

Design of Shallow-Depth Floor Systems with Asymmetric Steel I-Beams

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ABSTRACT

Shallow-depth composite floor systems aim to improve building economy, speed, and efficiency. Currently, an array of proprietary systems is available that comprises asymmetric steel sections (I-beam or trapezoidal box) with a relatively wide bottom flange. This wide bottom flange facilitates the placement of decking or precast/prefabricated panels. These systems are fast to assemble compared to cast-in-place flat slab construction. However, extensive fabrication is typically required for the current steel sections. As a result, this paper presents a design methodology for shallow-depth systems utilizing conventional asymmetric steel I-beams (AIBs) that can be hot-rolled or built-up. The design methodology covers the limit states critical for construction in the noncomposite state and in-service as a composite system. The composite design presented is a lower-bound approach with no mechanical connectors. The horizontal shear transfer is through steel-concrete bond and friction, resulting in a partial composite design. This design methodology was developed through an experimental testing program on shallow-depth floor systems. The testing reduced the uncertainty of several design parameters and validated certain assumptions. Three main phases of testing were (1) an initial large-scale concept test, (2) nine composite beam tests, and (3) a full-scale system validation test. The critical findings of each phase are provided. Finally, design aids are presented to equip future engineers with resources to efficiently design shallow-depth floor systems.

Keywords: structural steel, composite, asymmetric, shallow, and floor system.

INTRODUCTION

The type of floor system selected for multistory residential buildings can be driven by the depth (to optimize floor-to-floor height) and speed of construction, both of which affect cost. Flat slab floor systems are pervasive and comprise reinforced concrete or post-tensioned concrete. These systems yield a relatively shallow depth. However, they require extensive forming, shoring, and effort for reinforcement placement, making them relatively slow to construct. An alternative presented herein is shallow-depth steel-concrete composite floor systems, where stay-in-place formwork [either steel deep decking, precast hollow-core (HC) panels, or cross-laminated timber (CLT) panels] is installed on the bottom flange of steel beams (shown in Figure 1). To facilitate easy placement of decking or panels, asymmetric I-beams (AIBs), comprised of a narrow top flange, are utilized. Following formwork placement, a concrete slab is cast-in-place, and the concrete encases the top

flange and web of the steel beams. These steel AIBs can either be built-up sections or hot-rolled shapes.

Various proprietary built-up asymmetric steel beams exist in the marketplace, such as the Slim Floor system by ArcelorMittal (ArcelorMittal, 2017), the Girder-Slab floor system (Girder-Slab Technologies, 2016), and the DELTA-BEAM system (DELTABEAM Technical Manual, 2019). The beams used in these systems are different than AIBs because they require substantial fabrication with web openings intended for rebar to be placed perpendicular to the beam.

An alternative to the built-up sections just mentioned are hot-rolled asymmetric I-shapes (ASBs) produced by British Steel and Corus Construction (now part of TATA Steel). The ASBs have been rolled in Europe and used in a floor system called Slimdek® (Lawson and Brekelmans, 1999; Lawson et al., 1997, 1998; Rackham et al., 2006). Historically, these types of shapes were not rolled in the United States. However, several U.S. steel mills are considering hot-rolled AIBs, termed A-shapes. Specific A-shape cross sections were recently developed as part of an American Institute of Steel Construction (AISC) study (Yarnold et al., 2025), which were sized for multistory residential (and light commercial) shallow-depth composite floor systems. The motivation was to increase the speed and efficiency of floor systems. Table 1 provides the dimensions for each shape. Note that if A-shapes are unavailable, the dimensions shown are recommended for efficient built-up sections.

The overarching objective of this paper is to provide engineers with the design methodology for shallow-depth

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Table 1. Hot-Rolled A-Shape Recommended Dimensions (Yarnold et al., 2025)

Shape	Area	Depth	Web	Top Flange		Bottom Flange		Nominal Weight*
			Thickness	Width	Thickness	Width	Thickness	
	in. ²	in.	in.	in.	in.	in.	in.	lb/ft
A12×108	31.8	12.0	$\frac{9}{16}$	11.0	$1\frac{1}{8}$	16.0	$1\frac{3}{16}$	108
A12×85	25.0	12.0	$\frac{9}{16}$	8.0	$1\frac{1}{8}$	13.0	$\frac{3}{4}$	85
A10×88	25.9	10.0	$\frac{9}{16}$	9.0	$1\frac{1}{8}$	14.0	$\frac{3}{4}$	88
A10×65	19.1	10.0	$\frac{1}{2}$	7.0	1	12.0	$\frac{5}{8}$	65
A8×72	21.1	8.0	$\frac{5}{8}$	6.5	$1\frac{1}{4}$	11.5	$\frac{3}{4}$	72
A8×52	15.3	8.0	$\frac{1}{2}$	6.5	$\frac{7}{8}$	11.5	$\frac{1}{2}$	52
A6×43	12.3	6.0	$\frac{1}{2}$	5.0	1	10.0	$\frac{1}{2}$	43

* Includes additional weight for the fillets.

composite floor systems utilizing AIBs within a steel frame building. This design methodology was developed through an extensive experimental testing program. A summary of the testing program is also provided herein to support several assumptions. In addition, design aids are provided to further support the future design of shallow-depth floor systems.

DESIGN METHODOLOGY

A comprehensive design methodology was developed for shallow-depth floor systems utilizing AIBs (hot-rolled or built-up). The methodology first addresses the calculations during construction in the noncomposite state. The shallow-depth system may utilize panels (HC or CLT) or deep decking to support the concrete during construction. Next, the methodology addresses the calculations of the completed floor system in the composite state. The method presented is a lower-bound approach with no mechanical connectors for horizontal shear transfer (only bond and friction). Therefore, the calculations are for a partial composite design. If additional mechanisms are provided, this will only increase the performance. However, the speed of fabrication or assembly may be reduced.

Noncomposite Design—Construction

Two primary construction scenarios are analyzed for the noncomposite AIB: (a) concentric loading from the slab pour on adjacent bays and (b) eccentric loading from the slab pour of a single bay (inducing torsion). Figure 2 graphically illustrates these scenarios. This section examines the analysis required for the design of AIBs during construction.

Concentric Loading

The main concentric loading scenario is immediately after the concrete slab is placed over the entire tributary area of the AIB [Figure 2(a)]. The noncomposite section supports the entire dead load (if unshored) along with a construction live load, typically 20 psf (ASCE, 2014). The load combination for this analysis includes a load factor of 1.4 for variable construction dead load and a construction live load factor of 1.6 (ASCE, 2014).

According to the AISC *Specification for Structural Steel Buildings*, hereafter referred to as the AISC *Specification*, four flexural limit states and a shear limit state should be evaluated for this noncomposite concentric loading (AISC, 2022). The flexural limit states (for an asymmetric I-beam) are covered in the AISC *Specification* Section F4, which

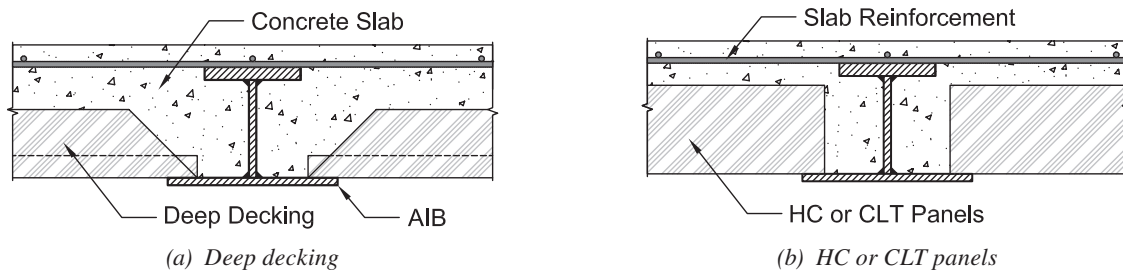


Fig. 1. Shallow-depth composite floor systems with AIBs.

includes compression flange yielding (CFY), tension flange yielding (TFY), compression flange local buckling (FLB), and lateral-torsional buckling (LTB). Treatment of CFY and TFY is relatively straightforward (refer to the AISC *Specification*). FLB is also relatively easy to apply; however, it is unlikely to control since AIB compression flanges are typically stocky and their width is narrow to facilitate panel or deep deck placement.

