Steel Structures Research Update

Future Hot-Rolled Asymmetric Steel I-Beams

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INTRODUCTION

Ongoing research on future hot-rolled asymmetric steel I-beams is highlighted. This study is currently under way at Texas A&M University, led by Dr. Matthew Yarnold, Assistant Professor in the Department of Civil and Environmental Engineering. Dr. Yarnold's research interests include building structural systems, bridge infrastructure, hybrid building energy-structure performance, and engineering education. Dr. Yarnold has been recognized for his teaching and research with the Robert J. Dexter Memorial Award, ASCE Nashville Branch Engineering Educator of the Year, an AISC Early Career Faculty Award, and AISC's Milek Fellowship. The four-year Milek Fellowship is supporting this research on the behavior of hot-rolled asymmetric I-beams. The research team is part way through year three of the four-year study. Selected results from a manufacturing study, full-scale experiments, and an initial sizing study are highlighted, along with a preview of future work.

MOTIVATION FOR THE STUDY

This investigation into the behavior of hot-rolled asymmetric I-shapes, or "A-shapes," is motivated by the expected gains in structural, fabrication, and construction efficiency.

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Improvements in steel building economy can be realized for shallow residential and commercial floor systems. Built-up asymmetric shapes have demonstrated advantages in composite construction, whether with precast or cast-in-place concrete. Precast concrete panels can be supported on the wider bottom flanges; the narrower top flange does not interfere with the panel during placement (Figure 1). This type of assembly could also be used for deep metal decking. For conventional cast-in-place concrete with the deck slab supported on the top flange, a narrower flange corresponds to its relatively low contribution to the strength and stiffness of the composite flexural member. However, the top flange is important for the bare-steel member's ability to support the construction loads. Currently, gains in structural and construction efficiency are outweighed by fabrication costs. Fabrication of current asymmetric shapes can be labor and material intensive with welding of pieces cut from two separate sizes of hot-rolled sections or welding of hot-rolled shapes and plates (Stoddard and Yarnold, 2022). Production of hot-rolled A-shapes would reduce fabrication costs while maintaining structural and construction efficiency.

RESEARCH GOAL AND PLAN

With the long-term goal of regular mill production of hot-rolled A-shapes, the research team is conducting an integrated numerical and experimental investigation into the behavior of these asymmetric shapes. The team aims to develop recommended A-shape cross-sections with comparable or improved structural efficiency compared to built-up asymmetric shapes (Stoddard and Yarnold, 2022). Over

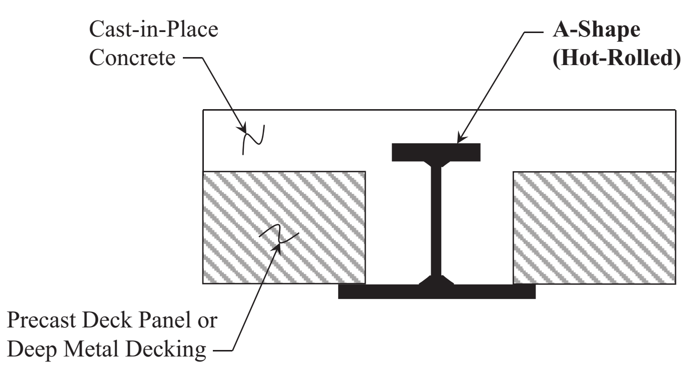

Fig. 1. Schematic of A-shape with cast-in-place concrete on deep metal decking or precast panel.

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the first two years of the project, the team conducted a manufacturing study, initial concept design and experimental testing, and an initial sizing study for residential facilities. Analytical models have also been developed and are being used to refine A-shape sizing. Experimental validation studies are also under way in the third year of the project, and year four will see a final shape study and results synthesized into design aids.

MANUFACTURING STUDY

Future production of the asymmetric shapes requires consideration of practical limitations and potential issues with hot-rolling of the A-shapes. Proportioning and sizing guidance was provided by mill experts. The producers also raised concerns about residual stresses and deformations after hot-rolling and cooling. A computational study and proof-of-concept (POC) beams were used to address those concerns.

Steel Mill Guidance

The steel mill guidance came from experts across different disciplines and producers. The three U.S. steel producers were Nucor, Steel Dynamics, and Gerdau. Industry roll-pass engineers, roll-pass designers, metallurgical engineers, roll-mill supervisors, and product developers collaborated with the researchers on recommendations for A-shape production. Cost considerations motivated a recommendation to match final flange dimensions with those of currently rolled shapes. Other sizing guidance included proportioning of shapes to have equal top and bottom flange areas, minimum web thickness of 0.50 in., and flange-to-web thickness ratios no larger than 2.0. Enlarged fillets were recommended to reduce cracking in the flange-to-web connection. A-shapes will be produced using ASTM A992/A992M steel.

Residual Stresses

Hot-rolled shapes develop residual stresses while cooling at the mill. Some steel experts expressed concerns for excessive residual compressive stresses and adverse impact on A-shape flange buckling. Those concerns were addressed through finite element modeling and reference to POC beams.

