacing = 12 in Anchor Spacing = 12 in $n_{flutes} =$ 15 flutes $n_{flutes} = n_{spaces} + 1$ $n_{anchors} =$ 11 anchors $n_{anchors} = \frac{\sum Q_n}{Q_n}$ Provide 11 anchors on each side of the beam centerline Check Capacity: $\sum Q_n =$ 189.2 kips > 183 kips	

Steel Composite System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
Steel Anchor Ductility	Steel Anchor Ductility Check: Beams are not susceptible to connector failure due to insufficient deformation capacity if they meet one of the following requirements: 1) Beam span less than 30 ft Span = 30 ft ≤ 30 ft **O.K.** 2) Beams with degree of composite action of at least 50% $\frac{\sum Q_n}{\min\{0.85 f_c' A_c, F_y A_s\}} =$ 42.8% **N.G.** 3) Beams with average nominal shear connector capacity of 16 k/ft along span. 12.613 k/ft < 16 k/ft **N.G.**	AISC Spec. Comm. Sect. I3.2d
Review Anchor Requirements	Steel Headed Stud Anchor Spacing Requirements: 1) Max anchor spacing - Min{ 8*t_{slab} , 36} 36 in > 12 in **O.K.**	AISC Spec. Sect. I8.2d and I3.2c
	2) Minimum Anchor Spacing Along Beam 4*d_{sa} = 3 in ≤ 12 in **O.K.**	I8.2.2d(d)
	3) Minimum transverse spacing between anchor pairs 4*d_{sa} = 3 in ≤ 3 in **O.K.**	I8.2.2d(d)
	4) Minimum Distance to Free Edge in Direction of Shear Center of anchor to free edge should be 8" Min.	
	5) Max Spacing of Deck Attachment Steel deck must be anchored at max spacing of 18"	I3.2c

Steel Composite System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
Shear Strength	Shear Design for Steel Beam Alone V_u = 39.48 kips $\varphi_v V_n$ = 112 kips > 39.48 kips **O.K.**	AISC Manual Table 3-2
Final Member Details	Final Composite Design: Member Size: W14x30 Camber: 1.25 in Shear Studs: 22 3/4" Diameter by 4.875" Shear Studs	

Steel Composite System Example	Master's Report	Cole Herpich

Step Description	Calculations			References
Bay Spacing/ Deck Size/ General Info	**Steel Composite Girder Design**			
	Beam Spacing/Length			
	Tributary width/Beam Spacing =	10	ft	
	Beam Length, $l_1 = l_2$ =	30	ft	
	Girder Length =	30	ft	
	Girder Spacing =	30	ft	
	Deck/Topping Information			
	Deck (Vulcraft) -	3VLI20		Vulcraft Catalog
	NWC Topping Depth -	4	in	
	Topping Self Weight -	150	PCF	
	Deck Total Depth -	7	in	
	Concrete Compressive Strength			
	Compressive Strength, f'_c =	4	ksi	
	Steel Strength			
	Yield Strength, F_y =	50	ksi	
	Ult. Strength, F_u -	65	ksi	
Loading Information	Loading Information			
	Dead Loads			
	Pre-Composite Loads:			
	Slab Weight =	69	PSF	Vulcraft Catalog
	Steel Weight =	5	PSF	
	Self Weight =	55	PLF	Assumed
	Composite Loads:			
	Misc. (MEP,etc) =	12	PSF	
	Live Loads			
	Pre-Composite Loads:			
	Construction Load =	25	PSF	
	Composite Loads:			
	Storage/Office =	100	PSF	

Steel Composite System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
Composite Deck and Anchor Code Checks	1) Concrete Strength: Based on LWC or NWC	AISC Specification
	3 ksi ≤ 4 ksi ≤ 10 **O.K.**	Sect. I1.3
	2) Rib Height: $h_r \leq 3"$	Sect. I3.2c
	$h_r =$ 3 in **O.K.**	
	3) Average Rib Width: $w_r \geq 2"$	Sect. I3.2c
	$w_r =$ 7.25 **O.K.**	
	4) Steel Headed Stud Anchor Diameter ≤ 3/4"	Sect. I8.1
	Shear Stud Diameter = 0.75 in **O.K.**	
	5) Flange Thickness Requirement: $t_f \geq 0.3"$	Sect. I8.1
	Confirm After Pre-Composite Design	
	6) Min. Stud Length Above Concrete: Stud Length ≥ 1.5"	
	Utilize 4.875" Anchors per Anchor Manufacturer	
	7) Minimum Length of Stud Anchors: $4*d_{sa}$	Sect. I8.2
	4.5 in > 3 in = $4*d_{sa}$ **O.K.**	
	8) Concrete Cover on Top of Studs: Cover ≥ 0.5"	Comm. I3.2c
	2.5 in > 0.5 in **O.K.**	
	9) Slab Thickness Above Steel Deck: Cover ≥ 2"	Sect. I3.2c