LTB is a critical limit state for AIBs in shallow-depth floor systems because the top flange is unbraced for the entire span before concrete curing. The AISC *Specification* LTB provisions can be strictly followed for AIBs, but this would be highly conservative. The authors recommend modification of these provisions to account for the load height effect.

The AISC *Specification* LTB provisions are derived for a load application at the beam centroid. For many situations, the applied load is at the top of the beam; however, the beam is usually braced with metal decking in typical composite floor systems. If the load is applied to the top flange with significant unbraced lengths, as the beam begins to buckle out-of-plane, a secondary moment is developed due to the rotation of the beam, resulting in an additional torsional moment in the direction of buckling [illustrated in Figure 3(a)]. However, if the load is applied to the bottom flange, a restoring torsional moment is formed in the opposite direction, which is shown in Figure 3(b).

To account for this load height effect, an adjustment is made to the LTB modification factor, C_b (AISC, 2022), through the load height modification factor, C_b^* (Helwig et al., 1997). This factor is a function of the warping stiffness of the cross section, W_s , and the type of loading. The warping stiffness parameter is evaluated in Equation 1 using the weak-axis moment of inertia, I_y ; the distance between flange centroids, h_o ; the shear modulus, G ; the modulus of elasticity of steel, E_s ; and the torsional constant of the section, J . Note that the warping stiffness, W_s , calculation in Equation 1 is for doubly symmetric I-shapes. This decision was made due to the recommendation by Helwig et al. (1997).

The load height modification factor, C_b^* , with load applied on the bottom flange (resultant centered) for a uniformly distributed load is expressed in Equation 2. C_b^* increases C_b from 10 to 35% (depending on the geometry), which directly increases the LTB strength.

$$W_s = \frac{\pi}{L_b} \sqrt{\frac{E_s I_y h_o^2}{4GJ}} \quad (1)$$

$$C_b^* = C_b (1 - 0.154W_s^2 + 0.535W_s) \quad (2)$$

The shear limit state is addressed in AISC *Specification* Section G2. The nominal shear strength considers the shear area of the web, the yield stress of the steel, and the web shear strength coefficient. The vertical shear strength

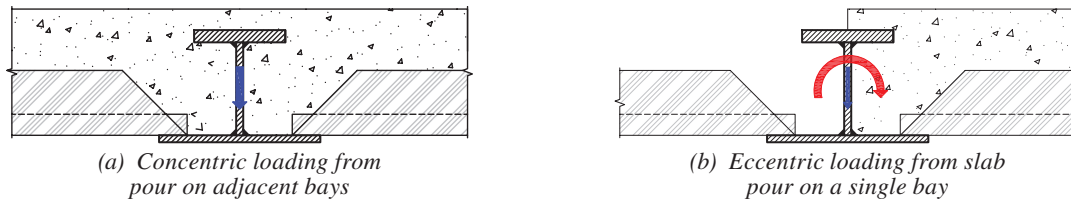


Fig. 2. Primary construction scenarios.

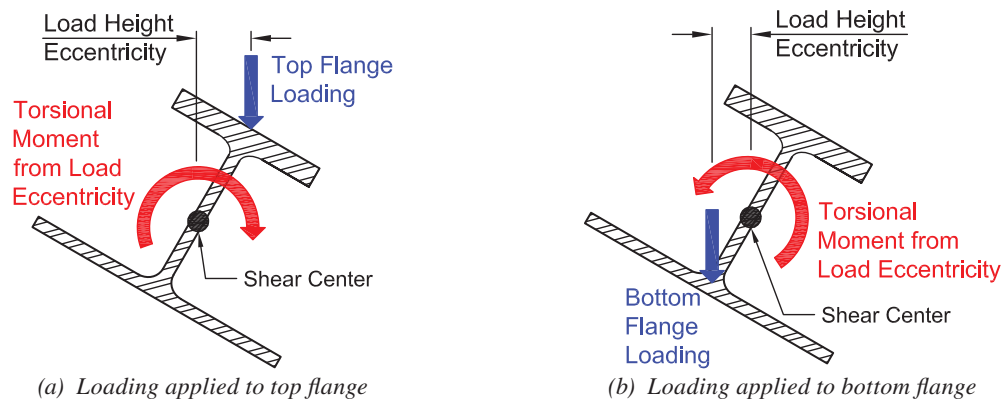


Fig. 3. Load height effect.

of AIBs is typically not a concern for A-shapes due to the web thicknesses. For built-up AIBs, the web thickness must be selected to satisfy AISC *Specification* Section G2 provisions.

Eccentric Loading

The primary eccentric loading scenarios (for unshored construction) are when panels (HC or CLT) are placed entirely on one side of the AIB or when the concrete slab is poured over a single bay [Figure 2(b)]. These scenarios induce normal stresses and shear stresses that must meet the limits provided in AISC *Specification* Section H3.3. These provisions include stress limits for yielding, along with buckling. The challenging portion of this analysis is the calculation of the required stresses. These stress calculations are summarized below. In addition, the angle of twist of the section should be considered. Guidance on this is also provided.

Note that bottom flange local bending should not control for the A-shapes provided in Table 1. Therefore, this calculation is not provided below. However, this should be checked for excessively large construction loading or sections with a reduced bottom flange thickness.

Normal Stress Yielding and Buckling

For the normal (or longitudinal) stress calculations, it is recommended to analyze the in-plane bending component separately from the torsional moment and then obtain the combined effect using superposition. Figure 4 illustrates the flange stress profiles due to flexure and torsion.

Calculation of the elastic in-plane bending stresses, σ_b , is relatively straightforward using Equation 3, which only includes the required bending moment, M_r , and the elastic section modulus, S_x .

$$\sigma_b = \frac{M_r}{S_x} \quad (3)$$

When analyzing a beam under torsion, it is essential to evaluate the two torsional moments resisted by the cross section. The first, given in Equation 4, is the uniform (or pure) torsion, also known as St. Venant torsion. The relationship between the first derivative of the angle of twist, θ' , and the torsional moment, T_t , is defined using the shear modulus of elasticity, G , and the torsional constant, J .

$$T_t = GJ\theta' \quad (4)$$

For noncircular cross sections such as AIBs, longitudinal deformation due to torsion (termed warping) is restrained from cross-flange bending. A torsional moment due to restrained warping of the section, T_w , is present. The extent of torsion caused by warping resistance is mathematically defined in Equation 5, which is the product of the modulus of elasticity, E_s ; warping constant, C_w ; and the third derivative of the angle of twist, θ''' . The warping constant for singly symmetric I-beams is provided in Equation 6, which is comprised of the distance between flange centroids, h_o , and the moment of inertia of the top flange about the y-axis, I_{yt} , and the moment of inertia of the bottom flange about the y-axis, I_{yb} .

$$T_w = -E_s C_w \theta''' \quad (5)$$

$$C_w = h_o^2 \frac{I_{yt} I_{yb}}{I_{yt} + I_{yb}} \quad (6)$$

The combined torsional resistance of the section, T_r , is provided in Equation 7. A detailed background of these expressions and section property definitions can be found in AISC Design Guide 9, *Torsional Analysis of Structural Steel Members* (Seaburg and Carter, 2003).

