Spring 4-20-2012

A Composite Structural Steel and Prestressed Concrete Beam for Building Floor Systems

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A COMPOSITE STRUCTURAL STEEL AND PRESTRESSED CONCRETE BEAM FOR BUILDING FLOOR SYSTEMS

by

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A THESIS

Presented to the Faculty of
The Graduate College at the University of Nebraska
In Partial Fulfillment of Requirements
For the Degree of Master of Science

Major: Architectural Engineering

Under the Supervision of Professor George Morcous

Lincoln, Nebraska

May, 2012
Precast prestressed concrete beams, such as rectangular and inverted tee beam, currently used in residential and commercial buildings are deep, heavy, and limited to span-to-depth ratios of 15. The research proposes a composite structural steel and prestressed concrete beam that is shallow, light, easy to produce and erect, and able to achieve a span-to-depth ratio of 24. The proposed beam is designed to be used with precast columns, hollow-core planks, and a cast-in-place topping to create a moment-resisting floor system that minimizes the need for shear walls. The goal of this system is to eliminate as many of the limitations of precast concrete buildings as possible while remaining economically competitive. The developed beams consist of one half of a standard steel W-section, embedded into the top of a shallow rectangular prestressed concrete bottom flange, to create a composite section that supports hollow-core planks. Cast-in-place concrete is then used to fill the voids between the hollow-cores and composite beam and provide a leveled topping. A typical commercial building was analyzed and designed using the proposed beam under normal loading conditions. This design example indicated that the proposed system is economical, shallower, lighter, and more resistant to lateral loads than conventional precast concrete floor systems.
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Chapter 1 - Introduction

1.1 - Problem Statement

Many options are available to designers when developing the structural system of a building. The most common approaches are precast concrete, cast-in-place (CIP) concrete, or structural steel, each with its own distinct advantages and disadvantages. Typical drawbacks of a standard precast system include low span-to-depth ratios of floors as well as the need for shear walls to resist lateral loads. This means increased costs in the façade, mechanical electrical and plumbing (M/E/P) systems, and greater energy consumption for heating and cooling, due to increased floor-to-floor heights as well as a decrease in the versatility of the space. Prestressing can be utilized to improve the span-to-depth ratios of precast but it still underperforms when compared to steel. Steel systems will often utilize bar joists to support the floor structure; however, they too have rather low span-to-depth ratios. One of the major deficiencies of steel structures is that they have low fire-resistance characteristics and require the use of special partitions or fireproofing methods. Steel structures typically will employ frames to resist the applied lateral loads, which allow the architect much more flexibility over a shear wall system when designing the layout and functionality of the building. Additionally, steel is much lighter than concrete and this saves on erection costs. One of the primary concerns of cast-in-place systems is they usually require the use of formwork and shoring while curing, which results in increases in construction cost and duration over typical precast construction. Post-tensioning of CIP concrete allows for some of the shallowest floor systems but they are rather complex systems to construct. The ideal structural system
would eliminate the drawbacks of each of these traditional systems while retaining as many of the advantages of each as possible.

1.2 - Objectives

The primary objective of this project is to develop a shallow composite structural steel and prestressed concrete beam for residential and commercial construction. The beam being developed is designed to have the following features to address the shortcomings of existing floor beams:

- All exposed surfaces should be concrete to take advantage of its fire-resistance characteristics.

- Utilize both steel and prestressing strand to achieve a span-to-depth ratio of 24 or better which will help decrease the floor-to-floor heights as well as decrease the erection weight over standard precast components.

- Detail member connections as full moment resistant connections to reduce the positive moments, and resist lateral loads, while maintaining simplicity of the connection for construction.

- Use precast concrete components for as much of the building as possible to take advantage of its greater production quality, improved construction and erection time, and eliminate the need for formwork and shoring.

- Produce all structural components with standard materials and in a minimally complex manner so that the system can be commercially viable in as many markets as possible.
1.3 - Scope

The scope of this project is constrained purely to the development and theoretical evaluation of the proposed system. An analysis is conducted on all structural components for resisting both the gravity and lateral loads of a typical six-story office building in zones with low seismic activity. This evaluation includes the analysis, design, production methodology, construction sequencing, and detailing of all typical members and member connections. Full-scale testing should be done in the future to verify the assumptions and performance of the structural system presented.
Chapter 2 – Literature Review

The literature review for this report focuses on current structural systems commonly available to designers for residential and commercial buildings, which is the target application for the proposed system. Currently, the most common systems are cast-in-place, structural steel with steel joist or wide flange beams, precast concrete, as well as several proprietary systems. The scope of the proprietary systems for this literature review is limited to composite structural systems. The remainder of this chapter briefly describes the attributes of each of these systems.

2.1 – Cast-in-place systems

CIP systems have many inherent advantages. It is an extremely versatile material because its shape is directed by formwork erected on site. The formwork can be configured into almost any shape or dimension desired. Another advantage of CIP concrete is the continuity of the system, which allows for the implementation of frames as the lateral load resisting system. This gives the architect significant versatility in the floor plan of the space by allowing for greater flexibility in placement of walls. Columns, beams, and floors are cast monolithically and therefore; do not require connections between members as most other structural systems do. The natural properties of concrete make it a very durable material, allowing it to withstand extreme weather and last a long time. It also has significant advantages over steel when it comes to fire protection, thermal performance, and acoustic characteristics. A typical span-to-depth ratio of a CIP beam is typically around 15 to ensure that the beam conform to the
requirements for live load deflections. For the slab, a conservative estimate is around 30 but this can vary greatly based on the floor being a one or two-way system and whether or not it is constructed as a continuous floor over the beam supports. For a 30’ x 30’, bay this translates to beams with a depth of 24” and an approximate slab depth of 10”-12”. These values can be significantly improved by implementing post-tensioning. However; this means added cost due to the need for specialty contractors to perform the post-tensioning, as the systems can be rather complex and have very strict construction tolerances. The main drawback of CIP systems is the cost and time required for formwork construction, as this is very labor intensive. The formwork may also require shoring to reach longer span lengths, thereby adding additional cost to the project. The subject of quality control is more of a concern with CIP concrete systems too, with issues such as material testing for strength and slump needing to be conducted on site. Weather also poses challenges to CIP systems; common concretes require certain temperatures for curing and typically cannot be poured during rain.

2.2 – Structural Steel Systems

Structural steel is a very common solution to many commercial applications as well. Steel is an attractive option due to the material’s high strength and relatively lower weight compared to concrete. Similar to CIP structures, steel allows for great versatility to the floor plan due to its ability to utilize beam-column frames to resist the lateral loads. Steel joists can also attain very large span lengths. Typically, steel structures will utilize a 2” steel deck with an additional 2” CIP floor. The floor can be either composite or
non-composite with the joists. Steel joist, spaced at approximately 6’ on center, support the floor and these are usually supported by either wide flange beams or steel joists spanning between columns. A 30’ x 30’ bay typically requires steel joist girders around 30” deep. Accounting for the 5” depth of the floor joist’s bearing height and an additional 4” for the floor, the total depth is around 39”. However, the joists allow mechanical chases to run between the webs in the joists, which enables finish ceiling to come up near the bottom of the joists. A standard wide flange (WF) section for this type of system would be a WF18x106. The drawbacks of steel include high material costs, a lead-time on the ordering of materials, and decreased fire and corrosion resistance compared to concrete, unless treated and maintained with fireproofing materials.

2.3 – Precast Systems

Precast concrete is another attractive solution to builders for a variety of reasons. It allows for much greater production quality and consistency of concrete over its CIP counterpart. Precast concrete also does not have the weather concerns that a CIP system is susceptible to because it is typically produced indoors away from the elements. Precast concrete also provides significant benefits in the schedule and cost of a project due to its fast erection time and the repetitious nature of the components. It does not require the setting up of formwork, pouring concrete, and then curing of the concrete to achieve an acceptable level of strength. To maximize these benefits, many precast elements have been standardized by the industry; these include solid slabs, hollow-core slabs, double tees, inverted tees, and L-beams. The cost effectiveness of precast is lost, however, if unusual sizes and shapes must be utilized and sometimes production
capabilities of the plant will not allow for these types of pieces to be produced. Most precast structures will typically consist of hollow-core slabs supported by inverted tee (IT) beams that are then supported by precast columns with corbels. Precast can also be effectively used in conjunction with other structural elements such as steel beams and CIP toppings. The topping can be either non-structural or composite with the precast to give additional strength and continuity to the system. Prestressing also helps to improve the performance of precast concrete. A typical precast prestressed concrete system with 30’ spans will have 6” hollow-core planks with a 2” topping supported by 28” IT beams for a total depth of around 30”. However, unlike joists, mechanical equipment cannot pass through the IT beams so additional depth is needed for drop ceilings to allow the mechanical equipment to pass below the IT beams. Precast concrete typically requires more planning in the preliminary design because pieces are produced off-site. Issues such as connections and penetrations must be coordinated with other trades prior to production. Precast concrete must be transported to the job site, and any cost savings can be lost if transporting pieces long distances. Concrete is significantly heavier than steel and this creates a need for cranes and/or lifting equipment with higher lifting capacities. Once constructed, however, the concrete provides great fire protection, durability, deflection performance, and vibration characteristics.

2.4 – Proprietary & Emerging Systems

There are a variety of systems available that could be considered ‘non-standard’ types of structural systems. The scope here will be limited to systems that are currently available, for residential and commercial applications, and are composite in nature.
2.4.1 – Delta Beam®

The Delta Beam® system is a rapidly emerging shallow floor system produced by Peikko Group which implements a composite beam composed of a hollow steel member formed with steel plates into a trapezoidal shape. This steel beam supports a hollow-core, thin shell slabs, or CIP floor system. The beam is then integrated with a CIP topping which fills all of the voids in the steel beam. This system can span approximately 30’ at a depth of around 18” with a 2” topping for a total depth of approximately 20 inches. This is shallower than precast concrete systems; however, the Delta Beam® requires shoring of each span and this adds costs in time and material to the system. Materials must be purchased for the shoring and floor-to-floor construction times are slowed while waiting for the CIP concrete to obtain adequate strength. The construction of a project using this system is shown in Figure 2.1.

Figure 2.1 – Construction of Deltabeam Floor System with Shoring (Peikko Group, 2012)
Peikko’s web site claims the system has a fire rating of R120 which is a two-hour fire rating. Higher ratings can be achieved with the delta beam but additional fireproofing steps must be taken. Another drawback of this system is that both the columns and beams cannot be continuous simultaneously which creates discontinuities for moment transfer in either the beams or columns.

2.4.2 – Girder-slab

Girder-Slab Technologies has developed the Girder-Slab® System which is a steel and precast composite structural system. It utilizes precast slabs with an integral steel beam to form a composite beam. Extended bottom flanges of the steel beam support the precast planks. Figure 2.2 shows what the typical floor system looks like.

![Figure 2.2 – Typical Girder-Slab Floor System (Girder-Slab Technologies, 2008)](image)

The system is only 8-10” deep, dependent on if a 2” topping is utilized, and provides a flat soffit which allows for mechanical equipment to be easily run without adding more depth to plenum space of the system. An effective bay size for this type of system is a 20’ x 28’ with the girders spanning the 20’ dimension. This is lower than would typically
be expected for office applications and limits the space as well as increases the number of columns required. Due to its shorter span lengths, this product is marketed primarily for residential applications.

2.4.3 – Versa: T Beam®

Diversakore has developed a system similar to the Delta Beam® with a different take on the steel shape utilized to create the composite girder. The steel is formed into a channel with studs anchored to the bottom to help the concrete achieve composite action with the steel. The assembly is reinforced with longitudinal bars and stirrups. The Versa: T Beam® can be used in conjunction with 8” hollow-core planks, hollow-core with a CIP topping (as shown in Figure 2.3), or a steel deck. This method gives it an advantage over traditional CIP systems because the floor system and steel beam act as the formwork for the CIP concrete. The primary disadvantage of this system is again the need to shore the steel members during construction. This system also only has a two-hour fire rating and additional measures must be taken if greater fire-resistance is required.

Figure 2.3 – Diversakore Floor System (Diversakore, 2012)
2.4.4 – Shallow Hollow-Core Floor System

Tested and developed by the University of Nebraska, the Shallow Hollow-Core Floor System is a prestressed concrete system that utilizes precast prestressed IT beams with hollow-core and a CIP topping. This system achieves a better span-to-depth ratio by making the IT beam wider instead of deeper and making all the components continuous throughout the structure. A cross section of the beam can be seen in Figure 2.4. The system achieves continuity through the column by having a key-way for CIP concrete and moment reinforcing. For bay sizes of 32’ x 34’, the system was able to have a beam depth of 16”. Because the system is completely composed of concrete, it can easily achieve at least a two-hour fire rating, but it also causes the members to be much heavier than steel members. This creates the need for larger cranes with greater capacities.

Figure 2.4 – Shallow Hollow-Core Floor System (Tadros and Morcous, 2011)

2.4.5 – Patented Systems

Many patents exist for a wide variety of composite beams and slab systems. Hanlon developed a precast system (US patent 2008/0060293) where columns are cast integrally with large ‘capitals’ at the floor level. The largest system described allows for spans up
to 50’ with a capital depth of 24” and width of 12’ x 12’. The capitals support precast planks which span from capital to capital through the use of bearing connections across the precast joint. All components are preferably prestressed and reinforced with additional mild steel. Figure 2.5 shows a diagram of the systems.

Kim et al. recently developed a system which was published and patented late in 2011 (US 2011/0265422) after this project had been undertaken. It utilizes some of the same concepts to be presented in this project. The girder is composed of a steel WF beam cut in half with a saw tooth type pattern as shown in Figure 2.6. Each T-shaped portion of the steel beam can then be embedded into a prestressed precast concrete beam. A steel
deck with a non-composite CIP topping then creates the floor. The voids in the T-shaped beam are intended to allow mechanical equipment to pass through as can be seen in figure 2.7. A diagram of the girder with cross sections is given in Figure 2.8. Limited technical information is known about the geometry and performance of this system from the patent, however, several key differences exist between this system and the one presented. These differences are addressed in more detail later in Chapter 3 of this document.

Figure 2.6 – T Shaped Steel Cutting (Kim, 2011)

Figure 2.7 – Composite beam with T-Type Steel (Kim, 2011)
120- Steel Web
200- T-shaped steel
400- Inverse T-shaped steel
500- Vertical stiffener
550- Reinforcing
600- Prestressing tendons
700- Casing Concrete
800- Stud

Figure 2.8 – Composite Beam with T-Steel (Kim, 2011)
Chapter 3 – System Overview

3.1 – System Development

The concept for the design of this system revolves around improving the performance of standard precast prestressed concrete structural systems. The proposed system incorporates as many of the advantages as possible from both steel and concrete systems previously described. The span-to-depth ratio of standard precast prestressed IT beams is poor relative to the other systems. Improving on this will be one of the goals of the proposed system. Making the beams wider and adding additional strand can reduce the depth of the precast IT beams, however, the section then becomes much larger and heavier. Also, when using more prestressing strands to increase the moment capacity it becomes difficult for concrete alone to counteract the large tension force generated. This results in the compression block of concrete moving much deeper into the cross section. As the compression block moves deeper, it begins to reduce the effectiveness of the prestressing strand because the strand is not being strained enough to reach its yield stress, and this causes the section to become compression controlled and significantly reduces the moment capacity. The solution to the problem for this proposed system was to introduce a large area of steel in the compression. The steel acts as the primary element utilized for carrying the compressive force during flexure instead of the concrete.