Steel Composite System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
Pre-Composite Design	Dead Load From Beams:	

Dead Load From Beams:

$P_d =$ 23.85 kips $\quad P_D = \frac{l_1 + l_2}{2}(TW)(DL)$

live Load From Beams:

$P_L =$ 7.5 kips $\quad P_L = \frac{l_1 + l_2}{2}(TW)(LL)$

Ultimate Point Load From Beams:

$P_u =$ 40.62 kips $\quad P_u = 1.2P_D + 1.6P_L$

Ultimate Moment:

$M_u =$ 406.2 k*ft $\quad M_u = P_u a$

Moment determined based on two point loads on girder that are equivalent and spaced symmetrically at 10' O.C.

Beam Selection:

$\boxed{\text{Utilize a W21x55 Girder}}$

Beam Section Properties:

						References
$l_b =$	10	ft	$\varphi_b BF=$	16.3	k	AISC Manual
$l_p =$	6.11	ft	$\varphi_b M_{px}=$	473	k*ft	Table 3-2
$l_r =$	17.4	ft	$\varphi_b M_{rx}=$	289	k*ft	
$A_g =$	16.2	in^2	h/t_w -	50		AISC Manual
$t_f =$	0.522	in	I_x -	1140	in^4	Table 1-1

Available Moment Capacity:

$$\emptyset_b M_n = C_b[\emptyset_b M_{px} - \emptyset_b BF(l_b - l_p)] \le \emptyset_b M_{px}$$

$\emptyset_b M_n =$ 409.6 k*ft > 406.2 k*ft

Steel Composite System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
Pre-Composite Design Cont.	Deflection Design - Pre-Composite: $\Delta_{nc}=$ 1.6007 in $\quad \Delta_{nc}=\frac{Pa}{24EI}(3l^2-4a^2)+\frac{5w_{sw}l^4}{384EI}$ Deflection Limit: $\Delta_{All}=$ 1 in Camber Necessary For Design: 80% of Calculated Deflection = 1.2805 in Utilize 1.25 in. Camber	
Composite Design	Dead Load From Beams: $P_d=$ 25.8 kips $\quad P_D=\frac{l_1+l_2}{2}(TW)(DL)$ Live Load From Beams: $P_L=$ 30 kips $\quad P_L=\frac{l_1+l_2}{2}(TW)(LL)$ Ultimate Point Load From Beams: $P_u=$ 78.96 kips $\quad P_u=1.2P_D+1.6P_L$ Ultimate Moment: $M_u=$ 789.6 k*ft $\quad M_u=P_u a$	

Steel Composite System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
Composite Design Cont.	Determine Effective Width, b, 1) 1/8th of the beam span, center to center of supports 7.5 ft 2) 1/2 distance to the centerline of adjacent beam 30 ft 3) Distance to Edge of Slab N/A Governing Effective Width: b = 7.5 ft	
	Available Flexural Strength $h/t_w =$ 50 $\leq$ 90.6 Therefore Plastic Stress Distribution Applies Trial Value for Compression Block: $a_{trial} =$ 1.3235 in $a_{trial} = \frac{0.5(A_s F_y)}{0.85 f_c' b}$ Compressive Concrete Flange Force to Beam Top Flange, Y2 $Y_{con} =$ 7 in Y2 = 6.3382 in $Y2 = Y_{con} - \frac{a_{trial}}{2}$	

Steel Composite System Example	Master's Report	Cole Herpich
Step Description	Calculations	References
	AISC Manual Table 3-19 Y2 = 6.3382 in $\sum Q_n =$ 381 kips Selecting PNA Location 5 with Q_n = 381 k $\varphi_b M_n$ = 795 k*ft > 789.6 k*ft Actual Compression Block Depth, a: a= $\frac{\sum Q_n}{0.85 f_c' b}$ a = 1.2451 in < 1.3235 in = a_{trial} Live Load Deflection Check: I_{LB} = 2530 in^4 Δ_c = 0.0333 in $\Delta_c = \frac{Pa}{24EI_{LB}}(3l^2 - 4a^2) + \frac{5w_{sw}l^4}{384EI_{LB}}$ Δ_c = 0.0333 in < 1.0 in = Δ_{all} AISC Design Guide 3 Deflection Limit Δ_c = 0.017 in < 1 OK $\Delta_c = 0.5 * \Delta_c < 1"$	AISC Manual Table 3-19 AISC Manual Table 3-20 AISC Design Guide 3
Final Member Selection	Final Member Selection Section Properties: W14x30 Is Adequate with composite design A_g = 16.2 in^2 h/tw - 50 t_f = 0.522 in Ix - 1140 in^4 I_{LB} = 2530 in4	