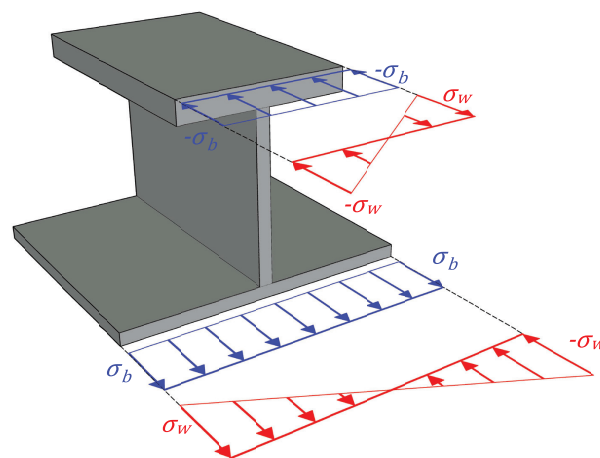


Fig. 4. Flange stresses due to bending and warping torsion.

$$T_r = GJ\theta' - E_s C_w \theta''' \quad (7)$$

Ultimately, the beam's angle of twist, θ , must be computed, along with the corresponding derivatives. The first step is to rearrange Equation 7 by dividing by the warping rigidity of the cross section, EC_w , resulting in Equation 8. Then, a parameter definition, a^2 , is introduced (shown in Equation 9). The resulting expression is provided in Equation 10, which is for a concentrated torsional moment, T_r .

$$\frac{T_r}{EC_w} = \frac{GJ\theta'}{E_s C_w} - \theta''' \quad (8)$$

$$a^2 = \frac{E_s C_w}{GJ} \quad (9)$$

$$\frac{T_r}{EC_w} = \frac{\theta'}{a^2} - \theta''' \quad (10)$$

Equation 10 can be revised to analyze a uniformly distributed torsional moment, t , as shown in Equation 11. The solution for this differential equation is conveyed in Equation 12 (Seaburg and Carter, 2003).

$$\frac{-t}{EC_w} = \frac{\theta''}{a^2} - \theta'''' \quad (11)$$

$$\theta = A + Bz + C \cosh \frac{z}{a} + D \sinh \frac{z}{a} - \frac{tz^2}{2GJ} \quad (12)$$

A simplification can be made by applying support conditions and solving for the constants, A , B , C , and D , where l is the beam length, and z is the location on the beam where θ is evaluated. For this analysis, θ and θ'' are needed for the calculation of the beam rotation and stresses, respectively. Equations 13 and 14 provide the resulting equations for torsionally pinned support conditions.

$$\theta = \frac{ta^2}{GJ} \left[\frac{l^2}{2a^2} \left(\frac{z}{l} - \frac{z^2}{l^2} \right) + \cosh \frac{z}{a} - \tanh \frac{l}{2a} \sinh \frac{z}{a} - 1.0 \right] \quad (13)$$

$$\theta'' = \frac{ta^2}{GJ} \left(\frac{1}{a^2} - \frac{\cosh \frac{z}{a}}{a^2} + \frac{\sinh \frac{z}{a} \tanh \frac{l}{2a}}{a^2} \right) \quad (14)$$

The longitudinal warping stresses, σ_w , are calculated using Equation 15, which requires the normalized warping constant, W_{no} . For an AIB, this value is different with respect to the top and bottom flanges. Equations 16 and 17 provide the normalized warping constant for the top flange, W_{not} , and bottom flange, W_{nob} (Heins, 1975). The required dimensional parameters shown in these equations include the location of the shear center from the centroid of the top flange, a (provided in Equation 18), the distance between the centroids of the flanges, h_o , the width of the top flange, b_{ft} , and the width of the bottom flange, b_{fb} .

$$\sigma_w = E_s W_{no} \theta'' \quad (15)$$

$$W_{not} = \frac{\alpha b_{ft}}{2} \quad (16)$$

$$W_{nob} = \frac{h_o - \alpha}{2} b_{fb} \quad (17)$$

$$\alpha = \frac{b_{fb}^3 t_{fb}}{b_{fb}^3 t_{fb} + b_{ft}^3 t_{ft}} h_o \quad (18)$$

As mentioned, stresses must be accurately combined to calculate the required normal stresses, f_r , at the critical points along the cross section (see Equation 19). These stresses must meet AISC *Specification* Section H3.3 limits.

$$f_r = \sigma_a \pm \sigma_b \pm \sigma_w \quad (19)$$

Shear Stress Yielding

Similar to the calculation of the normal stresses, shear stresses should be calculated using superposition. The shear stress due to in-plane loading, τ_b , can be calculated as shown in Equation 20, which is comprised of the applied shear demand, V , first moment of the area, Q , moment of inertia, I_x , and thickness of the element, t .

$$\tau_b = \frac{VQ}{I_x t} \quad (20)$$

The shear stresses due to pure torsion, τ_t , and warping torsion, τ_w , can be found with Equations 21 and 22, respectively. Another torsional property, S_w , is introduced in the warping equation, which is defined with respect to the top and bottom flanges in Equations 23 and 24, respectively. In these equations, the thickness of the top flange, t_{ft} , and the thickness of the bottom flange, t_{fb} , are needed. Also required for these equations are the first and third derivatives of the angle of twist (θ' and θ''' , respectively), which are provided in Equations 25 and 26. Note this is for pinned supports and uniformly distributed loading.

$$\tau_t = Gt\theta' \quad (21)$$

$$\tau_w = \frac{E_s S_w \theta'''}{t} \quad (22)$$

$$S_{wt} = \frac{W_{not} b_{ft} t_{ft}}{4} \quad (23)$$

$$S_{wb} = \frac{W_{nob} b_{fb} t_{fb}}{4} \quad (24)$$

$$\theta' = \frac{ta^2}{GJ} \left[\frac{l^2}{2a^2} \left(\frac{1}{l} - \frac{2z}{l^2} \right) + \frac{\sinh \frac{z}{a}}{a} - \frac{\cosh \frac{z}{a} \tanh \frac{l}{2a}}{a} \right] \quad (25)$$

$$\theta''' = \frac{ta^2}{GJ} \left(\frac{\sinh \frac{z}{a}}{a^3} - \frac{\cosh \frac{z}{a} \tanh \frac{l}{2a}}{a^3} \right) \quad (26)$$

To calculate the shear stresses, f_v , at different points along the cross section, stresses must be accurately combined (see Equation 27). These stresses must meet the AISC *Specification* Section H3.3 limits.

$$f_v = \tau_b \pm \tau_t \pm \tau_w \quad (27)$$

Angle of Twist

It is recommended to check the maximum angle of twist of the section, θ , calculated from Equation 13, which typically occurs at mid-span. This twist check is for constructability. Engineering judgment should be used for the allowable twist limit. This limit may be different for panelized systems versus deep decking. The development of the A-shape cross sections utilized a 5.0° limit. However, this is not a strict limit and is left to the discretion of the designer for their situation.

Composite Design—In-Service

The final composite system must meet strength and serviceability criteria. The required loading and load factors are provided in ASCE/SEI 7, *Minimum Design Load and Associated Criteria for Buildings and Other Structures* (2022). The required strength and serviceability structural checks are provided in the following sections. In addition, a discussion on fire resistance is included.

Strength Checks

The flexural strength of a partially concrete-encased AIB (compression flange and web), without mechanical connectors, is based on a partial composite design that includes two limit state checks. The first is the composite strength based on the bond shear strength between the steel-concrete interface. The second is the longitudinal shear strength of the concrete (only applicable for panel systems without

chamfers). These limit states and the corresponding calculations are provided later in this subsection.

The full composite moment strength must be determined since it is needed for the partial composite strength calculations. To perform this analysis, the effective width of the section, B_e , can be found using AISC *Specification* Section I3.1a. The composite section properties must be calculated for an effective section of the floor system (see Figure 5 for the parameter nomenclature). A plastic stress distribution is assumed for the full composite flexural strength. The steel AIB resists tensile demands below the plastic neutral axis (PNA). The compressive demands are resisted by the AIB above the PNA and the concrete topping slab, which has an effective width, B_e , and depth, D_s . The PNA of the composite section is determined by ensuring that the compressive forces in the concrete, C_c , and steel, C_s , are in equilibrium with the tensile force, T , from the steel cross section below the PNA.

The equations determining the compressive and tensile force resultants depend on the PNA location. There are five possible cases for the PNA. The first three cases are when the top of the precast panels or deep decking is below the bottom of the top flange. The PNA is located in (1) the steel top flange, (2) in the steel web above the top of panels/deep decking, or (3) in the steel web but below the top of panels/deep decking. The final two cases consider whether the top of the panels/deep decking is above the bottom of the top flange. These cases consist of the PNA located in the (4) steel top flange or (5) the web.