Steel is more expensive than concrete so determining the most effective manner to utilize the steel is extremely important in order to develop an economical solution. Figures 3.1
through 3.4 show the general effect that increasing the area of compression steel has on the moment capacity of a member, divided by the depth (d) of that member. The data shown is for a standard 28IT28 prestressed IT beam made with common 5 ksi and 7 ksi concrete. Several different conditions with varying amounts of prestressing strand were evaluated using strain compatibility to establish when it becomes more effective to employ steel as the compression element instead of concrete. Several determinations can be established from these figures: 1) With lower areas of prestressing strand (Figure 3.1), the compressive capacity of the concrete can match the tension force of the strand by itself and compression steel has little to no value. 2) Increasing the area of compression steel is effective only to a certain extent. Eventually, the prestressing strand becomes fully utilized and additional compression steel no longer increases the section’s moment capacity. 3) For cases where a large area of prestressing is being used, the effects of additional compression steel reinforcement yield the greatest results. In cases like this, the concrete cannot effectively generate an adequate compressive capacity to equal the full tension force of the strand, and the moment capacity is greatly reduced due to the prestressing strand being underutilized. Increasing the amount of steel allows for the complete utilization of the strand’s strength and greater moment capacities.

Figure 3.1 – Effects of Increasing Compression Steel Area ($A_{ps} = 1.53 \text{ in}^2$)
3.2 – System Description

3.2.1 – Structural System Overview

The proposed system is designed to be similar to a precast prestressed concrete structure in its construction, behavior, and performance characteristics. The columns will be...
continuous through the beam connections at each floor level and each column can stretch roughly 35’ or approximately three and a half stories for a typical office building. This would result in two precast column pieces at each grid line for a six-story building. The columns need to carry the gravity loads from the floors above as well as be able to resist significant moments from lateral loads. This is due to two different load cases: 1) They are part of the frame for the lateral load resisting system and 2) They must to be able to resist the negative moments from the continuous floor beams.

A composite steel and prestressed concrete beam, referred to as the T-RECS beam (The Real Easy Composite System), is the primary members used to support the floor system. An elevation of the beam can be seen in Figure 3.5. The T-RECS beam has three primary components:

1) Multiple low-lax prestressing strand (270 ksi)
2) A rectangular precast beam
3) One half of a 50 ksi WF steel beam cut into a form similar to Figure 3.6.

![Figure 3.5 – Typical Beam Elevation](image-url)
The steel beam is cast directly into the top of the prestressed concrete rectangular section and becomes a singular unit with the precast concrete once it has hardened. The T-RECS beams bears on corbels cast into the columns at each story elevation.

Precast hollow-core slabs span from beam to beam and make up the floor system. They bear on the ledge between the edge of the steel flange and the edge of the precast concrete. A structural CIP topping is added once all the components have been put in place. Typical cross sections at ‘A’ and ‘B’ from Figure 3.5 are shown in Figures 3.7 and 3.8 at the final stage of construction which includes the hollow-core planks and CIP topping.
The T-RECS beam is not used at the perimeter of the structure because it is not well suited for cases where high torsion loads are present. Instead, precast prestressed L-beams are utilized where only one side of the beam is being loaded. The L-beams will likely need to be deeper than the T-RECS beams to resist the torsion loads, but it should have little effect on the span-to-depth ratio of the system because the L-beams can be integrated into the walls. Using L-beams at the perimeter also helps when pouring the CIP topping. It creates a projection above the floor level that acts as the sidewalls of the formwork for the CIP topping. Without this projection, construction of formwork would have been necessary around the perimeter.

The structural system can easily be tailored to accommodate any building façade the architect desires, and it is ideally suited for the use of insulated architectural prestressed wall panels. Insulated wall panels can easily be connected to columns and/or perimeter
beams of the structure. They provide excellent durability, thermal insulation, fire protection, and are economical to produce.

### 3.2.2 - Production

All structural components in the system, with the exception of the CIP place topping, are manufactured or assembled in a precast plant. The columns are cast with any corbels and embed plates required to make the connections. They are then shipped to the job site for erection. There are a total of five embed plates needed at each floor level for a typical column connection. Four of the embed plates are used as torsion anchor restraints and the other is a plate assembly used to connect the T-RECS beam to the column. The plate assembly is shown in Figure 3.9 and consists of two flat plates held together by rows of rebar. The spacing of these bars can be altered to allow for the column reinforcement to pass through them.

![Figure 3.9 – Column Embed Detail](image)

The steel beam, which is to be embedded in the precast concrete, is a WF steel beam that has effectively been cut in half. The beam is cut the entire length of the web in the general form shown in Figure 3.6. The beam is being cut in this fashion for four primary reasons: 1) It allows for fully composite behavior between the steel and concrete during flexure and the transfer of horizontal shear. 2) The voids created between the precast concrete and the steel where it has been cut back above the top of the precast allows for
reinforcement to be passed through. This will allow for greater continuity of the hollow-core, thereby improving its capacity. 3) It allows for the easy placement of the CIP topping as the concrete can easily flow from one side of the beam to the other. 4) It makes it possible to use less steel in terms of total weight because it uses a shallower section than would be possible if the beam were to be cut directly in half. Exact dimensions for the cut will vary slightly based on the required WF beam size determined by the design criteria of each building. Each end of the beam needs to be coped to allow for the placement of the backing bar to make the weld between the beam and column. Holes that are 9/16” ∅ are drilled in each ‘tooth’ of the cut beam to allow a #4 bar to be eventually placed through it during production.

When preparing the precast bed to pour the concrete, the necessary size and quantity of stirrups should be threaded onto the 0.6” ∅ strand prior to tensioning. Once the stirrups are placed on the strand, the strand can be tensioned to the appropriate stress and the stirrups spaced as required by design.

The manufactured steel beam should be placed in the bed from above, prior to pouring the concrete and positioned so the top of the flange corresponds to the same elevation as the top of the hollow-core once erected. For example, if 8” hollow-core is required for the design, this means that the top of the steel should be located 8” above the top of the precast concrete. Positioning it this way will allow for a smooth transition from the hollow-core on one side of the beam, to the steel, and then over to the hollow-core on the
other side. Similarly, if 10” hollow-core was to be used, the steel section should be positioned so that the top of the steel is 10” above the top of precast. To assist in positioning of the beam, the holes should be drilled at the proper elevation so that when the #4 bar is threaded through them, it can be tied to the underside of the strand and the steel beam is held in the proper location. The #4 bars also act to interlock the steel beam with the precast concrete and prevent it from pulling out of the precast. Once all of the steel is properly positioned, the concrete can be poured. When the concrete is cured to the appropriate strength the strands can be cut, thereby shifting the tension force of the prestressing strand into a compressive force acting on the concrete and steel beam. The assumption that the steel also carries the compressive force from the prestressing strand is an important one that greatly affects the effective area, centroid, and moment of inertia of the cross section at release. This is an assumption that will be made throughout the design of the beam; however, it is unknown exactly how much of the prestressing force will actually be transferred to the steel.

While the beam at this stage shows a resemblance to the patented system described previously in Chapter 2, several processes and aspects of the beam are different. Figure 2.6 shows the intended manufacturing process for the patented system. The beam is cut longitudinally from the mid-height of each end of the steel beam for a length of L1 before the saw tooth pattern begins. The intent is to then cut off pieces ‘F’ and ‘G’. These two pieces should be of equal length so that once removed, length L2 will be equal to the length L1 at each end of the two beams thus making them identical. Compare this to
Figure 3.6, where the cut does not begin at the mid-height and travels horizontally; instead, the saw tooth pattern begins immediately. A second difference is the method for tolerating variance in the beam length. The proposed system addresses it through an irregularity in the pattern at the mid span of the beam while the patented beam allows for the adjustment to take place by varying the length of the first horizontal cut. Additionally, the proposed system requires that only one additional cut needs to be made at each end, instead of two, to make the halves identical.

The means of anchoring the steel beam into the precast concrete is different as well. The proposed system relies on bars fed through pre-drilled holes in the steel which interlocks with the prestressing stands; whereas the other system uses headed studs (800 in Figure 2.8) welded to each tooth of the steel beam.

The ends of the beam have differences as well. A separate T shaped section (400 in Figure 2.8) must be attached to the bottom half of the beam to recreate an I shape cross section at each end. This is not required by the proposed system. Stiffeners are also being added along the web, as seen in section H – H, which are not required by the proposed system.

When setting up the formwork to pour concrete, the proposed system utilizes a void form at each end of a standard prestressing bed to create an offset from the end to the steel to the end of the concrete. Section E – E in Figure 2.8 shows that steel plates are welded to
the beam a certain distance from the end. These plates are intended to allow the strand to pass through them as well as act as the end-caps for the formwork. The proposed system uses the standard techniques that would be utilized for any typical prestressed beam.

### 3.2.3 – Erection & Construction

The T-RECS structural system is one that is extremely easy and straightforward to construct. Once the columns are erected, beams can be put into place on bearing pads on the column corbels. When the beam is properly positioned, loose angles are welded to the torsion embed plates in the column previously described. These angles act to prevent the beam from overturning during erection of the hollow-core and they ensure the entire torsion load is resisted by only the precast portion of the T-RECS beam.

There are three distinct loading situations that the T-RECS beam must be designed for during construction of the building. The first load case to be considered occurs when all of the hollow-core for one bay has been put in place. This results in an unbalanced load where one side of the beam is loaded with the dead weight of the hollow-core, and the other is not. This creates a torsion load that the T-RECS beam and the connections must be designed to resist. Next, rebar should be fed through the openings in the steel web and into the voids in the hollow-core. This rebar will be used to help create greater continuity of the structural system. After the rebar has been put in place, the hollow-core for the other bay can be erected. Once all of the precast components have been put in place, the connection of the steel beam to the precast embed can be made, as shown in a column section in Figure 3.10, and as a section through the beam in Figure 3.11. The connection
to the column is made by placing a backing bar in the coped portion of the steel beam near the flange and then making a plug-fillet weld over the full length of the joint. This whole process can then be repeated for each level of the structure.

Loads that the beam must resist during the second load case include the beam’s self weight, the weight of the hollow core, and a construction live load. This load case is present until placement of the CIP topping. The non-composite beam’s ability to resist these loads gives the erector the choice and flexibility of when to pour the topping at each floor level or to continue the erection of precast components for the story above. This type of flexibility is major advantage over traditional CIP which requires waiting for the concrete to gain adequate strength before continuing to the next floor. The third load case occurs when pouring the CIP topping. The loads at this stage consist of the beam’s self weight, the weight of the hollow-core and the weight of the CIP topping. Under all of these load cases the beam behaves as a simple span with only the cross section of the non-composite T-RECS beam resisting the loads. A minimal amount of formwork is required to be constructed alongside the edge of the T-RECS beam and column to contain
the concrete that flows into the gap between the precast beam and the face of the column. The formwork can be anchored into the column and removed once the concrete has hardened.

Figures 3.12 - 3.18 on the following pages detail in 3D representations the construction of the proposed system.

Figure 3.11 – Typical Column Connection

Figure 3.12 – Erect precast columns cast with corbels and embed plates
Figure 3.13 – Erect T-RECS beams and L spandrels

Figure 3.14 – Make connection to precast column and install torsion restraints
Figure 3.15 – Place hollow-core on one half of beam & feed rebar into hollow-core

Figure 3.16 – Place second bay of hollow-core & reposition rebar
Figure 3.17 – Position topping reinforcement over beams & around columns

Figure 3.18 – Pour CIP topping
Additional differences between the proposed system and the patented system from the literature review lie in the member connections. Figure 2.7 shows a typical bolted connection for the patented system which allows for continuous beams. These beams can also be connected to the columns using a standard bolted connection. The structural system is intended to be used in conjunction with a metal deck and CIP slab. This is also different than the hollow-core floor of the proposed system, and there is no intent to allow for the splicing together of two beams as is possible in the patented system.

3.2.4 – Service

Once the CIP topping hardens, it becomes a fully composite cross section with the T-RECS beam. The hardening of the CIP topping creates a moment connection between the beams and columns so that the structure now behaves as a moment frame with continuous beams. Due to the continuous beam loading, significant negative moments are generated at the face of the column. This moment must be resolved into the precast connections. The weld between the steel beam and precast column embed carries the tension force generated by these negative moments. Similarly, the compression force from the negative moment is carried by the hardened CIP topping between the precast column and the precast concrete of the T-RECS beam.

At this point, any superimposed dead loads such as the mechanical, electrical, plumbing, flooring, drop ceilings, and partitions can now be installed. These loads as well as the live loads applied to the system are resisted by the fully composite cross section which
now includes the CIP concrete for the in-fill between hollow-core planks and the effective width of the CIP topping flange as prescribed in the ACI 318 (2008).

There are also distinctions in the design intent between the patented system and the proposed one. The patented system is intended to be predominately a steel structure instead of concrete as evidenced by the connections and exposed steel web and top flange of the beam. It can be seen in Figure 2.7 that the designer wants to allow for M/E/P equipment to be able to pass through the voids in the steel web of the beam. This is quite different than the proposed system where reinforcing is passed through these openings, filled with concrete, and M/E/P systems must pass under the beam. The proposed system also does not have any exposed steel surfaces after pouring to the topping which is different that the patented system where the flange and web remain exposed.

There are many advantages to the proposed system discussed in this chapter:

- All exposed surfaces are concrete. This gives the system superior fire performance characteristics over steel.
- The system utilizes moment connections at the columns. This allows for moment frames to be formed with the columns thereby eliminating the need for shear walls in the direction of the frame. Moment connections also make it possible for reductions in the moment demand due to continuous beam loading.
- The proposed system is able to achieve beam continuity while using continuous columns.
• Cutting the steel beam in half in the manner proposed allows for a large reduction in the amount of steel required.

• The system uses all precast components with the exception of a CIP structural topping, which may be poured at any point prior to the installation of any superimposed dead loads.

• Having all precast components greatly increases the quality and allows for an expedited construction schedule.

• Perimeter L-beams act as formwork for the CIP topping thereby eliminating this costly and time-consuming process for typical CIP structures.

• The system is able to achieve span-to-depth ratios of over 20, which is an improvement over standard precast products.

• Field connections are simple, requiring only three welds at each column connection.

• Beams are able to resist construction loads and do not require the use of shoring during the construction process.
Chapter 4 – System Analysis & Design

4.1 - Design Criteria

The structural system developed in this paper is tailored toward applications such as residential and commercial buildings. The design criteria for these types of structures generally call for an approximate column-to-column spacing of 30’ in each direction. The floor plan and a typical building section for the structure being designed for are shown in Figures 4.1 and 4.2 respectively. The overall plan dimensions for the structure are 124’ x 120’. Each story has a height of 10’, and the building is six stories tall for a total height of 60’. Span lengths for the beams are 30’ at the exterior and 32’ at the interior. In the opposite direction, the spans lengths for the hollow-core are 30’. Span lengths at the exterior have been shortened to account for the larger moments that are present at the end spans of continuous beams.