Steel Composite System Example	Master's Report	Cole Herpich
Step Description	**Calculations**	**References**
Steel Anchor Strength	Steel Anchor Strength: 1 Anchor Per Rib = 21.5 kips	AISC Manual Table 3-21
Number and Spacing of Anchors	Number and Spacing of Anchors: Deck Flute Spacing = 12 in Anchor Spacing = 12 in $n_{flutes} =$ 15 flutes $\quad n_{flutes} = n_{spaces} + 1$ $n_{anchors} =$ 18 anchors $\quad n_{anchors} = \frac{\sum Q_n}{Q_n}$ Provide 18 anchors on each side of the beam centerline Check Capacity: $\sum Q_n =$ 387 kips > 381 kips	
Steel Anchor Ductility	Steel Anchor Ductility Check: Beams are not susceptible to connector failure due to insufficient deformation capacity if they meet one of the following requirements: 1) Beam span less than 30 ft Span = 30 ft ≤ 30 ft **O.K.** 2) Beams with degree of composite action of at least 50% $\frac{\sum Q_n}{\min\{0.85 f_c' A_c, F_y A_s\}} =$ 47.8% **N.G.** 3) Beams with average nominal shear connector capacity of 16 k/ft along span. 25.8 k/ft < 16 k/ft **O.K.**	AISC Spec. Comm. Sect. I3.2d

Steel Composite System Example	Master's Report	Cole Herpich
Step Description	**Calculations**	**References**
Review Anchor Requirements	Steel Headed Stud Anchor Spacing Requirements:	AISC Spec. Sect. I8.2d and I3.2c
	1) Max anchor spacing - Min{ $8*t_{slab}$, 36}	
	36 in > 12 in **O.K.**	
	2) Minimum Anchor Spacing Along Beam	I8.2.2d(d)
	$4*d_{sa}$ = 3 in ≤ 12 in **O.K.**	
	3) Minimum transverse spacing between anchor pairs	I8.2.2d(d)
	$4*d_{sa}$ = 3 in ≤ 12 in **O.K.**	
	4) Minimum Distance to Free Edge in Direction of Shear	
	Center of anchor to free edge should be 8" Min.	
	5) Max Spacing of Deck Attachment	I3.2c
	Steel deck must be anchored at max spacing of 18"	
Shear Strength	Shear Design for Steel Beam Alone	
	V_u = 78.96 kips	
	$\varphi_v V_n$ = 112 kips > 78.96 kips **O.K.**	AISC Manual Table 3-2
Final Member Details	Final Composite Design:	
	Member Size: W14x30	
	Camber: 1.25 in	
	Shear Studs: 36 3/4" Diameter by 4.875" Shear Studs	

Steel Composite System Example	Master's Report	Cole Herpich

Step Description	Calculations	References
	<u>Typical Steel Column Design</u>	
Loading	Loads Determined based on floor loads and tributary area,	
	Floor Dead Load - 86 PSF	
	Floor Live Load - 100 PSF	
	Tributary Area = 900 SF (30x30) (Based on assuming more than one bay, interior column)	
	Column Load,	
	$P_D =$ 77.4 kips	
	$P_L =$ 90 kips	
	$P_u = 1.2P_D + 1.6P_L =$ 236.88 kips	
	Try a W12x40 for column design,	
Column Properties	Column Properties	
	$L_b =$ 16 ft (16 ft unbraced length, given)	
	$L_c =$ 16 ft (K=1, Pinned-Pinned Connection)	
	$r_x =$ 5.13 in	
	$r_y =$ 1.94 in	
	$A_g =$ 11.7 in^2	
	E = 29000 ksi	
	$F_y =$ 50 ksi	
Column Strength	Compressive Strength Equation:	
	$\emptyset_c P_n = 0.9 F_{cr} A_g$	AISC Spec. Eqn. E3-1

Steel Composite System Example	Master's Report	Cole Herpich
Step Description	**Calculations**	**References**
Slenderness	Check Slenderness of Column $\frac{L_c}{r_x} =$ 37.427 $\frac{L_c}{r_y} =$ 98.969 (Governs) $\frac{L_c}{r_y} =$ 98.969 < 113.43 $= 4.71\sqrt{\frac{E}{F_y}}$ Therefore, $F_{cr} = (0.658)^{\frac{F_y}{F_e}} * F_y =$ 24.4300697 ksi Where: $F_e = \frac{\pi^2 E}{(\frac{L_c}{r})^2} =$ 29.22 ksi	AISC Spec. Eqn. E3-2 AISC Spec. Eqn. E3-4
Strength	Column Strength, $\emptyset_c P_n = 0.9 F_{cr} A_g =$ 257.25 kips $\emptyset_c P_n =$ 257.25 kips > 236.88 kips $= P_u$ **W12x40 Column is Adequate for this Design**	AISC Spec. Eqn. E3-1
Final Design	The final design for the steel composite system is as follows: Secondary Beams - W14x30, 1.25" Camber, 22 Shear Studs Girders - W21x55, 1.25" Camber, 36 Shear Studs Column - W12x40	