The PNA generally falls into the steel top flange, which means not all the topping slab is in compression (see Figure 5). For this case, the compressive force resultants C_c and C_s are determined with Equations 28 and 29, respectively. The compressive strength of the concrete, f'_c , is needed, along with the concrete cover above the AIB, D_c . The tensile force resultant is calculated with Equation 30, where A_s is the area of the steel in tension and F_y is the yield stress of the steel. The depth of the PNA, y_{pna} , is found by iteratively solving the equations below until equilibrium is satisfied.

$$C_c = 0.85 f'_c [B_e D_c + (B_e - b_{ft})(y_{pna} - D_c)] \quad (28)$$

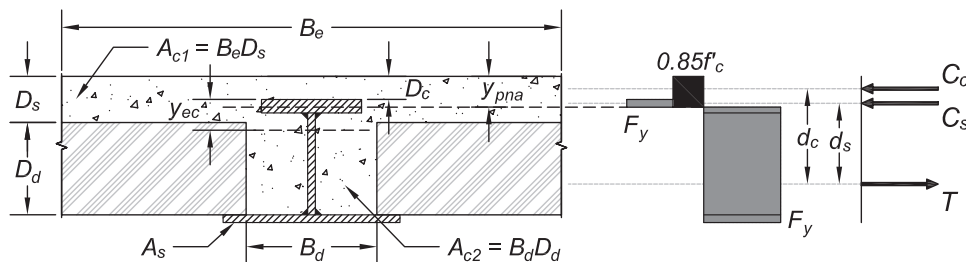


Fig. 5. Composite section nomenclature, plastic stress profiles, and force resultants.

$$C_s = F_y b_{ft} (y_{pna} - D_c) \quad (29)$$

$$T = A_s F_y - C_s \quad (30)$$

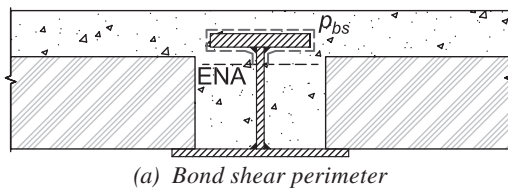
The force resultants are located at their respective centroidal locations, which are determined after the PNA is identified. These centroidal locations are calculated using standard geometric decomposition. Similar equations can be derived for the force components and their locations for the other PNA locations. The full composite plastic moment strength, M_{ncp} , is determined in Equation 31, which is the summation of moments due to the compressive forces in the concrete and steel about the tensile force centroid, with a moment arm of d_c and d_s , respectively.

$$M_{ncp} = C_c d_c + C_s d_s \quad (31)$$

A partial composite system will not achieve the full composite flexural strength as shown in Equation 31. However, a partial composite system can achieve significant flexural strength through the bond between the steel and the concrete. The steel-concrete bond is assumed to act around the top flange and web above the elastic neutral axis (ENA) with a uniform stress distribution (Lawson et al., 1997). The cross section will likely not reach the plastic state due to localized bond failure.

The ENA location, y_{ec} , is referenced from the top of the steel section as shown in Figure 5. The ENA can be determined by splitting the section into three areas: the AIB area, A_s , the topping slab above the panels or the top of the deep decking, A_{c1} , and the beam encasement area, A_{c2} . For the beam encasement area, the width of the encasement, B_d , and the depth of encasement, D_d , are needed (as shown in Figure 5). The ENA, y_{ec} , is determined with standard geometric decomposition, which results in Equation 32. This equation also includes the modular ratio, n , which is simply the ratio between the modulus of elasticity of the steel and concrete.

$$y_{ec} = \frac{A_s y_e + \frac{A_{c1}}{n} \left(\frac{D_s}{2} - D_c \right) + \frac{A_{c2}}{n} \left(d - \frac{D_d}{2} - t_{fb} \right)}{A_s + \frac{A_{c1}}{n} + \frac{A_{c2}}{n}} \quad (32)$$



(a) Bond shear perimeter

Bond Shear Strength

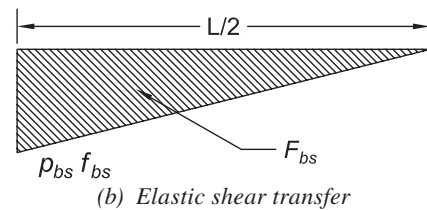
The first partial composite flexure limit state must determine the maximum compressive force that can be transferred between the concrete and steel from the bond shear, F_{bs} . This can be considered the longitudinal force to “push” the steel through the concrete when under flexure. Elastic shear flow is used for beams subjected to a uniformly distributed load. See Figure 6(b) for an illustration of the force transferred due to the formation of bond shear.

The bond shear strength is dependent on the bond shear perimeter, p_{bs} , which is illustrated in Figure 6(a) and calculated in Equation 33. The strength is also determined by the allowable bond shear stress, f_{bs} . The allowable bond shear stress recommended is 100 psi, which was determined through extensive experimental testing (further test details provided later). The research team adopted this allowable bond stress, which is the same value for filled composite members with rectangular cross sections (AISC, 2022). In literature, there is a wide range of estimates for the allowable bond shear, some of which are a function of f'_c (McCormac, 1981). However, experimental push-out tests show no distinct correlation between the allowable bond shear and f'_c (Roeder et al., 1999). The maximum compressive force resistance from the bond shear, F_{bs} , for simple span beams under uniformly distributed loading, is determined by calculating the resultant force that is transferred over half of the length of the beam [see Equation 34 and Figure 6(b)].

$$p_{bs} = 2(b_{ft} + y_{ec}) - t_w \quad (33)$$

$$F_{bs} = p_{bs} f_{bs} \frac{L}{4} \quad (34)$$

The partial composite moment strength can be calculated using several methods (further discussed later). The most conservative method linearly interpolates the strength between the steel-only yield moment, M_{sy} , and the full composite moment strength, M_{ncp} . Regardless of the determined flexural strength for the noncomposite (steel-only) section during the construction limit states, the steel-only yield moment is implemented as the minimum strength. This is because stability is not of concern since the concrete



(b) Elastic shear transfer

Fig. 6. Bond shear illustration.

has cured, providing continual section bracing. Therefore, the steel can reach the minimum yield moment strength of the compression or tension flange. For AIBs, the compression flange yield moment is determined with the steel yield strength, F_y , and the elastic section modulus of the top flange, S_{xc} , will control due to the asymmetry. Furthermore, the web plastification factor, R_{pc} , is also included per AISC *Specification* Section F4. This factor often increases the strength of the AIB to the full steel plastic moment strength. The minimum compression flange yield moment is defined in Equation 35.

$$M_{sy} = R_{pc} F_y S_{xc} \quad (35)$$

The percentage of composite action is determined by evaluating the maximum allowable compressive force resistance from the bond shear, F_{bs} , to the concrete compressive force, C_c , that forms in the slab if full composite action is achieved. See Equation 36 for the linear interpolation utilized to determine the partial composite moment strength, M_{nc} (Lawson et al., 1997).

$$M_{nc} = M_{sy} + \frac{F_{bs}}{C_c} (M_{ncp} - M_{sy}) \quad (36)$$

Aside from the linear interpolation method, other methods can be utilized to calculate the partial composite flexural strength. This research explored the partial plastic stress distribution method, which uses a plastic stress distribution approach. However, the compressive force resultant from the concrete is limited by the bond shear force. The other force resultants are found from equilibrium; then the resulting moment can be calculated. The research also considered the strain compatibility method. This method assumes a linear distribution of strains over the entire depth of the composite cross section. Unlike in the previous method, stress-strain models of concrete and steel are utilized in this procedure rather than a plastic stress distribution. The comparison of all three methods with composite beam testing (discussed later) found that the linear interpolation was accurate and slightly conservative (Ottmers et al., 2025a).