In addition to the self weight of the beams, the hollow-core, and CIP topping, a superimposed dead load (SIDL) of 15 psf is applied over the entire floor. This is to account for any M/E/P, additional flooring, drop ceiling loads, or partitions that may be required for typical office applications. ASCE 07 (2005) Table 4.1 requires that offices be designed for a minimum uniformly distributed live load of 50 psf at each floor level. The hollow-core and beams are design to meet the deflection limit of L/360 for applied live loads according to ACI 318 (2008) Table 9.5(b). For a 30’ span, this requires that the service live loads do not cause deflections of more than 1”. An occupancy Category
of II has been assumed for the building. The chosen location for the building is in the western portion of the state of Iowa.

Non-standard manufacturing techniques or special materials would greatly add to the cost of producing the system and would limit the proposed system’s commercial viability. To address this, an additional restriction imposed on the design is to limit the materials to standard materials that can be economically procured by, or produced at, typical prestressing plants. With this in mind, the following is a list of materials used for the design:

- **Precast concrete:** compressive release strength = $f_{ci}' = 6500$ psi
  
  28-day compressive strength = $f_c' = 8000$ psi

- **Prestressing strand:** ASTM A416, 0.6ø 270 ksi low-relaxation strand

- **Reinforcing bars:** ASTM A615, Grade 60 ($F_y = 60$ ksi)

- **Structural steel:** ASTM A992, $F_y = 50$ ksi, $F_u = 65$ ksi

- **Cast-in-place concrete:** 28-day compressive strength = $f_c' = 4000$ psi

- **Welding electrodes:** conform to AWS D1.4 with E70XX or E80EXX electrodes

- **Plates, angles, channels, all-thread rod and other misc. shapes shall be ASTM A36 steel ($F_y = 36$ ksi, $F_u = 58$ ksi), UNO.

- **Columns:** 28-day compressive strength = $f_c' = 5000$ psi

- **Hollow-Core:** 28-day compressive strength = $f_c' = 5000$ psi
Figure 4.1 – Typical Floor Plan

Figure 4.2 – Typical Building Section
Design procedures were conducted in accordance with the following design codes and manuals:

- American Society of Civil Engineers “ASCE 7-05 Minimum Design Loads for Buildings and Other Structures”, 2005
- American Concrete Institute (ACI) “Building Code Requirements for Structural Concrete (ACI318-08) and Commentary”, 2008

4.2 – Gravity Loads
The criteria previously described are used in designing the members for the gravity loads on the system at three separate stages: Stage 1 - production, Stage 2 - erection/construction, and Stage 3 - service. The member designs are detailed in the following sections.

4.2.1 – Hollow-Core Design
Hollow-core is an economical, easy to produce, and easily erected product. The hollow-core for the proposed system is similar to any conventional floor system. It comes in standard depths of 8”, 10”, and 12”, with a standard width of 4’. Design tables are available from precast manufacturers and can be used to determine the appropriate size needed for the applied loads and span length of the member. The hollow-core is designed as simply supported for gravity loads applied prior to pouring of the topping. Once the
CIP topping is poured, the continuity reinforcement previously described helps to make the hollow core continuous over the beam supports for the superimposed dead and live loads. This helps to give the hollow-core additional capacity. Stain compatibility was used to ensure adequate negative moment capacity at these supports for the continuous live loads. The following table is taken from the PCI Handbook (2004).

### 3.6 Hollow-Core Load Tables (cont.)

<table>
<thead>
<tr>
<th>Strand Pattern Designation</th>
<th>Section Properties</th>
<th>No Topping</th>
<th>2 in. topping</th>
</tr>
</thead>
<tbody>
<tr>
<td>76-S</td>
<td>A = 215 in²</td>
<td>f = 1600 lb/ft²</td>
<td>3071 lb/ft²</td>
</tr>
<tr>
<td></td>
<td>j = 3000 lb</td>
<td>j = 4000 lb</td>
<td>7200 lb</td>
</tr>
<tr>
<td></td>
<td>Si = 417 lb/ft</td>
<td>S = 531 lb/ft</td>
<td>932 lb/ft</td>
</tr>
<tr>
<td></td>
<td>wi = 221 lb/ft²</td>
<td>Ctl = 36 lb/ft</td>
<td>8 lb/ft</td>
</tr>
<tr>
<td></td>
<td>V/S = 1.02 in</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### Key
- N = Total superimposed service load, lb/ft²
- E = Estimated moment of reaction, in.
- D = Estimated load per camber, in.

#### Notes:
- Stain compatibility is used to ensure adequate negative moment capacity at these supports for the continuous live loads.
- The following table is taken from the PCI Handbook (2004).

#### Table of auto superimposed service load, lb/ft², and cambers, in.

| Span, ft | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 | 26 | 27 | 28 | 29 | 30 | 31 | 32 | 33 | 34 | 35 | 36 | 37 | 38 | 39 | 40 |
|----------|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|
| 76-S     | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 | 448 |

### Table of auto superimposed service load, lb/ft², and cambers, in.

| Span, ft | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 | 26 | 27 | 28 | 29 | 30 | 31 | 32 | 33 | 34 | 35 | 36 | 37 | 38 | 39 | 40 |
|----------|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|
| 66-S     | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 | 450 |
| 78-S     | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 | 620 |

Strength is based on stain compatibility; bowl tension is limited to 1.25E-3 times 3.0 minus 3.0 for explanation. See [3.3.2](#) for explanation of vertical tension.

#### Notes:
- Stain compatibility is used to ensure adequate negative moment capacity at these supports for the continuous live loads.
- The following table is taken from the PCI Handbook (2004).
The boxed region in the table indicates the appropriate hollow-core strand configuration for a 30’ span with a superimposed service load of 85 psf. A 4HC8+2 with a strand designation code 58-S satisfies the design requirements. This describes an 8” deep hollow-core plank with a 2” CIP structural topping and containing 5 - ½” ø strands. The typical cross section for the hollow-core is shown in Figure 4.3.

4.2.2 - T-RECS Beam Design

The T-RECS beam is a prestressed member, and because of this, both stress design and ultimate strength design must be considered. The concrete should be designed to remain uncracked throughout the various stages of loading. PCI Handbook (2004), Chapter 4 details the design requirements that must be met. The stress limits used are 0.7 $f_c'$ for compression and $12\sqrt{f_c'}$ for tension. Service loads for the various design stages can be seen in Figures 4.4 through 4.7. A summary of the section properties for each stage is provided in table 4.1. Refer to the appendix for detailed calculations of these properties.

<table>
<thead>
<tr>
<th>Table 4.1 - Summary of Section Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>-----------------</td>
</tr>
<tr>
<td>Stage 1</td>
</tr>
<tr>
<td>Stage 2</td>
</tr>
<tr>
<td>Stage 3</td>
</tr>
</tbody>
</table>

Figure 4.3 – Composite Hollow-Core Section
Figure 4.4 – Stage 1 Loads at Release

Self weight = 0.183 k/ft
P/s force = 505 k
506 k

Figure 4.5 – Stage 2 Loads During Erection

Const LL = 20 psf
HC = 56 psf

Figure 4.6 – Stage 2 Loads During CIP Pour

HC = 56 psf
Concrete = 0.129 k/ft
IP topping = 24.2 psf

Figure 4.7 – Stage 3 Loads at Service Life

Live load = 50 psf
SIDL = 15 psf
4.2.2.1 - Stage 1 – Production

The precast member must be designed for the first stage, which is at release of the prestressing strand. At this stage the member is analyzed as a simple span member. It is unknown exactly how much of the prestressing force is transferred to the steel beam once the prestressing force is released. The assumption made for this analysis is that the steel member is fully composite with the precast concrete; therefore, the section properties include the transformed area of the steel based on the modular ratio. Cross-sections detailing the T-RECS beam used for the design can be seen in Figures 4.8 and 4.9. Reference the elevation view in Figure 4.10 for locations of section cuts.

A W18x76 was the steel section selected for the design. Figure 4.11 details the dimensions for how the steel beam should be cut to allow for proper production. 12 – 0.6”ø strands, tensioned to 0.78 $f_{pu}$, are used to achieve the appropriate precompressive force in the concrete for stress design and moment capacity the strength design. The stress design for the beam is detailed in Table 4.2. A final concrete strength of 8 ksi with a release strength of 6.5 ksi are required to meet the stress limits for the design. Only the transformed area of the steel flange is used in the analysis, due to the uncertainty of how the cut portion of the web would behave when subjected to the prestressing force. This is a conservative assumption and should not result in an overestimation of the member’s capacity. Initial stress losses in the prestressing strand due to anchorage seating and elastic shortening of the concrete were assumed to be approximately 7%. Analysis initially shows that stresses at the ends of the member are above the acceptable stress
limits for the concrete at release. To address this, the two strands in the top row are debonded at each end for 5’ which reduces the stress at the ends of the member to acceptable limits.

Table 4.2 - Stress Check at Release

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>P/A</td>
<td>16.504</td>
<td>2.644</td>
<td>16.504</td>
<td>2.644</td>
</tr>
<tr>
<td>Pe/S</td>
<td>-19.085</td>
<td>2.048</td>
<td>-19.085</td>
<td>2.048</td>
</tr>
<tr>
<td>M/S</td>
<td>2.905</td>
<td>-0.312</td>
<td>2.179</td>
<td>-0.234</td>
</tr>
<tr>
<td>Σ</td>
<td>0.324</td>
<td>4.380</td>
<td>-0.402</td>
<td>4.458</td>
</tr>
<tr>
<td>Limit</td>
<td>50.000</td>
<td>4.550</td>
<td>50.000</td>
<td>4.550</td>
</tr>
<tr>
<td>Check</td>
<td>OK</td>
<td>OK</td>
<td>OK</td>
<td>OK</td>
</tr>
</tbody>
</table>

Figure 4.8 – Cross Section at A

Figure 4.9 – Cross Section at B
Figure 4.10 – T-RECS Beam Elevation with Reinforcement

Figure 4.11 – WF Beam Manufacturing Detail
4.2.2.2 - Stage 2 – Erection

During the erection of the precast, three loading cases need to be analyzed. All loads during the erection of the precast are temporary load cases, and therefore a load factor of 1.0 is applied.

The first case occurs when one half of the beam is loaded with the weight of the hollow-core. This unbalanced load results in torsion being applied to the beam and the precast must be designed to resist this load. When analyzing the beam for torsion, strength of the steel beam was neglected because the torsional strength of the rectangular precast is much greater relative to the T-shaped steel section embedded in the top of the concrete. The torsion angles previously described are provided to ensure that the entire torsion force is resisted by the precast concrete and that the beam is restrained from overturning. Torsion loads were analyzed using the PCI Handbook (2004) recommended method developed by Zia and McGee. The detailed calculation of the torsion analysis is presented here and the design of the torsion restraints is detailed in the connection design section of this report. Results from the analysis require the use of 1.28 in$^2$/ft not spaced at greater than 10” O.C. Stirrup placement can be seen by referencing back to Figure 4.10. Stirrups are #4 bars spaced at 1½” on center which equals 1.6 in$^2$/ft for the first segment of the beam where torsion is the greatest. The number of stirrups can be reduced closer the midspan where the torsion is theoretically equal to zero. The torsion design is conducted as follow:
Torsion Design

Note: Uses simplified PCI method which assumes $f_{se} > 40\% f_{pu}$

Beam Geometry

- Width of stem $b_w = 24$ in
- Beam height $h = 6.0$ in
- Depth to prestressing reinforcement $d = 4.0$ in
- Moment of Area $\Sigma x y = 864$ in$^3$
- Short side of closed tie $x_1 = 5$ in
- Long side of close tie $y_1 = 20$ in

Material properties

- Compressive strength of concrete $f'c = 8$ ksi
- Yield strength of reinforcement $f_y = 60$ ksi
- Concrete density modifier $\lambda = 1.0$
- Strand transfer length $l_t = 30$ in
- Area of prestressing strand $A_{ps} = 2.604$ in$^2$
- Ultimate tensile strength of strand $f_{pu} = 270$ ksi
- Stress @ centroid of concrete after p/s losses $f_{pc} = 1.75$ ksi

Design load data

- Factored vertical load at critical section $V_u = 12.3$ kips
- Factored moment at critical section $M_u = 92.4$ kip_in
- Factored Torsional moment $T_u = 130$ kip_in
- Section under investigation $l = 3$ in
- Strength reduction factor $\phi = 0.75$
**Preliminary Torsion Check**

Prestressing factor

\[ \gamma = \sqrt{(1+10 \times f_{pc}/f'_c)} = 1.79 \]

Torsion neglecting limit

\[ T_{\text{umin}} = \phi(0.5 \times \lambda \times (f'_c \times 1) \]

\[ \text{lbs/in}^2 \times \Sigma x2y) \times \gamma = 51.74 \text{ kip_in} \]

NG - Member must be analyzed for torsion

**Check Shear & Torsional Limits**

Torsion factor

\[ K_t = \gamma(12-10 \times f_{pc}/f'_c) = 17.52 \]

Geometric torsion factor

\[ C_t = 1 \times \text{in} \times b_w \times \Sigma x^2 y \]

Maximum torsional limit

\[ T_{\text{max}} = (1/3) \times K_t \times \gamma \times (f'_c \times 1 \text{ lbs/in}^2) \times \Sigma x^2 y / (1+(K_t \times V_u/(30 \text{ in}^2)) \]

\[ = 404 \text{ kip_in} \]

Maximum shear limit

\[ V_{\text{max}} = 10 \times \lambda \times (f'_c \times 1 \text{ lbs/in}^2) \times b_w \times d / \sqrt{(1+(30 \text{ in}^2) \times C_t \times T_u} \]

\[ = 42.93 \text{ kip} \]

Support shear strength

\[ V_{\text{sup}} = 3.5 \times \lambda \times (f'_c \times 1 \text{ lbs/in}^2) \times b_w \times d \]

\[ = 30.05 \text{ kip} \]

Transition zone strength at ‘x’ kip

\[ V_{\text{x}} = V_{\text{sup}} + (V_{\text{max}} - V_{\text{sup}}) \times \min(l_1, l_t) / l_t \]

\[ = 31.34 \text{ kip} \]

**Concrete Shear & Torsion Design**

**Shear Strengths**

Nominal shear strength

\[ V_c = (0.6 \times \lambda \times \sqrt{(f'_c \times 1 \text{ lbs/in}^2)} + (700 \text{ lbs/in}^2) \times \]

\[ \times (1+10 \times f_{pc}/f'_c) \times \Sigma x^2 y) \times \gamma = 40.93 \text{ kip} \]

Maximum shear strength limit

\[ V_{\text{cw max}} = 5 \times \lambda \times \sqrt{(f'_c \times 1 \text{ lbs/in}^2)} \times b_w \times d = 42.93 \text{ kip} \]

Minimum shear strength limit

\[ V_{\text{cw min}} = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ lbs/in}^2)} \times b_w \times d = 17.17 \text{ kip} \]

Support shear strength

\[ V_{\text{cw sup}} = 3.5 \times \lambda \times \sqrt{(f'_c \times 1 \text{ lbs/in}^2)} \times b_w \times d = 30.05 \text{ kip} \]

Transition zone strength at ‘x’ kip

\[ V_{\text{x}} = V_{\text{cw sup}} + (V_{\text{cw max}} - V_{\text{cw sup}}) \times \min(l_1, l_t) / l_t \]

\[ = 31.34 \text{ kip} \]

**Controlling value**

Nominal shear strength

\[ V_{nx} = 31.34 \text{ kip} \]

\[ V_c = V_{nx} = 31.34 \text{ kip} \]

**Torsional Strength**

Nominal torsional strength

\[ T_c = 0.8 \times \lambda \times \sqrt{(f'_c \times 1) \]

\[ \times (1+10 \times f_{pc}/f'_c) \times \Sigma x^2 y(2.5 \times \gamma-1.5) = 183.21 \text{ kip_in} \]

**Shear-Torsion Interaction**

Combined concrete torsional moment strength

\[ T_c = T_c/\sqrt{(1+(T_c \times V_u/(T_u \times V'_u)))} \]

\[ = 160.32 \text{ kip_in} \]

Combined concrete shear strength

\[ V_c = V_c/\sqrt{(1+(V_c \times T_u/(V_u \times T'_u)))} \]

\[ = 15.17 \text{ kip} \]

**Reinforcement Design**

Closed stirrup design

Spacing requirements

\[ s = \min(x_1+y_1/4, 12 \text{ in}) = 10 \text{ in} \]

\[ s_{\text{prov}} = 4 \text{ in} \]

Pass - Spacing provided is adequate
\( \alpha_t = \min((0.66 + 0.33 \times \frac{y_1}{x_1}), 1.5) = 1.5 \)

\( A_t = \left( \frac{T_u}{\phi} - T_c \right) \times s_{prov} \times (\alpha_t \times x_1 \times y_1 \times f_y) = 0.01 \) in²

Area of shear steel required

\( A_{vreq} = \left( \frac{V_u}{\phi} - V_c \right) \times s_{prov} \times f_y \times d \) = 0.02 in²

Minimum area of shear steel required

\( A_{vmin} = A_{ps} \times f_{pu} \times s_{prov} \div (80 \times f_y \times d \times (\sqrt{d/b_w})) \)

Total area of stirrup reinforcing required

\( A_s = \max(A_{vreq}, A_{vmin}) = 0.06 \) in²

Controlling area of shear steel

\( A_{v} = \max(A_v, 2 \times A_t, 50 \text{ lbs/in}^2 \times b_w \times s_{prov} / f_y, 200 \text{ lbs/in}^2 \times b_w \times s_{prov} / f_y) = 0.32 \) in²

*Shear reinforcement (user input)

\( D_{stir} = 0.5 \) in

Area of horizontal reinforcement provided

\( A_{prov} = 2 \times \pi \times D_{stir}^2 / 4 = 0.393 \) in²

Pass - Torsion reinforcing is adequate

Longitudinal reinforcement design

Required longitudinal reinforcement

\( A_{l1} = 2 \times A_t \times (x_1 + y_1) / s_{prov} = 0.07 \) in²

\( L = \max(2 \times A_t / s_{prov}, \min(200 \text{ lbs/in}^2 \times b_w / f_y, 50) \)

\( A_{l2} = (400 \text{ lbs/in}^2 \times b_w / f_y \times (T_u / (T_u + V_c / (3 \text{ in}^2 \times C))) - L) \times (x_1 + y_1) \)

Controlling longitudinal steel area

\( A_l = \max(A_{l1}, A_{l2}) = 1.3 \) in²

*Longitudinal reinforcement (user input)

Area of prestressing strand

\( A_{ps} = 0.217 \) in²

Number of longitudinal bars

\( n_l = 12 \)

Area of longitudinal reinforcement provided

\( A_{lProv} = n_l \times A_{ps} = 2.604 \) in²

Pass - Torsion reinforcing is adequate

Design Summary

~Beam height of 6.00" by 24.0" wide with 8000 psi concrete
~Section under investigation is located at 0.25 ft from support
~Reinforcement at section under investigation consists of 0.50 in dia vertical stirrups spaced at 4 in O.C.
~Longitudinal reinforcement consists of 12-0.60 in dia strand.

Once all of the hollow-core has been erected, two additional load cases must be checked. These loads are detailed in Figures 4.5 and 4.6 and the controlling load is shown in Figure 4.12. In this design example, the load case during the pouring of the CIP topping
is the critical load case. Shear and moment diagrams for this load case are given in Figures 4.13 and 4.14 respectively. The clear span distance is 30\' for the interior spans and 28\' at the exterior spans.

At this stage, the entire shear force must be resisted by the precast concrete alone. This is because the loads are applied directly to the precast and the steel beam is not subjected to any shear until the CIP topping has hardened. Design of the precast is done in accordance with the PCI Design Handbook (2004). For a detailed calculation of the shear design, see the Appendix. The design requires 0.72 in$^2$/ft. Reinforcement that was utilized to resist the torsion loads that are no longer present can now be considered as
reinforcement for the shear strength of the member. The amount of reinforcement required for torsion was 1.6 in$^2$/ft, which exceeds the 0.72 in$^2$/ft required here for shear.

When analyzing the moment strength of the T-RECS beam during construction, the moment strength of the prestressing strand is not a concern because it is designed for higher factored loads during the service life of the member. However, the strength of the steel beam during this stage must be checked. The steel beam acts as the compression flange during flexure until the CIP topping has hardened. Because the steel beam is anchored in concrete at its base and has portions of the web removed, it is uncertain exactly how the steel behaves under loading. To help simplify this analysis, the steel beam is analyzed similar to any typical steel beam. This is likely a conservative assumption because a beam during flexure often undergoes lateral torsion buckling before achieving the full plastic strength of the beam. Lateral torsional buckling is a phenomenon where failure of a beam takes place normal to the plane of bending while also twisting about its shear center. This is not likely with the T-RECS beam due to the significant differences in the cross section and material properties. For instance, the moment of inertia in the weak axis for the T-RECS beam is significantly larger than a standard steel section and therefore global buckling of the beam in a lateral direction is extremely unlikely. It is possible, however, that this could occur locally in the compression flange in some similar manner. However, to be conservative the steel beam is still analyzed as if it were a W18x76 steel section unbraced for its entire length. The
results for this design are shown in table 4.3 and the detailed analysis can be seen in the Appendix.

<table>
<thead>
<tr>
<th>Table 4.3 - Flexure Analysis of Steel Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment capacity</td>
</tr>
<tr>
<td>Moment demand</td>
</tr>
</tbody>
</table>

Stress levels in the prestressed concrete are also checked at this stage even though it is fairly safe to assume that stress levels will be adequate because much higher loads are present during the service life. The results of the stress analysis for an interior span are shown in table 4.4. Section properties used in the calculations are based on those for Stage 2 in table 4.1. The section properties vary slightly from stage 1 due to changes in the concrete strength from 6.5 ksi at release to 8 ksi at final strength.

<table>
<thead>
<tr>
<th>Table 4.4 - Stress Check at Erection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mid-Span</td>
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<tr>
<td></td>
</tr>
<tr>
<td>Steel Top (ksi)</td>
</tr>
<tr>
<td>P/A</td>
</tr>
<tr>
<td>Pe/S</td>
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<tr>
<td>M/S</td>
</tr>
<tr>
<td>$\Sigma$</td>
</tr>
<tr>
<td>Limit</td>
</tr>
</tbody>
</table>

Check **OK** **OK**

4.2.2.2 - Stage 3 – Service Life

The third and final stage, which must be analyzed, is during the service life of the structure. At this stage, the CIP topping has hardened and is fully composite with the T-RECS beams. Any superimposed dead and live loads now act on the fully composite section. The structure now behaves as a moment frame with continuous beams and
columns, producing negative moments at the column connections. Refer to Figure 4.7 for the service loads applied at this stage for the example building.

First, the beam is analyzed for ultimate strength design. Figures 4.15 through 4.17 show the uniform factored loads applied prior to Stage 3 and the shear and moment diagrams for those loads. The factored loads, shear, and moment diagrams for Stage 3 are shown in Figures 4.18 through 4.20. By using superposition, the loads previously applied are combined with the post-composite loads of Stage 3 shown in Figure 4.18 to get the total factored load applied to the beams. The final shear and moment diagrams for the design can similarly be obtained by superposition of the shear and moment diagrams for Stages 2 and 3.

![Figure 4.15 – Stage 2 – Factored Load Diagram (Pre-Composite)](image1)

![Figure 4.16 – Stage 2 – Factored Shear Diagram (Pre-Composite)](image2)

![Figure 4.17 – Stage 2 – Factored Moment Diagram (Pre-Composite)](image3)
The assumed load case for the design at this point is 1.2D + 1.6L. The controlling load case will be verified upon completion of the lateral analysis conducted in subsequent sections. The final, fully composite cross section of the T-RECS beam is shown in Figures 4.21 and 4.22. This section is analyzed for its shear capacity, moment capacity, and a deflection performance. Details of the column connection with component sizes are provided in Figures 4.23 and 4.24.
Figure 4.21 – Fully Composite Cross Section

Figure 4.22 – Full Composite Cross Section at Web Opening

Figure 4.23 – Typical Connection at Column Section
Some simplifying assumptions have been made to make calculating the shear capacity for the cross section easier because the member is a combination of prestressed concrete, reinforced concrete, and a steel beam and current design manuals to not provide specific guidelines to analyze a beam of this nature. To perform the analysis, the beam is assumed to be a prestressed concrete beam with a concrete strength equal to the topping strength because the majority of the depth of the member is the CIP topping. Also, there are too many irregularities in the steel beam to assume consistent performance for analysis as a steel member. These are conservative both fairly conservative assumptions. Completely ignoring the shear capacity of the steel is a too conservative approach so a methodology for incorporating the additional shear capacity of the T-shaped steel section must be considered as well. The assumption made is that each longer tooth of the steel beam is the equivalent of a stirrup with an area of steel equal to the cross sectional area of the stem. For this example, the stem has a minimum width of 3” with a thickness of 0.425” for a total area of 1.275 in². Each tooth is located at 12” O.C. which results in an area 1.275 in²/ft. This is likely a conservative assumption as well because shear stress is a maximum near the neutral axis and the 3”
width used is located near the bottom of the beam. A detailed calculation for the shear design can be found in the Appendix.

Due to the continuous loads applied to the floor, the section must be analyzed for both positive and negative flexure. Flexural analysis of the beam for strength design is again done using strain compatibility. For the positive moment design, the area of the T-shaped flange is considered to act as longitudinal compression reinforcement. For the cross section shown, this area is equal to 7.23 in$^2$. An additional area of steel at the transition from the flange to the web is also included in the calculation. The effective width of the 2” CIP topping also acts as a compression element. This width is calculated according to ACI 318-08, (2008) and is equal to 52” wide. The two layers of strand, as detailed in the cross section, are the tension elements during flexure.

The negative moment at the face of the column relies on the steel flange to act as tension reinforcement. This tension force is then transferred into the column through the weld to the precast embed plate. The area of the rebar in the precast embed plate must be able to carry this force to ensure that the steel flange can be fully developed. The capacity of this connection is calculated in Section 4.5.1. Concrete from the CIP topping fills in the void between the precast beam and the column, and this concrete acts at the compression element for the negative moment. Both the negative and positive moment capacities are calculated in detail in the Appendix. The results of the design for an interior span are
tabulated in Table 4.5. The governing value for the negative moment is found after completion of the lateral analysis.

<table>
<thead>
<tr>
<th>Table 4.5 - Strength Design of T-RECS Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear capacity - ( \varphi V_n )</td>
</tr>
<tr>
<td>Shear demand - ( V_u )</td>
</tr>
<tr>
<td>Moment capacity - ( \varphi M_n (+) )</td>
</tr>
<tr>
<td>Moment demand - ( M_u (+) )</td>
</tr>
<tr>
<td>Moment capacity - ( \varphi M_n (-) )</td>
</tr>
<tr>
<td>Moment demand - ( M_u (-) )</td>
</tr>
</tbody>
</table>

Stresses in the precast are analyzed at this stage as well. Service level load, shear, and moment diagrams applied during Stage 3 for the stress design are shown in Figures 4.25 through 4.27. The total loads for stress design are obtained by superposition of loads from Stage 2, (Figures 4.12 – 4.14) with those shown below.

![Figure 4.25– Stage 3 Service Loads](image)

![Figure 4.26– Stage 3 Service Load Shear Diagram](image)

![Figure 4.27– Stage 3 Service Load Moment Diagram](image)
The final stress analysis assumes 15% losses in the prestressing strand stress due to additional relaxation of the strand, creep of the concrete, and shrinkage of the concrete. Checks are performed under the total and sustained loads. For the tensile stresses at the bottom of the precast section, the limit is $12\sqrt{f_c'}$. This value is the limit of the transition zone between cracked and uncracked concrete, and using this limit allows for the gross moment of inertia of be used at all stages of design. If the stresses were not limited so that the concrete remains uncracked, the changes in the moment of inertia due to cracking would need to be accounted for in the design. The limit for the compressive stress on the CIP topping is $0.7f_c'$. All dead loads prior to achieving the fully composite section are resisted by the construction stage cross section (re: Table 4.1) and the superimposed dead and live loads are resisted by the composite stage section properties. Table 4.6 summarizes the results from the service life analysis. Figures 4.28 and 4.29 summarize the stresses and strain values for the composite transformed cross section relative to 8 ksi concrete for each stage.

<table>
<thead>
<tr>
<th>Table 4.6 - Stress Check at Service Life</th>
<th>Mid-Span (+)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Service</td>
</tr>
<tr>
<td>P/A</td>
<td>0.000</td>
</tr>
<tr>
<td>P_e/S</td>
<td>0.000</td>
</tr>
<tr>
<td>M_DL/S</td>
<td>0.000</td>
</tr>
<tr>
<td>M_SIDL/S ^a</td>
<td>0.143</td>
</tr>
<tr>
<td>M_LL/S ^a</td>
<td>0.477</td>
</tr>
<tr>
<td>\Sigma</td>
<td>0.620</td>
</tr>
<tr>
<td>Limit</td>
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<tr>
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<td><strong>OK</strong></td>
</tr>
</tbody>
</table>

a. uses composite section modulus
Figure 4.28–Stress Diagrams for Transformed Cross Section (8 ksi)
Figure 4.29– Strain Diagrams for Transformed Cross Section (8 ksi)
4.2.3 – Precast Column Design

Similar to the hollow-core, the precast columns can be designed according to manufacturer’s load tables. A square column was chosen to maximize the efficiency of production and to make the column-to-column connections as easy as possible to construct. The design should conform to ACI 318 (2008) Chapter 10. Columns are cast with integral corbels to support the structural beams. The columns for this design are 24” x 24” column. Moments applied to the columns are due to the columns acting as part of the frame for the lateral load resisting system. These loads are detailed later in the lateral analysis portion. A factored axial load of 901 kips and bending moment of 271 k-ft are the design loads for the column at the first floor. The following interaction load tables (Figures 4.28 and 4.29) were taken from the PCI design handbook (2004).

Figure 4.30– Interaction Diagram for Prestressed Concrete Column
One is for 24” x 24” reinforced concrete columns and the other is for 24” x 24” prestressed columns. Reduced column sizes are possible based on the interaction diagrams, however, maintaining the 24” x 24” size allows for easier connections to the T-RECS beams, simple placement of formwork alongside both the beam and column, and a greater moment capacity at the face of the support. Therefore, the size has been selected to match the dimensions of the precast beam.