Longitudinal Concrete Shear Strength

The second limit state applies to HC or CLT panels that do not have a chamfered top edge, as shown in Figure 7. This limit state considers the failure mode from the panel to the edge of the top flange (line A-B; d_{A-B}), across the width of the top flange (line B-C), and down to the panel edge on the other side of the beam (line C-D; d_{C-D}), shown by the dashed line in Figure 7. The longitudinal shear force in the cross section is transferred by the allowable bond stress between the concrete and steel (100 psi recommended from full-scale testing, which is presented later) and the allowable concrete shear stress, f_{cs} , in Equation 37 (McCormac, 1981). The longitudinal shear available strength per unit length, v_{bs} , is quantified with Equation 38.

$$f_{cs} = 0.12 f'_c \quad (37)$$

$$v_{bs} = f_{cs} d_{A-B} + f_{bs} b_{ft} + f_{cs} d_{C-D} \quad (38)$$

The longitudinal shear limit state check loading demand depends on whether the floor system is shored during construction. Unshored construction is likely to be used for floor systems that utilize panels due to the panel span capabilities. Therefore, the load demand resisted by the longitudinal shear for unshored construction is only due to the in-service live load.

To evaluate the longitudinal shear required strength per unit length, v_d , the maximum shear flow due to the load demand is obtained. The shear flow is dependent on the maximum vertical shear force, V , due to live (and possibly dead) loads. The first moment of area of the concrete, Q_c , above line A-B-C-D in Figure 7, and the moment of inertia of the composite section, I_{xc} (shown later), are also utilized (see Equation 39).

$$v_d = \frac{V Q_c}{I_{xc}} \quad (39)$$

Serviceability Checks

The primary serviceability requirements for floor systems include deflections and vibration to ensure comfort for building residents. These criteria control in many cases for

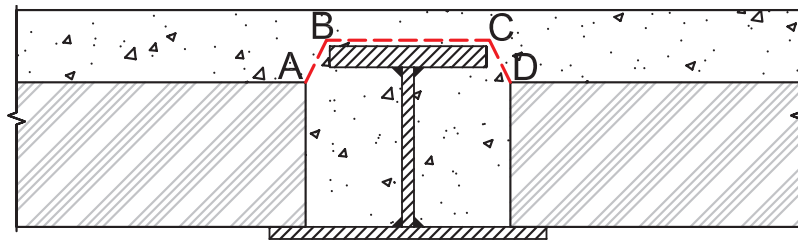


Fig. 7. Longitudinal shear transfer.

shallow-depth floor systems (even with a partial composite flexural strength). Designing for service deflections and vibrations is presented next.

Deflections

Vertical deflections of a floor system vary depending on whether shored or unshored construction is performed. Calculating deflections for unshored construction requires the AIBs to support the construction dead loads fully. The dead load deflection, δ_{DL} , due to uniformly distributed noncomposite dead load, w_{DL} , is evaluated with the AIB moment of inertia, I_x , as shown in Equation 40.

A common industry practice is to camber the beams, δ_{Camber} , to satisfy the deflection criteria. Cambering the beams also produces a relatively flat floor. Specifying camber is accomplished with approximately 80% of the dead load to ensure the camber is removed from the beam. However, a minimum camber of 0.75 in. is commonly required by fabricators. Further guidance on camber can be found in AISC Design Guide 36, *Design Considerations for Camber* (Kloiber and Burmeister, 2020).

The live load deflection, δ_{LL} , and superimposed dead load deflection, δ_{SDL} , are calculated according to Equations 41 and 42, respectively. The uniformly distributed live load, w_{LL} , and superimposed dead load, w_{SDL} , are evaluated with the uncracked transformed composite moment of inertia, I_{xc} . Note that if shored construction is used, the concrete will cure in the shored position. As a result, a portion of the dead deflection is determined with composite section properties.

$$\delta_{DL} = \frac{5w_{DL}L^4}{384E_sI_x} - \delta_{camber} \quad (40)$$

$$\delta_{LL} = \frac{5w_{LL}L^4}{384E_sI_{xc}} \quad (41)$$

$$\delta_{SDL} = \frac{5w_{SDL}L^4}{384E_sI_{xc}} \quad (42)$$

The moment of inertia for the uncracked composite transformed section, I_{xc} , can be quantified with Equation 43. Notice that the full composite section is utilized to capture the stiffness of the floor system. The full-scale testing (discussed later) found this to be the most accurate, yet conservative, assumption.

$$I_{xc} = I_x + A_s(y_e - y_{ec})^2 + \frac{A_{c1}D_s^2}{12n} + \frac{A_{c1}}{n} \left(\frac{D_s}{2} - D_c - y_{ec} \right)^2 + \frac{A_{c2}D_d^2}{12n} + \frac{A_{c2}}{n} \left(d - \frac{D_d}{2} - t_{fb} - y_{ec} \right)^2 \quad (43)$$

To ensure resident comfort, deflections from all dead and live loads (summation of Equations 40–42) must be less than the beam span length, L , divided by 240, and the live load deflection must be less than the span length divided by 360 (IBC, 2018). Note that the total deflection limit should also include concrete creep effects, which are not included here.

Vibration

Vibrations in steel floor systems can cause discomfort to residents, depending on their activity. To satisfy the vibration limit state, the ratio of the peak floor acceleration to gravity, a_p/g , of the floor system due to loading must be below the tolerated vibration levels for human comfort, a_p/g (Murray et al., 2016).

Similar to the deflection calculations, the composite uncracked transformed moment of inertia should be computed using the recommendations of AISC Design Guide 11, *Vibrations of Steel-Framed Structural Systems due to Human Activity* (Murray et al., 2016). However, the modulus of elasticity of concrete is increased by 35%, which modifies the modular ratio for vibration, n_v . The modular ratio for vibration replaces the modular ratio, n , in Equation 43 to quantify the increase in stiffness of the concrete deck during dynamic loading compared to static loading.

The natural frequency, f_n , is estimated based on the deflection of the floor system in Equation 44. Vibrations are usually the most severe when the floor is lightly loaded; therefore, expected day-to-day dead and live loads are utilized. Loadings consist of the structural dead load, an additional 4.0 psf for mechanical and ceiling installations when present, and live load for the applicable occupancy, as recommended by AISC Design Guide 11. The deflection for vibration, δ_v , is quantified similarly to the calculations in the prior section, but with the vibration-altered composite moment of inertia.

$$f_n = 0.18 \sqrt{\frac{g}{\delta_v}} \quad (44)$$

The type of loading producing the most severe dynamic response is dependent on the natural frequency of the floor system. For low-frequency floors, $f_n < 9$ Hz, the dynamic response is governed by the floor undergoing resonance from continuous footstep. For high-frequency floors, $9 \text{ Hz} < f_n < 15 \text{ Hz}$, the impulse response due to single footsteps governs the dynamic response. Floor systems with a steel deck and concrete slab having a natural frequency above 15 Hz are not known to be prone to excessive vibration (Murray et al., 2016).

The acceleration ratio, a_p/g , due to resonance is quantified in Equation 45. The amplitude of the driving force,

P_o , is a constant value of 65 lb. A damping ratio, β , ranges from 1 to 5% considering the structural system, ceiling and ductwork, furniture in the residence, and possible partitions (Murray et al., 2016). The effective panel weight, W , supported by the AIB is also needed.

$$\frac{a_p}{g} = \frac{P_o e^{-0.35 f_n}}{\beta W} \quad (45)$$

Predicted acceleration, a_p/g , is quantified in Equation 46 utilizing the damping ratio, β , floor natural frequency, f_n , and the effective panel weight, W . The footstep frequency, f_{step} , is determined based on the step frequency harmonic matching the natural frequency, h_s (Murray et al., 2016).

$$\frac{a_p}{g} = \frac{154}{W} \frac{f_{step}^{1.43}}{f_n^{0.3}} \sqrt{\frac{1 - e^{-4\pi h_s \beta}}{h_s \pi \beta}} \quad (46)$$

The peak floor acceleration response, either from resonance or impulse, is quantified and evaluated against the tolerant vibration levels for human comfort, a_p/g .

Fire

It is expected that some form of fire protection will be applied to the bottom flange of the AIBs (e.g., gypsum board, cementitious coatings, or intumescent coatings). The AIBs within shallow-depth floor systems have an inherent fire resistance due to concrete encasement. However, sustained high temperatures in the bottom flange can reduce the strength contribution of the bottom of the section. Therefore, fire protection is recommended, unless full-scale fire testing proves otherwise.