4.2.4 – Precast L-Beam Design

Precast L-Beams at the perimeter of the building can be designed using load tables as well. The L-beams will be designed as simple span members subject to the same loads.
but with half the tributary width. This results in a uniform superimposed service load of 2,349 plf. The table shown below is taken from the PCI design handbook (2004) and for a 30’ clear span, calls for a 20LB24.

\[ f_c = 5000 \text{ psi} \]
\[ f_y = 270,000 \text{ psi} \]
\[ V_c = \text{in.-diameter} \]

**Normalweight Concrete**

<table>
<thead>
<tr>
<th>Designation</th>
<th>h</th>
<th>h_1</th>
<th>h_2</th>
<th>A</th>
<th>f</th>
<th>( \gamma_s )</th>
<th>S_1</th>
<th>S_2</th>
<th>W</th>
<th>lb/ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>20LB20</td>
<td>20</td>
<td>12</td>
<td></td>
<td>304</td>
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<td>8.74</td>
<td>1163</td>
<td>922</td>
<td>317</td>
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<tr>
<td>20LB24</td>
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<td></td>
<td>806</td>
<td>17</td>
<td>13.666</td>
<td>7.78</td>
<td>1023</td>
<td>800</td>
<td>286</td>
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<tr>
<td>20LB40</td>
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<td>1124</td>
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<td>11.192</td>
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<td>1767</td>
<td>566</td>
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<td>9.15</td>
<td>1954</td>
<td>1767</td>
<td>566</td>
</tr>
</tbody>
</table>

1. Check flexure at supports of beam.
2. Gaits below shown include 50% superimposed dead load and 50% live load.
3. 60 psi top tension has been obtained, therefore additional top reinforcement is required.
4. Slab flexure can be significantly improved by use of structural composite topping.

**Key**
- \( \sigma_{60} \): Safe superimposed service load, lb/ft
- \( \sigma_{61} \): Estimated camber at erection, in.
- \( \sigma_{62} \): Estimated long-time camber, in.

**Table of safe superimposed service load, lb/ft, and cambers, in.**

<table>
<thead>
<tr>
<th>Designation</th>
<th>Number</th>
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<th>20</th>
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<th>40</th>
<th>42</th>
<th>44</th>
<th>46</th>
<th>50</th>
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</thead>
<tbody>
<tr>
<td>20LB20</td>
<td>9</td>
<td>2.44</td>
<td>6500</td>
<td>5190</td>
<td>4100</td>
<td>3340</td>
<td>2700</td>
<td>2310</td>
<td>1960</td>
<td>1670</td>
<td>1430</td>
<td>1240</td>
<td>1070</td>
<td></td>
<td></td>
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<tr>
<td>20LB40</td>
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<td>2.80</td>
<td>6570</td>
<td>5490</td>
<td>4500</td>
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<td>1850</td>
<td>1510</td>
<td>1280</td>
<td>1050</td>
<td>909</td>
<td></td>
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<tr>
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<td>5490</td>
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<td>1510</td>
<td>1280</td>
<td>1050</td>
<td>909</td>
<td></td>
</tr>
</tbody>
</table>
4.3 – Lateral Analysis

A first-order linear elastic lateral analysis is conducted for the building. The intent of this analysis is to ensure that the moment frames are reasonably capable of achieving the required capacities from lateral loads, and not to perform a complete and detailed lateral analysis. Under this pretense, the second-order P-Δ effects and moment magnification procedures of an elastic second-order analysis have been omitted. There are five separate beam lines referenced at grids 1 through 5 in Figure 4.1. This allows for a possible total of five moment frames which could be used in the lateral load resisting system. Only the interior frames are used, however, for a total of three frames. This is done because the exterior beams are L-beams. Using these beams as part of a frame would require special detailing to create moment connections at the column supports and would add additional complexity to the system. In the orthogonal direction to the moment frames, the lateral load resisting system is a typical precast shear wall system.

The lateral analysis is conducted according to ASCE 7 (2005) which contains provisions for wind and seismic design of structures in Chapters 6 and 11 respectively. As stated in the design criteria, the building is assigned an Occupancy Category of II and is located in western Iowa. For the lateral analysis due to wind loading, the location dictates an average wind speed of 90 mph. The structure is 60’ tall and therefore conforms to the definition of a low-rise building, and it is also assumed to be fully enclosed. Additionally, it is assumed that the building is located in an urban or suburban area not
near hills, ridges, or escarpments and therefore the exposure category is B. The design wind pressure is calculated according to the provisions for the Main Wind-Force Resisting System (MWFRS) of a low rise building. A detailed calculation showing the wind pressure profile can be found in the Appendix. This wind pressure profile is applied to the surface of the building and then the story forces are calculated. Each story force is divided among the three moment frames and these forces are shown in Figure 4.30.

For a seismic analysis the weight assigned to each floor level, needs to be determined. In addition to the weights of the floors, beams, and column, a 6” precast wall panel is assumed to be the exterior envelope of the building. Summing the weight of all these components results in a weight of approximately 1,865 kips for a typical floor. This weight gets reduced to 1,717 kips at the top story due to the square footage of the precast wall panels being cut in half. A detailed breakdown of the component weights is shown in the Appendix.

ASCE 7 (2005) stipulates that when soil properties are unknown, the site class shall be assumed as D. The seismic design category based upon the short and one second period response acceleration parameters is category B. Selecting the appropriate response modification coefficient ‘R’ is highly subjective for this system because the ductility of members and behavior of the connection during a seismic event is unknown. Experimental testing is likely required in order to assign an appropriate value. A conservative assumption is made here that the structure is an ‘ordinary reinforced
Figure 4.32– Service Level Story Forces for Each Moment Frame Due to Wind Loading
concrete moment frame’. This designation assigns and an R value of 3 and a system overstrength factor, \( \Omega_0 \), of 3. The overstrength factor is applied to the connections to ensure that the connection can hold up to the full force of the seismic event without failing. Total base shear for the design is 315 kips. Detailed calculations showing the analysis and the distribution of the base shear to each story is shown in the Appendix. Story forces distributed to each of the three moment frames are shown in Figure 4.31.

RISA 3-D software is used to perform the lateral analysis. Columns are modeled as 24”x24” concrete columns with 5000 psi concrete, and the T-RECS beams are modeled as a generic section with a moment of inertia and modulus of elasticity equal to that of the final fully composite cross section. Nodes are modeled as full moment connections. Vertical offsets have been added at each node in all directions to resemble the thicknesses of the column and beams. This will help produce more accurate deflection results. The load case producing the greatest moments from the lateral analysis is 1.2D + 1.0E + L. This loading condition is shown in Figure 4.32. Beam and column moments generated by these loads are shown in Figures 4.33 and 4.34 respectively. The greatest negative moment produced by the lateral loads for the beam design is 95.5 k-ft.
Figure 4.33—Service Level Story Forces for Each Moment Frame Due to Seismic Loading
Figure 4.34– Ultimate Factored Loads for Each Moment Frame Due to Seismic Loading
Figure 4.35 – Ultimate Factored Moment Diagrams for Beams
Figure 4.36– Ultimate Factored Moment Diagrams for Columns
4.4 – Deflections

Deflection limits for the design also need to be satisfied. There are two deflections limits that are checked in the design. The first is beam deflections due to live load and the second is the inter-story drift from lateral loads.

ACI 318 (2008) sets limits for live load deflections of floors not supporting or attached to nonstructural elements likely to be damaged by large deflections at L/360. Figure 4.35 shows the expected deflections at release of prestressing strand based on the PCI Design handbook (2004). Deflections due to loads during construction are shown in Figure 4.36. Deflections of the fully composite section due to SIDL and live loads are shown in Figure 4.37 and the final total deflection experienced by the beam is shown in Figure 4.38. All deflection values shown are for an interior span.

Figure 4.37 – Deflections at Release
Figure 4.38 – Deflections During Construction

Figure 4.39 – Deflections for Service Loads

Figure 4.40 – Total Deflections
A detailed breakdown of the deflections caused by each load can be found in the Appendix along with the deflection equation used to determine the deflection at each portion of the beam. The total deflection value is obtained by adding the previous three stages together. The primary value of concern is the live load deflections. These must not exceed 1". With live load deflections of only 0.1", the limit is easily satisfied.

Limits on inter-story drift are prescribed by the ASCE 7 (2005). The table in Chapter 12 calls for the inter-story drift to be limited to 0.020h_{sx} based on an occupancy category of II. h_{sx} is the story height below level ‘x’. This calls for a limit of 2.4" between stories. Deflections for inter-story drift are taken from the RISA model constructed for the lateral analysis, and the results are shown in Table 4.7. This shows that the inter-story drift is acceptable.

<table>
<thead>
<tr>
<th>Level</th>
<th>Displacement (in)</th>
<th>Relative Displacement (in)</th>
</tr>
</thead>
<tbody>
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<td>0</td>
<td>0</td>
</tr>
<tr>
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<td>0.079</td>
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<td>0.105</td>
</tr>
<tr>
<td>6</td>
<td>0.698</td>
<td>0.066</td>
</tr>
</tbody>
</table>
4.5 – Connection Design

This section details the calculations for ensuring adequate strength of connecting materials for the system. The connections designed in this section are the torsion restraints and the moment connection at the column support.

4.5.1 – Moment Embed Connection Design

Design Loads

Negative Moment at Column: \( M_u = 271 \text{ kip-ft} \)

Given:
\( a = 4.59 \text{ in} \); \( d = 14 \text{ in} \);
\( T_u = M_u/(d - a/2) = 277.83 \text{ kip} \);

Weld Design

Given:
\( \phi = 0.75 \); \( F_{exx} = 80 \text{ ksi} \);
\( \phi T_n = f \times 0.6 \times F_{exx} \times t_w \times l_w \);
\( \phi T_n = 297.95 \text{ kips} \);

*see Figure 4.23 for weld
Embed Plate design

Given:
\[ \phi = 0.9; \quad n = 12; \quad A_s = n \times 0.6 \text{ in}^2 = 7.2 \text{ in}^2; \quad t_{pl} = 0.75 \text{ in}; \quad b_{pl} = 16 \text{ in}; \]
\[ f_y = 60 \text{ ksi}; \quad z_{pl} = b_{pl} \times t_{pl}^2/4 = 2.25 \text{ in}^3; \quad s = 3 \text{ in}; \]

Rebar Tension Capacity;

\[ \phi T_n = \phi \times A_s \times f_y = 388.8 \text{ kip}; \]
*use 12 #7 bars

Plate Bending Capacity;

- Assume plate behaves as a fixed-fixed condition between the rebar

\[ \phi M_n = f \times z_{pl} \times f_y; \quad \geq \text{ Mu } = T_u \times s / 8; \]

Therefore;
\[ \phi T_n = 8 \times (f \times z_{pl} \times f_y) / s; \]
\[ \phi T_n = 324 \text{ kips}; \]
*use a 3/4” thick embed plate

4.5.2 – Torsion Connection Design

Design Loads

\[ T_u = 130 \text{ kip-in}; \quad d = 18 \text{ in}; \quad x = 10 \text{ in}; \quad W_{beam} = 0.183 \text{ klf} \times 30 \text{ ft} = 5.49 \text{ kip}; \]
\[ V_u = (T_u - W_{beam} \times x)/d = 4.17 \text{ kip}; \]
Loose angle bending

Given:

- \(t_{pl} = 0.75\) in; \(b_{pl} = 3\) in; \(k_{ang} = 1.1875\) in;
- \(f_y = 36\) ksi; \(z_{pl} = b_{pl} \times t_{pl}^2/4 = 0.42\) in³;

- Assume the angle behaves as a fixed cantilever

\[e_x = 3.5\) in - \(k_{ang} = 2.31\) in;
\[
\phi \ M_n = \phi \times z\times F_y; \ \geq \ M_u = V \times e_y;
\]

Therefore,
\[
\phi \ V_n = \frac{(\phi \times z_{pl} \times f_y)}{e_x} = 5.91\) kip
\]

*use a 3½ x 5 x ¾” x 0’-3” angle

Weld to Loose Angle

In-Plane Vertical Load

Given:

- \(\phi = 0.75; \ F_{exx} = 70\) ksi; \(a = 0.25\) in; \(t_w = a / \sqrt{2} = 0.18\) in;
- \(b = 3\) in; \(d = 2\) in;

Weld Capacity:

\[A_w = t_w \times (d) \times 2 = 0.71\) in²;

" \(\phi f_u = f\times(0.6\times F_{exx})";\n"f_u = T_u / A_w";
"f\times(0.6\times F_{exx}) > T_u / A_w";

\[\phi \ V_n = \phi \times 0.6 \times F_{exx} \times A_w\]
\[\phi \ V_n = 22.27\) kips;

*see Figure 4.24 for weld
Chapter 5 – Cost Estimate and Design Aids

5.1 – Design Aids

The following tables and charts have been developed based on the detailed methodology previously presented in this document. Load tables for four possible standardized T-RECS beams are presented in Figures 5.1 through 5.4. The hollow-core weight and capacities for generating these tables are taken from Concrete Industries, Inc., for 8”, 10”, and 12” hollow-core planks. These load tables are provided for reference below. In generating the tables, the span length of the hollow-core is constrained to the span length allowing for a maximum load of 100 psf. For example, an 8” hollow-core can achieve a span length of 30’ under a service load of 100 psf according to the load table below. The span length of the T-RECS beam is then allowed to vary.
10" HOLLOWCORE WITH 2" COMPOSITE TOPPING

Notes:
1. Designs assume 2" composite topping at mid-span. Topping thickness at ends will be larger than 2".
2. Spans and loads shown are typical. Longer spans and larger loads by special design. Contact Concrete Industries.
3. All superimposed load is considered live load with live load deflection < 0.300 in.
4. Estimated camber for span lengths in boxed region is 1/16" to 3/4" after topping placement. In other spans, 1/32" or less.

12" HOLLOWCORE WITH 2" COMPOSITE TOPPING

Notes:
1. Designs assume 2" composite topping at mid-span. Topping thickness at ends will be larger than 2".
2. Spans and loads shown are typical. Longer spans and larger loads by special design. Contact Concrete Industries.
3. All superimposed load is considered live load with live load deflection < 0.300 in.
4. Estimated camber for span lengths in boxed region is 1/16" to 3/4" after topping placement. In other spans, 1/32" or less.
The naming convention proposed incorporates the total depth of the beam, the width of the precast concrete, and the number of strands in the precast. For example, a beam with a height of 14”, a width of 24”, and 12 – 0.6”Ø strand is named 14TR24-S12. Using this naming convention, the four beams presented in this chapter are: 14TR42-S12, 18TR24-S14, 18TR28-S18, and 20TR32-S22. Composite section properties for each of the proposed standardized beams are presented in Table 5.1. Each of the Figures, 5.1 through 5.4, contain the cross section of the beam and a chart demonstrating the safe allowable superimposed service loads for a given span length in pounds per square foot. The load tables are intended to be approximate loads used for preliminary sizing of the beam. More detailed analysis should be conducted in each case to ensure adequate strength and stress design.