A lower-bound numerical fire study was performed where the AIB bottom flange was unprotected (Yarnold et al., 2025). The system performed adequately during fire events for an array of building geometries. As part of the numerical study, two strategies were evaluated for improved fire performance. The first was an increased AIB web thickness, and the second was fire reinforcement near the web of

the AIB (shown in Figure 8). Overall, it is recommended to add steel fire reinforcement as shown in Figure 8, instead of increasing the AIB web thickness. Fire reinforcement has a more significant impact on the performance while adding minimal cost. Note that this longitudinal reinforcement also increases the composite flexural capacity, which can be accounted for within the in-service design.

EXPERIMENTAL TESTING SUMMARY

An array of experimental testing was performed for AIBs within shallow-depth floor systems. The objective of the testing was to reduce the uncertainty of several design parameters and validate certain assumptions. Three main phases of testing were performed, which are summarized in this section.

Initial Concept Testing

The purpose of the initial large-scale testing was a preliminary constructability assessment of a shallow-depth steel-concrete composite floor system utilizing AIBs, along with a quantitative evaluation of the structural performance during construction and service loading. An initial proof-of-concept test was performed to analyze the behavior with precast HC panels. Figure 9 provides a typical cross-section detail. The AIBs utilized were a wide-flange shape with flanges cut down. These sections were available from earlier research (Stoddard and Yarnold, 2022).

Measurements were recorded throughout the construction process and during the simulated in-service loading. Figure 10 shows the setup during the final loading application. The primary takeaway from this work was that AIBs supporting precast panels and a concrete deck are a viable option for building floor systems. A detailed account of the initial concept testing and the results can be found in Davis et al. (2023). The critical findings included:

- Shallow-depth floor systems are easy and fast to construct by placing precast concrete panels on the bottom flange

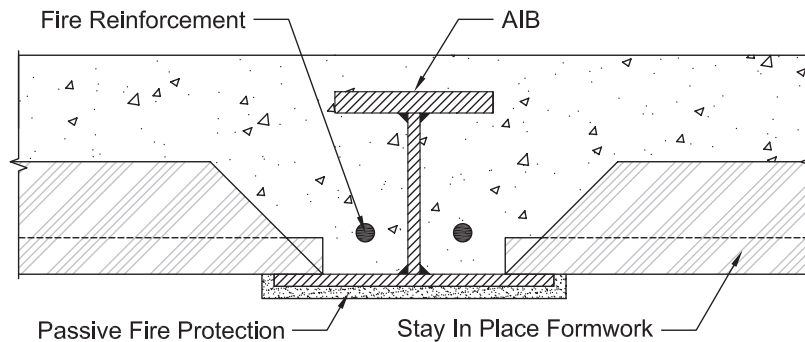


Fig. 8. Shallow-depth composite floor systems with fire reinforcement.

of an AIB and casting a deck slab that tops the panels and encases the steel beam.

- The noncomposite AIBs performed well in terms of lateral-torsional buckling and torsional response due to loading the beams on the bottom flange, creating a stabilizing load-height effect. The measured out-of-plane rotation was below the theoretical values.
- The floor system had a stiff response during service-level loading, indicating that the moment of inertia of the gross composite section can be utilized for serviceability analysis.
- Significant partial composite strength was achieved through bond shear forming between the steel beam and concrete. The composite floor system failed at an equivalent load of 500 psf applied using four-point bending over the center beam.

- Due to the beam spacing (limited by laboratory constraints) and the concrete not encasing the entire edge beam, the edge beams failed first due to load distribution from the center to the edge beams. Therefore, component-level beam tests and a full-scale system test with wider beam spacings were later utilized to analyze the composite behavior developed through bond stress for shallow-depth composite floor systems.

Composite Beam Testing

This testing aimed to evaluate the composite stiffness and strength of the shallow-depth steel-concrete composite floor system by conducting nine component-level beam tests (eight primary and one later validation). A detailed account of the initial eight tests can be found in Ottmers et al. (2025a), and the additional validation test can be found in Ottmers et al. (2025b).

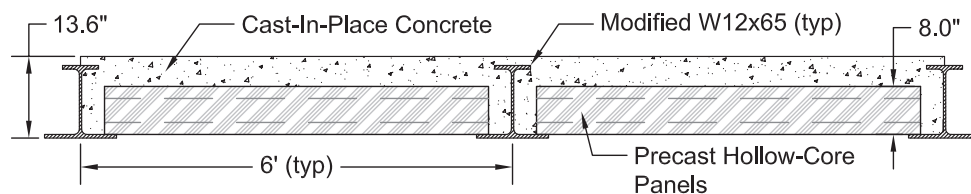


Fig. 9. Initial concept test cross section (slab reinforcement note shown).



Fig. 10. Initial concept testing.

Overall, the composite beam tests were designed to better understand the bond strength between the concrete and unpainted steel to address the uncertainty in composite action gained without mechanical connectors. Furthermore, the predicted partial composite flexural strength was analyzed using three methods: (1) linear interpolation between the steel and full composite moment strength, (2) partial plastic stress distribution, and (3) strain compatibility. These methods were compared to the experimental composite strength to establish the best methodology to analyze partially encased composite beams.

The composite beam tests were performed in three main stages: (1) lower load levels to analyze elastic behavior, (2) primary test cycle to define the partial composite strength, and (3) additional test cycles to analyze the reserve strength and ductility of the composite beams. Furthermore, the flexural stiffness of the composite section was analyzed at varying load levels. Figure 11 illustrates a component test and a sample loading time-history plot, with the three loading stages denoted. The steel beam yield moment, M_{sy} , and plastic moment, M_{sp} , are provided, along with the full composite strength, M_{ncp} , all using measured properties. The main conclusions drawn from the composite beam testing include:

- The flexural stiffness of the composite beams remained intact throughout the test, and the gross moment of inertia of the transformed section can be utilized for serviceability checks.
- The component-level beams reached 74 to 83% of the full composite strength in stage two loading before minimal bond slip occurred or a large flexural crack formed

(denoted as M_{fail}). The strength was well above the steel beam's plastic moment strength in all tests.

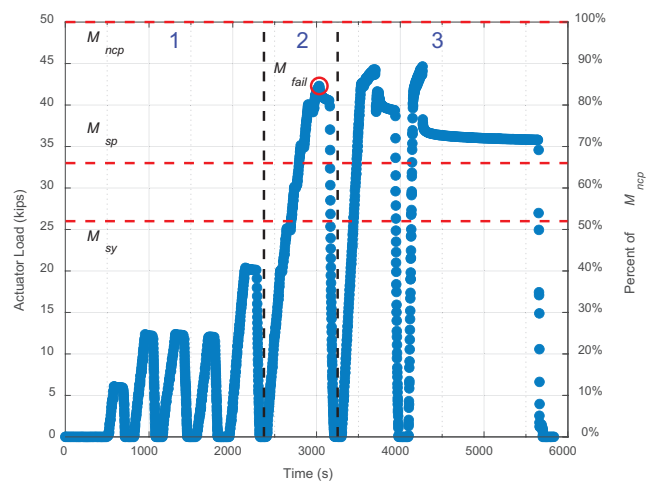
- The beams were reloaded in stage three and reached 77 to 91% of the full composite strength due to the bond reengaging between the steel and concrete, proving the ductile and robust behavior of the composite section.
- A bond stress of 100 psi undercuts most experimental results quantified with the maximum moment in stage two (111 psi average with 98 psi and 132 psi minimum and maximum, respectively). The constant bond stress was assumed to act over a bond perimeter above the composite section's elastic neutral axis. The maximum load transfer developed through bond shear is quantified as explained earlier in the Design Methodology section.
- The partial composite strength can be conservatively calculated using the linear interpolation method with a bond stress of 100 psi (this approach is recommended for future design with unpainted steel as outlined in the Design Methodology section). The partial plastic stress distribution and strain compatibility methods also provide reasonable results compared to the stage three experimental capacities.

System Validation Testing

A full-scale laboratory experiment was conducted to validate the use of AIBs within shallow-depth composite floor systems in multistory residential and commercial buildings. A prototype built-up plate girder equivalent to the A8×52 cross section (Ottmers et al., 2025c) was utilized in a system



Fig. 11. Example composite beam test under loading and load time-history plot.



test (dimensions shown in Table 1). Figure 12 shows the overall layout of the test specimen. The cross section was similar to that shown in Figure 1(a).