<table>
<thead>
<tr>
<th>Table 5.1 - T-RECS Section Properties</th>
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<tr>
<td></td>
</tr>
<tr>
<td>A (in²)</td>
</tr>
<tr>
<td>I (in⁴)</td>
</tr>
<tr>
<td>y₁ (in)</td>
</tr>
<tr>
<td>y_b (in)</td>
</tr>
<tr>
<td>S₁ (in³)</td>
</tr>
<tr>
<td>S_b (in³)</td>
</tr>
<tr>
<td>y_ps (in)</td>
</tr>
<tr>
<td>wt (plf)</td>
</tr>
<tr>
<td>Aps (in²)</td>
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</table>
### 8" HC x 30' span

<table>
<thead>
<tr>
<th>Span Length (ft)</th>
<th>25</th>
<th>26</th>
<th>27</th>
<th>28</th>
<th>29</th>
<th>30</th>
</tr>
</thead>
<tbody>
<tr>
<td>14TR24-S12 load (psf)</td>
<td>100</td>
<td>100</td>
<td>96</td>
<td>76</td>
<td>56</td>
<td>38</td>
</tr>
</tbody>
</table>

**Figure 5.1 – 14TR24-S12 Load Table**
10" HC x 35' span  14 - 0.6" strand

<table>
<thead>
<tr>
<th>Span length (ft)</th>
<th>29</th>
<th>30</th>
<th>31</th>
<th>32</th>
<th>33</th>
<th>34</th>
</tr>
</thead>
<tbody>
<tr>
<td>18TR24-S14 load (psf)</td>
<td>100</td>
<td>100</td>
<td>82</td>
<td>62</td>
<td>44</td>
<td>28</td>
</tr>
</tbody>
</table>

Figure 5.2 – 18TR24-S14 Load Table
18TR28-S18

12" HC x 39' span  18 - 0.6 " strand

<table>
<thead>
<tr>
<th>Span length (ft)</th>
<th>32</th>
<th>33</th>
<th>34</th>
<th>35</th>
<th>36</th>
</tr>
</thead>
<tbody>
<tr>
<td>18TR28-S18 load (psf)</td>
<td>100</td>
<td>100</td>
<td>91</td>
<td>66</td>
<td>44</td>
</tr>
</tbody>
</table>

Figure 5.3 – 18TR28-S18 Load Table
12" HC x 39' span  22 - 0.6 " strand

<table>
<thead>
<tr>
<th>Span length (ft)</th>
<th>34</th>
<th>36</th>
<th>37</th>
<th>38</th>
<th>39</th>
<th>40</th>
</tr>
</thead>
<tbody>
<tr>
<td>20TR32-S22 load (psf)</td>
<td>100</td>
<td>100</td>
<td>94</td>
<td>73</td>
<td>53</td>
<td>35</td>
</tr>
</tbody>
</table>

Figure 5.4 – 20TR32-S22 Load Table
5.2 – Cost Estimate

A cost estimate is presented in this section to provide an approximation of how the T-RECS beam’s cost compares to that of a steel-framed building. It is not the intent of this estimate to provide an exhaustive detailed cost breakdown of all components required for the construction of a building including labor, transportation, and time cost-savings. Rather, the intent is to establish that the proposed system is not significantly more expensive and therefore commercially unviable when compared to a structural steel system. The estimate will focus on the raw material costs to establish this approximation. Material costs are taken from the RSMeans Building Construction Cost Data (2012). The structure for the comparison uses WF beams for all structural framing elements and the floor system is an 8” hollow-core plank system. All loads and span distances are similar to those described in section 4.1. Tables 5.2 and 5.3 provide a breakdown of this cost estimate. Results from this analysis show an estimate material cost of $24/sq ft for a steel system and around $18/sq ft for the proposed system. This shows that it is reasonable to expect that the proposed system should not be significantly more expensive and therefore can be a commercially viable option for designers in the future.
<table>
<thead>
<tr>
<th>Description</th>
<th>units</th>
<th>Material</th>
<th>Qty</th>
<th>Total</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>30' Interior Beams</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W18 x 106</td>
<td>lf</td>
<td>146</td>
<td>360</td>
<td>$ 52,560</td>
<td></td>
</tr>
<tr>
<td><strong>Floor System</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8&quot; Hollow-core</td>
<td>s.f.</td>
<td>7.15</td>
<td>900</td>
<td>$ 6,435</td>
<td>30'x30' bays</td>
</tr>
<tr>
<td>2&quot; Topping</td>
<td>c.y</td>
<td>103</td>
<td>5.56</td>
<td>$ 573</td>
<td>30'x30' bays</td>
</tr>
<tr>
<td>Topping reinf.</td>
<td>lb</td>
<td>0.49</td>
<td>194</td>
<td>$ 95</td>
<td>10' long @ 1' O.C.; 0.668 #/ft</td>
</tr>
<tr>
<td>Material cost per sq. ft. of floor</td>
<td>$ 7.89</td>
<td>14400</td>
<td>$ 113,642</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Columns</strong></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W12 x 79</td>
<td>lf</td>
<td>109</td>
<td>1200</td>
<td>$ 130,800</td>
<td></td>
</tr>
<tr>
<td><strong>Perimeter Beams</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W12 x 72</td>
<td>lf</td>
<td>99</td>
<td>480</td>
<td>$ 47,520</td>
<td></td>
</tr>
<tr>
<td><strong>Structure Cost</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Material cost per floor</td>
<td></td>
<td></td>
<td></td>
<td>$ 344,522</td>
<td></td>
</tr>
<tr>
<td>Material cost per sq foot</td>
<td></td>
<td></td>
<td></td>
<td>$ 24</td>
<td></td>
</tr>
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</table>
### Table 5.3 – Cost Analysis of Proposed System

<table>
<thead>
<tr>
<th>Description</th>
<th>units</th>
<th>Mat. Cost</th>
<th>Qty</th>
<th>Total</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>30' T-RECS Beam</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Precut W18x76</td>
<td>lf</td>
<td>89.25</td>
<td>30</td>
<td>2,678</td>
<td>1/2 of W18x76 + 75% increase for cutting</td>
</tr>
<tr>
<td>#4 Stirrups</td>
<td>lb</td>
<td>0.49</td>
<td>64</td>
<td>31</td>
<td>3 lf per stirrup @ 2 stirrups per foot; 0.668 #/lf</td>
</tr>
<tr>
<td>0.6&quot; dia strand</td>
<td>lb</td>
<td>0.49</td>
<td>266.4</td>
<td>131.2</td>
<td>12 - 0.6&quot; strand @ 0.74 #/lf</td>
</tr>
<tr>
<td>Concrete 8 ksi NW</td>
<td>c.y.</td>
<td>202</td>
<td>1.07</td>
<td>216</td>
<td>1 ft² per ft</td>
</tr>
<tr>
<td>high early strength</td>
<td>%</td>
<td>10%</td>
<td>22</td>
<td>22</td>
<td>concrete additive; 10% concrete increase</td>
</tr>
<tr>
<td>Total cost per lf of beam</td>
<td>$</td>
<td>106.11</td>
<td>360</td>
<td>38,199</td>
<td>total qty per floor</td>
</tr>
<tr>
<td><strong>Floor System for 30’x30’ bays</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8” Hollow-core</td>
<td>s.f.</td>
<td>7.15</td>
<td>900</td>
<td>6,435</td>
<td>30’x30’ bays</td>
</tr>
<tr>
<td>2” Topping 4 ksi</td>
<td>c.y.</td>
<td>103</td>
<td>5.56</td>
<td>573</td>
<td>30’x30’ bays</td>
</tr>
<tr>
<td>#6 Continuity reinf.</td>
<td>lb</td>
<td>0.49</td>
<td>302</td>
<td>148</td>
<td>10’ long @ 1’ O.C.; 1.043 #/ft</td>
</tr>
<tr>
<td>Topping reinf.</td>
<td>lb</td>
<td>0.49</td>
<td>194</td>
<td>95</td>
<td>10’ long @ 1’ O.C.; 0.668 #/lf</td>
</tr>
<tr>
<td>Material cost per sq. ft of floor</td>
<td>$</td>
<td>8.06</td>
<td>14400</td>
<td>116,013</td>
<td>total qty per floor</td>
</tr>
<tr>
<td><strong>35’ Columns</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete 6 ksi NW</td>
<td>c.y.</td>
<td>124</td>
<td>5.19</td>
<td>644</td>
<td>24”x24”</td>
</tr>
<tr>
<td>#8 rebar</td>
<td>lb</td>
<td>0.49</td>
<td>748</td>
<td>367</td>
<td>35” long @ 8 per column; 2.67 #/ft</td>
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<tr>
<td>#6 Stirrups</td>
<td>lb</td>
<td>0.49</td>
<td>219</td>
<td>107</td>
<td>6 if per stirrup @ 1 stirrup per ft; 1.043 #/ft</td>
</tr>
<tr>
<td>Corbels</td>
<td>ea</td>
<td>50</td>
<td>6</td>
<td>300</td>
<td></td>
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<tr>
<td>Material Cost per if of column</td>
<td>$</td>
<td>40.50</td>
<td>1200</td>
<td>48,596</td>
<td>total qty per floor</td>
</tr>
<tr>
<td><strong>30’ Precast L-Beam</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12&quot;x36&quot;</td>
<td>ea</td>
<td>3,300</td>
<td>1</td>
<td>3,300</td>
<td></td>
</tr>
<tr>
<td>Material Cost per if of beam</td>
<td>$</td>
<td>110</td>
<td>480</td>
<td>52,800</td>
<td>total qty per floor</td>
</tr>
<tr>
<td><strong>Structure Cost</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Material cost per floor</td>
<td>$</td>
<td>255,608</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Material cost per sq foot</td>
<td>$</td>
<td>18</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Chapter 6 – Conclusions

This research has proposed, developed, and designed a complete structural system for residential and commercial buildings. This system achieves significant improvements over typical structural systems such as steel, precast concrete, and CIP concrete. It is able to blend together, in an economical manner, nearly all of the advantages of each individual system into one composite steel and prestressed concrete system. These advantages are:

- All exposed surfaces are concrete. This gives the system superior fire performance characteristics over steel.
- The proposed system uses all precast structural members, which allows for greater quality and erection speed over CIP methods.
- The lateral load resisting system consists of moment frames, thereby eliminating the need for shear walls in the direction of the frame. This is an advantage over standard precast concrete systems.
- The proposed system is able to achieve span-to-depth ratios of over 24, which is an improvement over standard precast concrete systems.
- The proposed system does not require the CIP topping for construction loads. This gives the system incredible flexibility in its construction and greatly reduces floor-to-floor construction durations.
• The proposed system requires minimal formwork and no shoring or temporary bracing during construction. These are significant advantages over CIP concrete and other composite structural systems available to designers today.

• The proposed system is able to reduce the amount of steel used in the structural beams by nearly 30%, which results in significant cost savings.

• The proposed system achieves significant reductions in beam weights when compared to precast concrete.

• The proposed system is able to effectively resist the typical loads experienced by a six-story commercial building.

<table>
<thead>
<tr>
<th></th>
<th>Precast</th>
<th>Proposed</th>
<th>Achieved</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span-to-depth ratio ($)</td>
<td>15</td>
<td>&gt; 20</td>
<td>24</td>
</tr>
<tr>
<td>Fire Resistance ($)</td>
<td>Good</td>
<td>Maintain</td>
<td>Maintained</td>
</tr>
<tr>
<td>Construction Speed ($)</td>
<td>Good</td>
<td>Maintain</td>
<td>Maintained</td>
</tr>
<tr>
<td>Production Quality</td>
<td>Good</td>
<td>Maintain</td>
<td>Maintained</td>
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<tr>
<td>Shoring ($)</td>
<td>No</td>
<td>Maintain</td>
<td>Maintained</td>
</tr>
<tr>
<td>Material Price ($)</td>
<td>Low</td>
<td>Maintain</td>
<td>Possible increase</td>
</tr>
<tr>
<td>Weight</td>
<td>Heavy</td>
<td>Reduce</td>
<td>Approx 50% or greater reduction</td>
</tr>
<tr>
<td>Floor Plan</td>
<td>Restricted</td>
<td>Improve</td>
<td>Moment frames allow for greater flexibility</td>
</tr>
</tbody>
</table>
Bibliography

American Concrete Institute. (2008). Building code requirements for structural concrete (ACI 318-08) and commentary. Farmington Hills, MI: American Concrete Institute.


Appendix

A – Section Properties

Modulus of Elasticity

<table>
<thead>
<tr>
<th>Material</th>
<th>Modulus of Elasticity (ksi)</th>
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<tbody>
<tr>
<td>$E_s$</td>
<td>29000</td>
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<tr>
<td>$E_{ci}$</td>
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<tr>
<td>$E_c$</td>
<td>5153.6</td>
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<tr>
<td>$E_{cip}$</td>
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At Release

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<th>Value</th>
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<td>6 in</td>
</tr>
<tr>
<td>$b$</td>
<td>24 in</td>
</tr>
<tr>
<td>$T_f$</td>
<td>0.68 in</td>
</tr>
<tr>
<td>$b_c$</td>
<td>11.035 in</td>
</tr>
<tr>
<td>$f_y$</td>
<td>50 ksi</td>
</tr>
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<td>$f_{cy}$</td>
<td>6.5 ksi</td>
</tr>
<tr>
<td>$h_i$</td>
<td>14 in</td>
</tr>
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<td>$n_i$</td>
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<table>
<thead>
<tr>
<th>Area</th>
<th>$\bar{y}$</th>
<th>$A\bar{y}$</th>
<th>$I'$</th>
<th>$Ad^2$</th>
<th>$I'+Ad^2$</th>
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<tbody>
<tr>
<td>Flange</td>
<td>46.84</td>
<td>13.66</td>
<td>639.9</td>
<td>1.8</td>
<td>3030.7</td>
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<td>Precast</td>
<td>144.00</td>
<td>3.00</td>
<td>432.0</td>
<td>432.0</td>
<td>985.9</td>
</tr>
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<td>$\Sigma$</td>
<td>190.84</td>
<td>5.62</td>
<td>$I_{cr}$</td>
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At Construction

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<td>20 in</td>
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<tr>
<td>$T_f$</td>
<td>0.68 in</td>
</tr>
<tr>
<td>$b_c$</td>
<td>11.035 in</td>
</tr>
<tr>
<td>$f_y$</td>
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At Service

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Appendix A – Section Properties

Trans. Dim.
B – Pre-Composite Shear Design

Note: Uses simplified PCI method which assumes $f_{se} > 40\% f_{pu}$

CONCRETE SHEAR STRENGTH

Beam Geometry

- Width of stem $b_w = 24$ in
- Beam height $h = 6$ in
- Depth to prestressing reinforcement $d = 4.8$ in ($\geq 0.8h$)

Material properties

- Compressive strength of concrete $f'_c = 8$ ksi
- Yield strength of reinforcement $f_y = 60$ ksi
- Concrete density modifier $\lambda = 1.0$
- Strand transfer length $l_t = 26$ in
- Area of prestressing strand $A_{ps} = 2.604 \text{ in}^2$
- Ultimate tensile strength of strand $f_{pu} = 270$ ksi

Design load data

- Factored vertical load at critical section $V_u = 41.3$ kips
- Factored moment at critical section $M_u = 116 \text{ kip in}$
- Section under investigation $x = 10$ in
  (critical section typically located at $h/2 = 3$ in unless not loaded on a ledge – refer ACI 318-08 §11.1.3)
- Strength reduction factor $\phi = 0.75$

Shear Strengths

Nominal shear strength $V_c = (0.6 \times \lambda \times \sqrt{f'_c \times 1 \text{ lbs/in}^2}) + (700 \text{ lbs/in}^2 \times b_w \times d) = 86.82$ kip

Maximum shear strength limit $V_{cw, max} = 5 \times \lambda \times \sqrt{f'_c \times 1 \text{ lbs/in}^2} \times b_w \times d = 51.52$ kip