The system test setup was essentially two bays of a building. This consisted of three unpainted beams spanning between stub columns spaced 20 ft on center in both directions. New Millennium 4.5-in.-deep decking (18 gauge) was used to span over 19 ft between the A8×52 beams (Figure 13). A concrete slab was then cast monolithically to produce approximately a 10-in.-deep floor system that was 20 ft × 40 ft (center-to-center of the columns). A photo of the full-scale shallow-depth floor system is shown in Figure 14. The load frames shown were moved to various positions to conduct multiple loading scenarios. A detailed account of the full-scale validation testing design and execution can be found in Ottmers et al. (2025b).

The AIB behavior was evaluated throughout construction. This included the system constructability and the structural performance during the concrete slab pour. Next, vibration testing of the floor system was performed through heel drops, jumping, and walking scenarios. The floor system was then loaded in multiple service load locations to evaluate the stiffness. Finally, the center AIB was loaded until failure to determine the ultimate flexural strength (loading position shown in Figure 14). The main findings from the full-scale system testing included the following:

- The floor system was easy to construct, and the AIBs performed well structurally during construction. The measured deformations and rotations were below theoretical magnitudes with no signs of distress.

The deep decking was shored at midspan, per the manufacturer's requirements, which reduced the AIB demands. The maximum strain measured during the slab pour corresponded to a stress of 7.0 ksi.

- Vibration testing was performed in the constructed state to determine the fundamental frequencies and mode shapes. The first two natural frequencies of the floor system were 8.1 Hz and 9.3 Hz, and corresponded to the bending modes of the slab.
- Service-level testing was conducted in four locations around the slab to analyze the serviceability. This included loading at two locations near the midspan of the decking, close to the edge beams, and near the center beam. For midspan deck loading, equivalent to 80 psf, the maximum measured deflection was only $L/690$, indicating adequate deck stiffness.
- The AIB flexural stiffness was evaluated by loading directly over the center beam (shown in Figure 14). For a 100 psf equivalent area load over the center beam, the deflection of the floor system corresponds to $L/740$. The floor system had a relatively stiff response and further confirmed that the gross moment of inertia should be utilized for shallow-depth composite floor systems' stiffness calculations.
- The center AIB flexural performance from ultimate loading was excellent. The composite beam had a flexural stiffness greater than the gross moment of inertia of the composite section, quantified as two times the AISC *Specification* Section I3 specified effective width.

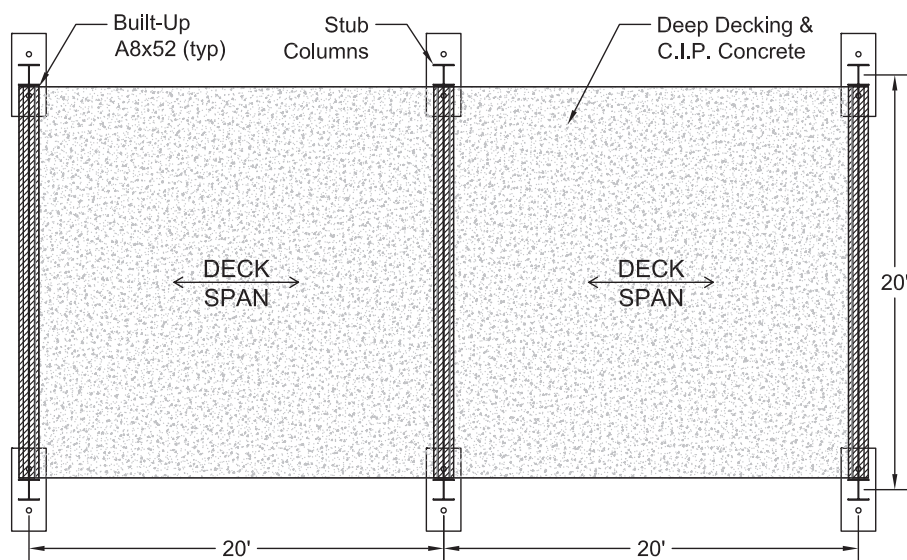
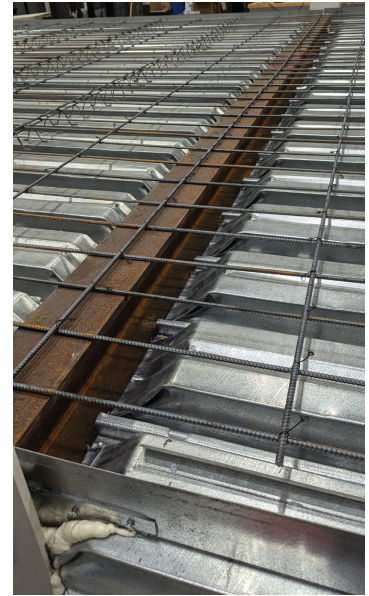


Fig. 12. System validation test plan view.



(a) AIBs supporting deep decking



(b) Close-up of center AIB

Fig. 13. System test specimen before deck casting.



Fig. 14. Full-scale system testing (final load position).

Furthermore, the system-level test reached 125% of the theoretical full composite strength. The overstrength and increased stiffness are due to more concrete than the specified effective width being engaged in the composite section. This was verified by measuring significant concrete strain on the slab surface beyond two times the specified effective width.

DESIGN AIDS

Design resources were developed to provide engineers with tools for the future design of steel-concrete composite floor systems utilizing AIBs. These resources are in the form of design tables and examples. The design tables provided are for hot-rolled A-shapes (dimensions shown in Table 1). However, the tables can be used for built-up AIBs with the same dimensions. The contribution of the fillets was not included in the calculations to produce the tables.

The first design table is for concentric noncomposite loading during construction (Table 2). The calculations for this table primarily follow AISC *Specification* Section F4. However, the stabilizing load height effect (discussed earlier) is incorporated. Note that the results are provided for allowable strength design (ASD) and load and resistance factor design (LRFD). Table 2 provides the available flexural strength for uniformly distributed loading. In addition, the available shear strength is provided. Note that the variables in the tables are consistent with the AISC *Steel Construction Manual* (2023).

The second design table is for eccentric noncomposite loading during construction (Table 3). The table provides the maximum distributed loading (ASD or LRFD) that can be applied to one bottom flange at a distance of 1.0 in. from the edge. This distance was selected because most deep decking and precast/prefabricated panel suppliers require a 2.0 in. minimum bearing length (placing the resultant force 1.0 in. from the edge). Overall, the intent of Table 3 is to allow a relatively easy check for anticipated construction loading. Table 3 follows the criteria provided earlier on a noncomposite design for eccentric loading.

The third design table is for composite loading in the completed state (Table 4). The available strength is provided for ASD and LRFD utilizing the earlier methodology. This includes the 100 psi bond stress (for unpainted steel) using the linear interpolation method for composite flexural strength (more conservative than the partial plastic stress distribution and strain compatibility methods). The effective width of the section is one-quarter of the span length. The concrete thickness above the top of the steel section is assumed to be only 1.5 in. However, the composite strength does not vary significantly with increasing the

topping thickness to 2.5 in. A welded wire fabric or single mat of steel reinforcement is recommended above the beam for crack control. This should be sized based on the grid geometry and decking type.

No mechanical connectors were assumed within the development of Table 4. If mechanical connectors (or other methods to increase the bond) are used, the flexural strength could be increased up to the full composite strength. The flexural strength could also be increased with supplemental longitudinal steel reinforcement (similar to that shown in Figure 8).

General capacity-to-demand tables were developed for each A-shape, but are not provided herein. Please refer to Yarnold et al. (2025), Appendix J. These tables include an array of beam lengths and spacings for different loading conditions.

Another valuable design resource that was developed is a comprehensive design example with all structural design checks. This can be found in Yarnold et al. (2025), Appendix I.

SUMMARY

The presented information aims to improve building floor system economy, speed, and efficiency. A design methodology was presented for shallow-depth composite floor systems utilizing AIBs (hot-rolled or built-up) with deck panels (HC or CLT) or deep metal decking. This methodology covers the limits states critical for construction in the noncomposite state and in-service as a composite system. The method presented is a lower-bound approach with no mechanical connectors for horizontal shear transfer (only bond and friction). Therefore, the calculations are for a partial composite design.