Minimum shear strength limit $V_{cw, min} = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ lbs/in}^2} \times b_w \times d = 20.61$ kip

Support shear strength $V_{cw, sup} = 3.5 \times \lambda \times \sqrt{f'_c \times 1 \text{ lbs/in}^2} \times b_w \times d = 36.06$ kip
Transition zone strength at ‘x’

\[ V_{cx} = V_{cw \, sup} + (V_{cw \, max} - V_{cw \, sup}) \times \min(x, l_t) / l_t = 42.01 \text{ kip} \]

Controlling value

Nominal shear strength

\[ V_{nx} = 42.01 \text{ kip} \]

Ultimate shear strength

\[ \phi V_c = \phi \times V_{nx} = 31.51 \text{ kip} \]

Fail - Concrete shear strength \( \phi V_c / 2 < V_u - \text{Provide shear reinforcing detailed below} \)

User note: (if \( \phi V_c / 2 > V_u \) – stop here, do not continue to shear reinforcement design)

**SHEAR REINFORCEMENT DESIGN**

Nominal steel shear strength required

\[ V_s = (V_u - \phi V_c) / \phi = 13.06 \text{ kip} \]

Maximum allowable steel shear strength

\[ V_{smax} = 8 \times \lambda \times \sqrt{(f'c \times 1 \text{ lbs/in}^2) \times b_w \times d} = 82.43 \text{ kip} \]

**OK- \( V_s < V_{smax} - \text{Section is Adequate} \)**

**Stirrup Spacing**

Stirrup center to center spacing

\[ s = \min(0.75 \times h, 24 \text{ in}) = 4.5 \text{ in} \]

Reduced stirrup spacing (if applicable)

\[ s_{max} = 4.5 \text{ in} \]

*Spacing Provided*

Pass - Spacing is adequate

Shear reinforcement check

Area of steel required

\[ A_{\text{req}} = V_s \times s_{prov} / (f_y \times d) = 0.18 \text{ in}^2 \]

Minimum area of steel required

\[ A_{\min} = A_{ps} \times f_{pu} \times s_{prov} \]

\[ A_v = \max(A_{\text{req}}, A_{\min}) = 0.18 \text{ in}^2 \]

*Shear reinforcement (user input)*

Diameter of stirrup bars

\[ D_{\text{stir}} = 0.5 \text{ in} \]

Area of horizontal reinforcement provided

\[ A_{\nu \, prov} = 2 \times \pi \times D_{\text{stir}}^2 / 4 = 0.393 \text{ in}^2 \]

Steel shear strength

\[ \phi V_s = \phi \times A_{\nu \, prov} \times f_y \times d / s_{prov} = 21.21 \text{ kip} \]

Pass - Shear reinforcing is adequate

Design strength

\[ \phi V_h = \phi V_c + \phi V_s = 52.71 \text{ kip} \]

**Design Summary**

~Member has a cross section of 24.0" wide by 6.0" deep with 8000 psi concrete

~Location of section under investigation is at 0.83 ft from support

~Required reinforcement at section under investigation consists of 0.50 in dia vertical stirrups spaced at 4 in O.C.
Steel beam analysis & design (AISC360-05)
In accordance with AISC360 13th Edition published 2005 using the LRFD method
Tedds calculation version 3.0.03

Support conditions
Support A
Vertically restrained
Rotationally free
Support B
Vertically restrained
Rotationally free

Applied loading
Beam loads
CIP Beam - Dead full UDL 0.129 kips/ft
CIP Top - Dead full UDL 0.725 kips/ft
H/C - Dead full UDL 1.68 kips/ft
Precast - Dead full UDL 0.145 kips/ft
Dead self weight of beam × 0.5

Load combinations
Load combination 1
Support A
Dead × 1.00
Span 1
Dead × 1.00
Analysis results

Maximum moment; $M_{\text{max}} = 305.7 \text{ kips}_\text{ft}$; $M_{\text{min}} = 0$

Maximum reaction at support A; $R_{A_{\text{max}}} = 40.8 \text{ kips}$; $R_{A_{\text{min}}} = 40.8 \text{ kips}$

Unfactored dead load reaction at support A; $R_{A_{\text{Dead}}} = 40.8 \text{ kips}$

Maximum reaction at support B; $R_{B_{\text{max}}} = 40.8 \text{ kips}$; $R_{B_{\text{min}}} = 40.8 \text{ kips}$

Unfactored dead load reaction at support B; $R_{B_{\text{Dead}}} = 40.8 \text{ kips}$

Section details

Section type; $W 18\times76$ (AISC 13th Edn 2005)
ASTM steel designation; A992
Steel yield stress; $F_y = 50 \text{ ksi}$
Steel tensile stress; $F_u = 65 \text{ ksi}$
Modulus of elasticity; $E = 29000 \text{ ksi}$

Resistance factors

Resistance factor for tensile yielding; $\phi_{\text{ty}} = 0.90$
Resistance factor for compression; $\phi_{\text{c}} = 0.90$
Resistance factor for flexure; $\phi_{\text{b}} = 0.90$

Lateral bracing

Span 1 has lateral bracing at supports only

Classification of sections for local bending - Section B4

Classification of flanges in flexure - Table B4.1 (case 1)

Width to thickness ratio; $b_f / (2 \times t_f) = 8.09$
Limiting ratio for compact section; $\lambda_{\text{pff}} = 0.38 \times \sqrt{E / F_y} = 9.15$
Limiting ratio for non-compact section; $\lambda_{\text{nff}} = 1.0 \times \sqrt{E / F_y} = 24.08$; Compact

Classification of web in flexure - Table B4.1 (case 9)

Width to thickness ratio; $(d - 2 \times k) / t_w = 37.74$
Limiting ratio for compact section; $\lambda_{\text{wff}} = 3.76 \times \sqrt{E / F_y} = 90.55$
Limiting ratio for non-compact section; \( \lambda_{\text{rwf}} = 5.70 \times \sqrt{\frac{E}{F_y}} = 137.27 \); Compact Section is compact in flexure

Design of members for flexure in the major axis - Chapter F

Required flexural strength; \( M_r = \max(\text{abs}(M_{s1_{\text{max}}}), \text{abs}(M_{s1_{\text{min}}})) = 305.656 \text{ kips}_f \)

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1; \( M_{\text{nyld}} = M_p = F_y \times Z_x = 679.167 \text{ kips}_f \)

Lateral-torsional buckling - Section F2.2

Unbraced length; \( L_b = L_s1 = 360 \text{ in} \)

Limiting unbraced length for yielding - eq F2-5; \( L_p = 1.76 \times r_y \times \sqrt{\frac{E}{F_y}} = 110.629 \text{ in} \)

Distance between flange centroids; \( h_o = d - t_f = 17.52 \text{ in} \)

\( c = 1 \)

\( r_{ts} = \sqrt{\frac{(l_y \times C_w)}{S_x}} = 3.022 \text{ in} \)

Limiting unbraced length for inelastic LTB - eq F2-6

\( L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{(J \times c / (S_x \times h_o)) \times \sqrt{[1 + \sqrt{(1 + 6.76 \times (0.7 \times F_y \times S_x \times h_o / (E \times J \times c))^2}]}} = 325.144 \text{ in} \)

Cross-section mono-symmetry parameter; \( R_m = 1.000 \)

Moment at quarter point of segment; \( M_A = 229.242 \text{ kips}_f \)

Moment at center-line of segment; \( M_B = 305.656 \text{ kips}_f \)

Moment at three quarter point of segment; \( M_C = 229.242 \text{ kips}_f \)

Maximum moment in segment; \( M_{\text{abs}} = 305.656 \text{ kips}_f \)

Lateral torsional buckling modification factor - eq F1-1; \( C_b = \min(3, 12.5 \times M_{\text{abs}} \times R_m / [2.5 \times M_{\text{abs}} + 3 \times M_A + 4 \times M_B + 3 \times M_C]) = 1.136 \)

Critical flexural stress - eq F2-4; \( F_{cr} = C_b \times \pi^2 \times E / (L_b / r_{ts})^2 \times \sqrt{[1 + 0.078 \times J \times c / (S_x \times h_o \times (L_b / r_{ts})^2)]} = 34.189 \text{ ksi} \)

Nominal flexural strength for lateral torsional buckling - eq F2-2; \( M_{\text{ltb}} = F_{cr} \times S_x = 415.962 \text{ kips}_f \)

Nominal flexural strength; \( M_n = \min(M_{\text{nyld}}, M_{\text{ltb}}) = 415.962 \text{ kips}_f \)

Design flexural strength; \( M_d = \phi_b \times M_n = 374.366 \text{ kips}_f \)

PASS - Design flexural strength exceeds required flexural strength
**D – Composite Shear Design**

**Note:** Uses simplified PCI method which assumes $f_{se} > 40\% f_{pu}$

## CONCRETE SHEAR STRENGTH

**Beam Geometry**

- *Width of stem* $b_w = 16\,\text{in}$
- *Beam height* $h = 16\,\text{in}$
- *Depth to prestressing reinforcement* $d = 14\,\text{in}$ ("$\geq 0.8h$"

**Material properties**

- *Compressive strength of concrete* $f'_c = 4\,\text{ksi}$
- *Yield strength of reinforcement* $f_y = 60\,\text{ksi}$
- *Concrete density modifier* $\lambda = 1.0$
- *Strand transfer length* $l_t = 26\,\text{in}$
- *Area of prestressing strand* $A_{ps} = 2.604\,\text{in}^2$
- *Ultimate tensile strength of strand* $f_{pu} = 270\,\text{ksi}$

**Design load data**

- *Factored vertical load at critical section* $V_u = 96\,\text{kips}$
- *Factored moment at critical section* $M_u = 10\,\text{kip\cdotin}$
- *Section under investigation* $x = 10\,\text{in}$
- *Strength reduction factor* $\phi = 0.75$

**Shear Strengths**

- Nominal shear strength $V_c = (0.6\times\lambda\times\sqrt{(f'_c\times1\,\text{lbs/in}^2)}+(700\,\text{lbs/in}^2\times b_w\times d) = 165.3\,\text{kip}$
- Maximum shear strength limit $V_{cw\,\text{max}} = 5\times\lambda\times\sqrt{(f'_c\times1\,\text{lbs/in}^2)}\times b_w\times d = 70.84\,\text{kip}$
- Minimum shear strength limit $V_{cw\,\text{min}} = 2\times\lambda\times\sqrt{(f'_c\times1\,\text{lbs/in}^2)}\times b_w\times d = 28.33\,\text{kip}$
- Support shear strength $V_{cw\,\text{sup}} = 3.5\times\lambda\times\sqrt{(f'_c\times1\,\text{lbs/in}^2)}\times b_w\times d = 49.58\,\text{kip}$
Transition zone strength at 'x'  
\[ V_{cx} = V_{cw\, sup} + (V_{cw\, max} - V_{cw\, sup}) \times \min(x, l) / l = 57.76 \, \text{kip} \]

Controlling value
- Nominal shear strength \( V_{nx} = 57.76 \, \text{kip} \)
- Ultimate shear strength \( \phi V_c = 43.32 \, \text{kip} \)

**Fail - Concrete shear strength \( \phi V_c / 2 < V_u \) - Provide shear reinforcing detailed below**

User note: (if \( \phi V_c / 2 > V_u \) – stop here, do not continue to shear reinforcement design)

### SHEAR REINFORCEMENT DESIGN

Nominal steel shear strength required \( V_s = (V_u - \phi V_c) / \phi = 70.24 \, \text{kip} \)

Maximum allowable steel shear strength \( V_{smax} = 8 \times \lambda \times (f'_c \times 1 \, \text{lbf/in}^2) \times b_w \times d = 113.34 \, \text{kip} \)

**OK - \( V_s < V_{smax} \) - Section is Adequate**

**Shear reinforcement check**
- Area of steel required \( A_{vreq} = V_s \times s_{prov} / (f_y \times d) = 1 \, \text{in}^2 \)
- Minimum area of steel required \( A_{vmin} = A_{ps} \times f_{pu} / s_{prov} \)
- Controlling area of steel \( A_v = \max(A_{vreq}, A_{vmin}) = 1 \, \text{in}^2 \)

*Shear reinforcement (user input)*
- Area of horizontal reinforcement provided \( A_{v_{prov}} = 3 \, \text{in} \times 0.425 \, \text{in} = 1.27 \, \text{in}^2 \)
- Steel shear strength \( \phi V_s = \phi \times A_{v_{prov}} \times f_y \times d / s_{prov} = 66.94 \, \text{kip} \)

**Pass - Shear reinforcing is adequate**

### Design Summary
- Member has a cross section of 16.0” wide by 16.0” deep with 4000 psi concrete
- Location of section under investigation is at 0.83 ft from support
- Reinforcement at section under investigation consists an area of steel equal to 1.27 in² spaced at 12 in O.C.
### Flexural Strength

#### Concrete Layers

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<th>f'c</th>
<th>Q (kip-in)</th>
<th>T (kip-in)</th>
<th>R (kip-in)</th>
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<th>K</th>
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<td>100</td>
<td>1.096</td>
<td>0.0000</td>
<td>-0.0013</td>
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</tbody>
</table>

#### Design

- RC & P/C ACI
- \( e_n = 0.003 \)
- \( a = 4.094 \)
- \( \sum f_{\text{forces}} = 0.00 \)

### Answer:

- \( b \): 0.850
- \( \beta \): 0.850
- \( M_{\text{L}, \text{m}} \): 2357 kip-ft

---

**Notes:**
- Units in kips and inches
- Concrete Layers: \( f'c \), Width, \( W \), Thickness, \( T \)
- Steel Layers: Area \( A \), Grade, Effective Prest., Depth \( d \)
- Flexural Strength Calculations: \( E \), \( T \), \( R \), \( K \), Moment
- Design: RC & P/C ACI

---

**Diagram:**
- Flexural Strength diagram showing \( e_n \), \( a \), and \( \beta \) with respective calculations.

**Values:**
- \( e_n = 0.003 \)
- \( a = 4.094 \)
- \( \sum f_{\text{forces}} = 0.00 \)

**Final Answer:**
- \( Av. \beta = 0.850 \)
- \( M_{\text{L}, \text{m}} = 2357 \text{ kip-ft} \)
**G – ASCE7 – 05 – MWFRS Wind Profile Calculation**

**ASCE7-05**

Wind speed 90 mph figure 6-1 (ASCE 7-05)

Occupancy Cat. 2

Import. factor (I) 1 table 6-1 (ASCE 7-05)

$G_{pi} = 0.18$ figure 6-5 ASCE 7-05

$G = 0.85$ assumed

$C_{pw} = 0.8$ figure 6-6 ASCE 7-05

$C_{pl} = -0.5$ figure 6-6 ASCE 7-05

exposure cat. B assumed

wall height 60 feet $q_z * K_z = 17.626$ (must still be multiplied by !)