The design methodology was developed through an experimental testing program, which was summarized herein. This includes various experimental testing performed for AIBs within shallow-depth floor systems. The testing reduced the uncertainty of several design parameters and validated certain assumptions. Three main phases of testing were (1) an initial large-scale concept test, (2) nine composite beam tests, and (3) a full-scale system validation test. The critical findings of each phase were provided, which included the following recommendations.

- Shallow-depth floor systems are efficient to construct by placing panels or deep decking on the bottom flange of an AIB and casting a slab that tops the panels and encases the AIBs.
- A 100-psi steel-concrete bond stress should be used for concrete-encased (compression flange and web)

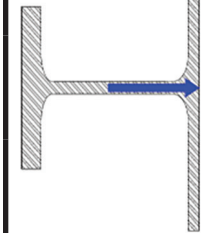
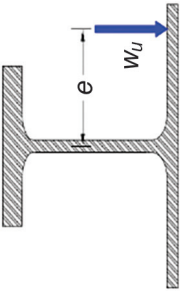
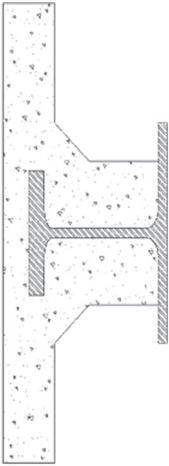


Table 2. Noncomposite Concentric Uniformly Distributed Loading Design Table
Available Flexural Strength
 kip-ft
 A-Shapes

$F_y = 50$ ksi

Shape	A12x				A10x				A8x				A6x		
	108		85		88		65		72		52		43		
Design	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	
		LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
	20	384	577	294	442	256	385	188	282	160	240	118	177	58	87
	21	384	577	294	442	256	385	188	282	160	240	118	177	58	87
	22	384	577	294	442	256	385	186	280	160	240	116	174	58	87
	23	384	577	294	442	256	385	181	272	160	240	112	169	58	87
	24	384	577	294	442	256	385	176	265	160	240	110	165	58	87
	25	384	577	287	431	256	385	171	257	160	240	106	160	58	87
	26	384	577	279	420	256	385	166	249	156	235	104	156		
	27	384	577	272	409	256	385	161	242	154	231	100	151		
	28	384	577	265	398	256	385	156	235	152	228	98	147		
	29	384	577	257	387	256	385	152	228	149	224	95	143		
	30	384	577	250	376	255	383	147	221	146	220	92	139		
	31	384	577	244	366	250	376	142	213						
	32	384	577	236	355	246	370	136	205						
	33	384	577	229	344	242	363	132	198						
34	383	575	221	332	237	356	127	191							
35	376	565	214	321	233	350	123	185							
ASD	LRFD	Available Shear Strength, kips													
$\Omega_b = 1.67$	V_n/Ω_{nv}	$\phi_v V_n$	V_n/Ω_{nv}	$\phi_v V_n$	V_n/Ω_{nv}	$\phi_v V_n$	V_n/Ω_{nv}	$\phi_v V_n$	V_n/Ω_{nv}	$\phi_v V_n$	V_n/Ω_{nv}	$\phi_v V_n$	V_n/Ω_{nv}	$\phi_v V_n$	$\phi_v V_n$
$\Omega_v = 1.50$	135	203	135	203	113	169	100	150	100	150	80	120	60	90	90

 Table 3. Noncomposite Eccentric Uniformly Distributed Loading Design Table Maximum Torsional Load kip/ft A-Shapes $F_y = 50 \text{ ksi}$ $e = 0.5b_{fb} - 1 \text{ in.}$													
Shape		A12x				A10x				A8x			
		108		85		88		65		72		52	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
15								2.40	3.60	3.03	4.55	1.43	2.15
16								2.03	3.05	2.79	4.20	1.20	1.80
17								1.76	2.65	2.59	3.90	1.03	1.55
18								1.53	2.30	2.43	3.65	0.90	1.35
19								1.33	2.00	2.16	3.25	0.80	1.20
20				1.96	2.95	1.90	2.85	1.20	1.80	1.93	2.90	0.70	1.05
21				1.73	2.60	1.66	2.50	1.06	1.60	1.73	2.60	0.63	0.95
22				1.56	2.35	1.50	2.25	0.93	1.40	1.56	2.35	0.57	0.85
23				1.40	2.10	1.33	2.00	0.86	1.30	1.43	2.15	0.53	0.80
24				1.26	1.90	1.23	1.85	0.77	1.15	1.30	1.95	0.47	0.70
25		1.40	2.10	1.16	1.75	1.10	1.65	0.70	1.05	1.20	1.80	0.43	0.65
26		1.26	1.90	1.06	1.60	1.00	1.50	0.63	0.95	1.10	1.65	0.40	0.60
27		1.16	1.75	0.96	1.45	0.93	1.40	0.60	0.90	1.03	1.55	0.37	0.55
28		1.06	1.60	0.90	1.35	0.86	1.30	0.53	0.80	0.93	1.40	0.33	0.50
29		0.96	1.45	0.80	1.20	0.80	1.20	0.50	0.75	0.86	1.30	0.30	0.45
30		0.90	1.35	0.77	1.15	0.73	1.10	0.47	0.70	0.83	1.25	0.30	0.45
31		0.83	1.25	0.70	1.05	0.67	1.00						
32		0.77	1.15	0.67	1.00	0.63	0.95						
33		0.70	1.05	0.60	0.90	0.60	0.90						
34		0.67	1.00	0.57	0.85	0.57	0.85						
35		0.63	0.95	0.53	0.80	0.53	0.80						
ASD	LRFD												
$\Omega_b = 1.67$													
		The available strength is based on AISC Specification Section H3.3.											
		$\phi_b = 0.90$											

 Table 4. Composite Design Table Available Composite Flexural Strength kip-ft Composite A-Shapes $F_y = 50 \text{ ksi}$ $f'_c = 4 \text{ ksi}$														
Shape	A12x				A10x				A8x				A6x	
	108		85		88		65		72		52		43	
Design	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Unbraced Length, L_b , ft	20	413	621	317	477	413	203	305	172	258	128	192	67	100
	21	414	622	318	478	413	203	305	172	258	128	193	67	101
	22	415	623	319	479	414	204	306	172	259	128	193	67	101
	23	415	624	319	479	415	204	307	172	259	129	194	67	101
	24	416	625	319	480	416	204	307	173	260	129	194	68	102
	25	417	626	320	481	416	205	308	173	260	130	195	68	102
	26	417	627	321	482	417	205	308	174	261	130	195		
	27	418	628	321	483	418	206	309	174	261	130	195		
	28	418	629	322	484	419	206	310	174	262	130	196		
	29	419	630	322	484	419	206	310	174	262	130	196		
	30	420	631	323	485	420	207	311	175	263	131	197		
	31	420	632	323	486	421	208	312						
	32	421	633	324	487	421	208	312						
	33	422	634	325	488	422	208	313						
	34	422	635	325	488	423	208	313						
	35	422	635	325	489	424	209	314						
ASD	Composite strength is quantified using linear interpolation between the steel and full composite moment capacity. The degree of composite action is determined with a bond stress of 100 psi that acts above the composite elastic neutral axis and develops over a quarter of the span length.													
$\Omega_b = 1.67$	LRFD													
	$\phi_b = 0.90$													

unpainted AIB partial composite flexural strength calculations.

- The gross moment of inertia is conservative for the in-service stiffness assumption when checking composite deflections and vibrations.
- The linear interpolation method is accurate, but conservative for partial composite flexural strength calculations. The partial plastic stress distribution and strain compatibility methods are also acceptable methods.
- The effective width per AISC *Specification* Section I3 (one-quarter the span length) is conservative for shallow-depth steel-concrete composite systems.
- Shallow-depth floor systems with AIBs are ductile and robust, withstanding substantial loading under repetitive load cycles.
- Finally, design aids are provided to equip future engineers with resources to design shallow-depth floor systems efficiently. These aids are in the form of design tables and a comprehensive design example.

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