L/B 1 $q_h = 15.158$

topo factor (Kzt) 1.0 assumed

enclosure type enclosed

direct. Factor (Kd) 0.85 table 6-4 (ASCE 7-05)

<table>
<thead>
<tr>
<th>Height (ft)</th>
<th>$K_z$ (table 6-3)</th>
<th>$q_z$</th>
<th>Case 1 (+GCpi)</th>
<th>Case 2 (-GCpi)</th>
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<td>0.62</td>
<td>10.928</td>
<td>4.70</td>
<td>-9.17</td>
</tr>
<tr>
<td>25</td>
<td>0.66</td>
<td>11.633</td>
<td>5.18</td>
<td>-9.17</td>
</tr>
<tr>
<td>30</td>
<td>0.7</td>
<td>12.338</td>
<td>5.66</td>
<td>-9.17</td>
</tr>
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<td>40</td>
<td>0.76</td>
<td>13.395</td>
<td>6.38</td>
<td>-9.17</td>
</tr>
<tr>
<td>50</td>
<td>0.81</td>
<td>14.277</td>
<td>6.98</td>
<td>-9.17</td>
</tr>
</tbody>
</table>

parapet

62 0.86 15.16 22.74 15.158 37.90

**Pressure Profile**

![Pressure Profile Graph](attachment:image-url)
### H – Structure Weight for Seismic Design

#### Weight for Seismic Design

Floor to floor height

<table>
<thead>
<tr>
<th>Weight (k) for a typical floor</th>
<th>units</th>
<th>qty</th>
<th>weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>(15)24x24 Columns</td>
<td>lf</td>
<td>150.0</td>
<td>87 kip</td>
</tr>
<tr>
<td>8&quot; Hollow-core</td>
<td>sq ft</td>
<td>14880</td>
<td>833 kip</td>
</tr>
<tr>
<td>Precast beams</td>
<td>lf</td>
<td>372</td>
<td>54 kip</td>
</tr>
<tr>
<td>Precast L-Beams</td>
<td>lf</td>
<td>248</td>
<td>96 kip</td>
</tr>
<tr>
<td>2&quot; CIP Topping</td>
<td>sf</td>
<td>14880</td>
<td>360 kip</td>
</tr>
<tr>
<td>CIP Beam in-fill</td>
<td>lf</td>
<td>372</td>
<td>48 kip</td>
</tr>
<tr>
<td>Steel beam</td>
<td>lf</td>
<td>372</td>
<td>14 kip</td>
</tr>
<tr>
<td>6&quot; P/c wall panels</td>
<td>sf</td>
<td>4480</td>
<td>325 kip</td>
</tr>
</tbody>
</table>

1,817 kips/floor

#### Weight (k) for half story

<table>
<thead>
<tr>
<th>Weight (k) for half story</th>
<th>units</th>
<th>qty</th>
<th>weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>(15)24x24 Columns</td>
<td>lf</td>
<td>150.0</td>
<td>87 kip</td>
</tr>
<tr>
<td>8&quot; Hollow-core</td>
<td>sq ft</td>
<td>14880</td>
<td>833 kip</td>
</tr>
<tr>
<td>Precast beams</td>
<td>lf</td>
<td>372</td>
<td>54 kip</td>
</tr>
<tr>
<td>Precast L-Beams</td>
<td>lf</td>
<td>248</td>
<td>96 kip</td>
</tr>
<tr>
<td>2&quot; CIP Topping</td>
<td>sf</td>
<td>14880</td>
<td>360 kip</td>
</tr>
<tr>
<td>CIP Beam in-fill</td>
<td>lf</td>
<td>372</td>
<td>48 kip</td>
</tr>
<tr>
<td>Steel beam</td>
<td>lf</td>
<td>372</td>
<td>14 kip</td>
</tr>
<tr>
<td>6&quot; P/c wall panels</td>
<td>sf</td>
<td>2440</td>
<td>177 kip</td>
</tr>
</tbody>
</table>

1,669 kips/floor
I – ASCE7-05 – Seismic Load Story Shear Calculation

Tedds calculation version 3.0.01

Site parameters

Site class; D
Mapped acceleration parameters (Section 11.4.1)
at short period; \( S_S = 0.075 \)
at 1 sec period; \( S_1 = 0.043 \)
Site coefficient at short period (Table 11.4-1); \( F_a = 1.6 \)
at 1 sec period (Table 11.4-2); \( F_v = 2.4 \)

Spectral response acceleration parameters

at short period (Eq. 11.4-1); \( S_{MS} = F_a \times S_S = 0.120 \)
at 1 sec period (Eq. 11.4-2); \( S_{M1} = F_v \times S_1 = 0.103 \)

Design spectral acceleration parameters (Sect 11.4.4)
at short period (Eq. 11.4-3); \( S_{DS} = \frac{2}{3} \times S_{MS} = 0.080 \)
at 1 sec period (Eq. 11.4-4); \( S_{D1} = \frac{2}{3} \times S_{M1} = 0.069 \)

Seismic design category

Occupancy category (Table 1-1); II
Seismic design category based on short period response acceleration (Table 11.6-1) A
Seismic design category based on 1 sec period response acceleration (Table 11.6-2) B
Seismic design category; B

Approximate fundamental period

Height above base to highest level of building; \( h_n = 60 \text{ ft} \)

From Table 12.8-2:
Structure type; Reinforce concrete moment frame
Building period parameter \( C_t \); \( C_t = 0.016 \)
Building period parameter \( x \); \( x = 0.90 \)

Approximate fundamental period (Eq 12.8-7); \( T_a = C_t \times (h_n)^x \)
\( \times 1 \text{ sec} / (1 \text{ ft})^x = 0.637 \text{ sec} \)
Building fundamental period (Sect 12.8.2); \( T = T_a = 0.637 \text{ sec} \)
Long-period transition period; \( T_L = 12 \text{ sec} \)

Seismic response coefficient

Seismic force-resisting system (Table 12.14-1);

\( C_{\text{MOMENT_RESISTING_FRAME_SYSTEM}} \)

S
7. Ordinary reinforced concrete moment frames

Response modification factor (Table 12.14-1); \( R = 3 \)
Seismic importance factor (Table 11.5-2); \( I_e = 1.000 \)
Seismic response coefficient (Sect 12.8.1.1)
Calculated (Eq 12.8-2); \( C_{s,\text{calc}} = S_{DS} / (R \times I_e) = 0.027 \)
Maximum (Eq 12.8-3); \( C_{s,\text{max}} = S_{D1} / (T \times (R / I_e)) = 0.036 \)
Minimum (Eq 12.8-5, Supp. No. 2); \( C_{s_{\text{min}}} = \max(0.044 \times S_{DS} \times I_{e}, 0.01) = 0.010 \)
Seismic response coefficient; \( C_{s} = 0.027 \)

**Seismic base shear (Sect 12.8.1)**
- Effective seismic weight of the structure; \( W = 11826.0 \text{ kips} \)
- Seismic response coefficient; \( C_{s} = 0.027 \)
- Seismic base shear (Eq 12.8-1); \( V = C_{s} \times W = 315.4 \text{ kips} \)

**Vertical distribution of seismic forces (Sect 12.8.3)**
- Vertical distribution factor (Eq 12.8-12); \( C_{vx} = \frac{w_{x} \times h_{x}}{\Sigma (w_{i} \times h_{i})} \)
- Lateral force induced at level \( i \) (Eq 12.8-11); \( F_{x} = C_{vx} \times V \)

**Vertical force distribution table**

<table>
<thead>
<tr>
<th>Level</th>
<th>Height from base to Level ( i ) (ft), ( h_{x} )</th>
<th>Portion of effective seismic weight assigned to Level ( i ) (kips), ( w_{x} )</th>
<th>Distribution exponent related to building period, ( k )</th>
<th>Vertical distribution factor, ( C_{vx} )</th>
<th>Lateral force induced at Level ( i ) (kips), ( F_{x} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0; 1000.0; 1.07; 0.000; 0.0;</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>10.0; 1829.0; 1.07; 0.044; 14.0;</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>20.0; 1829.0; 1.07; 0.093; 29.3;</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>30.0; 1829.0; 1.07; 0.143; 45.2;</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>40.0; 1829.0; 1.07; 0.195; 61.5;</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>6</td>
<td>50.0; 1829.0; 1.07; 0.248; 78.1;</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>7</td>
<td>60.0; 1681.0; 1.07; 0.277; 87.2;</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>
### At Release

<table>
<thead>
<tr>
<th>Location (in)</th>
<th>Deflection</th>
<th>0</th>
<th>36</th>
<th>72</th>
<th>108</th>
<th>144</th>
<th>180</th>
<th>216</th>
<th>252</th>
<th>288</th>
<th>324</th>
<th>360</th>
</tr>
</thead>
<tbody>
<tr>
<td>P/S End Mom 1</td>
<td>( M/(6EI)(3x^2-x^2-2lx) )</td>
<td>0.0000</td>
<td>0.2899</td>
<td>0.4883</td>
<td>0.6053</td>
<td>0.6511</td>
<td>0.6358</td>
<td>0.5697</td>
<td>0.4629</td>
<td>0.3526</td>
<td>0.1679</td>
<td>0.0000</td>
</tr>
<tr>
<td>P/S End Mom 2</td>
<td>( M/(6EI)(3x^2-x^2-2lx) )</td>
<td>0.0000</td>
<td>0.1679</td>
<td>0.3256</td>
<td>0.4629</td>
<td>0.5697</td>
<td>0.6358</td>
<td>0.6511</td>
<td>0.6053</td>
<td>0.4838</td>
<td>0.2899</td>
<td>0.0000</td>
</tr>
<tr>
<td>Beam weight</td>
<td>( 5wl^3/384EI )</td>
<td>0.0000</td>
<td>-0.0015</td>
<td>-0.0029</td>
<td>-0.0039</td>
<td>-0.0046</td>
<td>-0.0048</td>
<td>-0.0046</td>
<td>-0.0039</td>
<td>-0.0029</td>
<td>-0.0015</td>
<td>0.0000</td>
</tr>
<tr>
<td>Net at release</td>
<td></td>
<td>0.0000</td>
<td>0.456</td>
<td>0.811</td>
<td>1.064</td>
<td>1.216</td>
<td>1.267</td>
<td>1.216</td>
<td>1.064</td>
<td>0.811</td>
<td>0.456</td>
<td>0.0000</td>
</tr>
</tbody>
</table>

### At Construction

<table>
<thead>
<tr>
<th>Location (in)</th>
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<th>0</th>
<th>36</th>
<th>72</th>
<th>108</th>
<th>144</th>
<th>180</th>
<th>216</th>
<th>252</th>
<th>288</th>
<th>324</th>
<th>360</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hollow core</td>
<td>( \omega x/(24EI)(l^3-2lx^2+x^3) )</td>
<td>0.0000</td>
<td>-0.4764</td>
<td>-0.9013</td>
<td>-1.2339</td>
<td>-1.4452</td>
<td>-1.5175</td>
<td>-1.4452</td>
<td>-1.2339</td>
<td>-0.9013</td>
<td>-0.4764</td>
<td>0.0000</td>
</tr>
<tr>
<td>CIP beam</td>
<td>( \omega x/(24EI)(l^3-2lx^2+x^3) )</td>
<td>0.0000</td>
<td>-0.0365</td>
<td>-0.0691</td>
<td>-0.0947</td>
<td>-0.1164</td>
<td>-0.1109</td>
<td>-0.1109</td>
<td>-0.0947</td>
<td>-0.0691</td>
<td>-0.0365</td>
<td>0.0000</td>
</tr>
<tr>
<td>CIP topping</td>
<td>( \omega x/(24EI)(l^3-2lx^2+x^3) )</td>
<td>0.0000</td>
<td>-0.2056</td>
<td>-0.3889</td>
<td>-0.5325</td>
<td>-0.6237</td>
<td>-0.6549</td>
<td>-0.6237</td>
<td>-0.5325</td>
<td>-0.3889</td>
<td>-0.2056</td>
<td>0.0000</td>
</tr>
<tr>
<td>Net during construction</td>
<td></td>
<td>0.0000</td>
<td>-0.719</td>
<td>-1.359</td>
<td>-1.861</td>
<td>-2.180</td>
<td>-2.289</td>
<td>-2.180</td>
<td>-1.861</td>
<td>-1.359</td>
<td>-0.719</td>
<td>0.0000</td>
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</table>

### At Service

<table>
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<tr>
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<th>Deflection</th>
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<th>36</th>
<th>72</th>
<th>108</th>
<th>144</th>
<th>180</th>
<th>216</th>
<th>252</th>
<th>288</th>
<th>324</th>
<th>360</th>
</tr>
</thead>
<tbody>
<tr>
<td>SIDL End Mom 1</td>
<td>( \omega x/(24EI)(l^3-2lx^2+x^3) )</td>
<td>0.0000</td>
<td>-0.0539</td>
<td>-0.1020</td>
<td>-0.1396</td>
<td>-0.1635</td>
<td>-0.1717</td>
<td>-0.1635</td>
<td>-0.1396</td>
<td>-0.1020</td>
<td>-0.0539</td>
<td>0.0000</td>
</tr>
<tr>
<td>SIDL End Mom 2</td>
<td>( M/(6EI)(3x^2-x^2-2lx) )</td>
<td>0.0000</td>
<td>0.0376</td>
<td>0.0633</td>
<td>0.0785</td>
<td>0.0844</td>
<td>0.0824</td>
<td>0.0739</td>
<td>0.0600</td>
<td>0.0422</td>
<td>0.0218</td>
<td>0.0000</td>
</tr>
<tr>
<td>Total SIDL</td>
<td></td>
<td>0.0000</td>
<td>0.0011</td>
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<td>-0.0132</td>
<td>-0.0200</td>
<td>-0.0234</td>
<td>-0.0222</td>
<td>-0.0168</td>
<td>-0.0091</td>
<td>-0.0021</td>
<td>0.0000</td>
</tr>
<tr>
<td>LL End Mom 1</td>
<td>( M/(6EI)(3x^2-x^2-2lx) )</td>
<td>0.0000</td>
<td>0.1253</td>
<td>0.2110</td>
<td>0.2616</td>
<td>0.2814</td>
<td>0.2748</td>
<td>0.2462</td>
<td>0.2000</td>
<td>0.1407</td>
<td>0.0725</td>
<td>0.0000</td>
</tr>
<tr>
<td>LL End Mom 2</td>
<td>( M/(6EI)(3x^2-x^2-2lx) )</td>
<td>0.0000</td>
<td>0.0580</td>
<td>0.1125</td>
<td>0.1600</td>
<td>0.1970</td>
<td>0.2198</td>
<td>0.2251</td>
<td>0.2093</td>
<td>0.1688</td>
<td>0.1002</td>
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<td>-0.0439</td>
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<td>-0.0562</td>
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<td>0.0000</td>
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<tr>
<td>Net during service</td>
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<td>0.0065</td>
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<td>-0.101</td>
<td>-0.096</td>
<td>-0.073</td>
<td>-0.040</td>
<td>-0.009</td>
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</tr>
</tbody>
</table>

### Total Deflections

<table>
<thead>
<tr>
<th>Location (in)</th>
<th>Deflection</th>
<th>0</th>
<th>36</th>
<th>72</th>
<th>108</th>
<th>144</th>
<th>180</th>
<th>216</th>
<th>252</th>
<th>288</th>
<th>324</th>
<th>360</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Deflection</td>
<td></td>
<td>0.0000</td>
<td>-0.2575</td>
<td>-0.5697</td>
<td>-0.8538</td>
<td>-1.0504</td>
<td>-1.1232</td>
<td>-1.0595</td>
<td>-0.8698</td>
<td>-0.5880</td>
<td>-0.2712</td>
<td>0.0000</td>
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</tbody>
</table>