A three-dimensional, thermal-mechanical, finite element (FE) modeling approach was developed through a sensitivity study and reference to past research. A coupled thermal-displacement analysis utilized the nonlinear transient thermal and stress analysis capabilities of ABAQUS/CAE. Single tetrahedral elements were used through the flange and web thicknesses of simulated A-shapes. Element type and meshing were chosen with consideration for flange stress profiles, modeling of the flange-to-web fillets, and computational processing time. A modeled beam length of three times the depth allowed for sufficient development of the stresses (Stoddard, 2022). Beams were supported at corners only with pins or rollers to allow for free contraction. Temperature-dependent properties, such as elastic modulus and Poisson's ratio, were modeled based on published test results. The initial temperature of 2,372°F mimicked pre-rolling conditions; the beams then cooled to an ambient temperature of 68°F. Additional details can be found in Stoddard and Yarnold (2022).

The thermomechanical validation study included comparisons with POC beams. Simulated A-shapes were created with longitudinal cuts along the top flanges of W12×65 beams, as shown in Figure 2. The simulated A-shape had half the top flange width of the original W-shape. This procedure did not provide the correct grain structure for the rolled shapes but avoided the expense of retooling an entire roll line (Stoddard and Yarnold, 2022). The beams were reheated to approximately 1,740°F. Noncontact temperature measurements taken during the cooling process (Figure 3) were compared with the FE model results. Differences ranged from 1.9% to 15.8%, with larger differences being attributed to factors such as a slight wind in the facility. Additional information and a description of the literature comparison portion of the thermomechanical validation study can be found in Stoddard and Yarnold (2022).

The computational parametric study explored variations on two W-shapes. The W8×31 and W18×76 were selected to represent two section depths found in composite construction. The width or thickness of the top flange was then varied from 25% to 200% of the original. There were 30 unique cross sections, and the "limits produced unrealistic extreme cases that were included to determine the full spectrum of behavior" (Stoddard and Yarnold, 2022).

The residual stresses in the webs were most affected by variations in the flange properties. The beams were largely insensitive to changes in the flange thickness or width. The biggest changes in flange residual stress were seen with the width-to-thickness ratio, b/t. A b/t limit was introduced, corresponding to a flange residual stress of approximately 30% of the yield stress. Additional details on the residual stress study can be found in Stoddard (2022).

Deformations

Deformations (curvature) resulting from the cooling process were examined. A concern was that excessive deformations would require more rotary straightening and cause issues with handling throughout the mill. In addition, 30 ft beams were modeled and analyzed using the procedure described for the residual stress study. Initial imperfections were considered but were shown to have minimal impact on

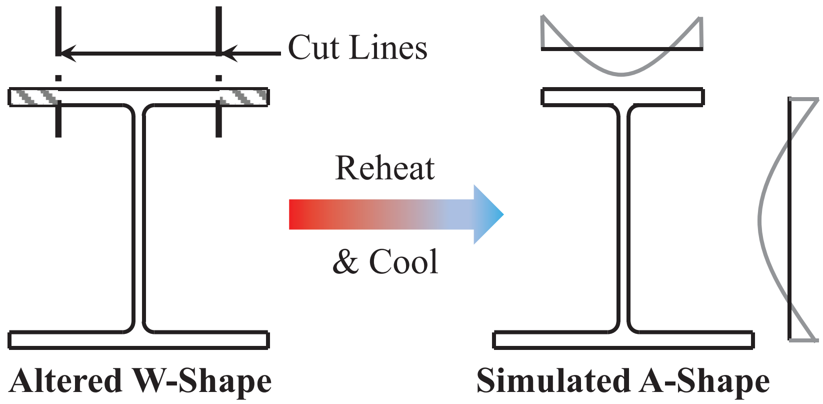


Fig. 2. W-shape altered, reheated, and cooled to simulate a hot-rolled A-shape; residual stresses qualitatively graphed on the simulated A-shape.

Fig. 3. Noncontact temperature measurements being taken of the cooling A-shapes.

the cooling deformations. The resulting camber was compared with the ASTM limit for a channel, another asymmetric hot-rolled section. Some beams did exceed the limit but were deemed to have small enough deformation to be corrected with rotary straightening (Stoddard and Yarnold, 2022). No cross-sectional limitation for cooling deformations was placed on future A-shapes. Additional details on the deformation study can be found in Stoddard (2022).

FULL-SCALE FEASIBILITY EXPERIMENTS

Full-scale experiments were used to evaluate the constructability and performance of a steel-concrete composite floor system with A-shapes. These experiments focused on A-shape beams with shallow precast panels and a cast-in-place topping slab. The experiments included three different phases of testing and contributed knowledge on the behavior of shallow composite steel-concrete members using A-shapes.

Experimental Setup

The test specimen and setup began with the steel frame for the construction loading (placement of panels), followed by the cast-in-place topping, and then vertical loading by the hydraulic actuator. There were three A-shape beams in parallel that spanned between stub columns and supported precast concrete panels with a cast-in-place topping (Figure 4). The load frame straddled the test specimen at mid-span. The center A-shape beam was loaded by an actuator and spreader beam.

The overall geometry of the test specimen was dictated by the laboratory and POC beams used. The floor tie-down locations are at 3 ft spacing. The load frame could be configured for 3 ft increments as well. With consideration for the POC beams, the tie-down locations, and the physical dimensions of the stub columns and load frame, a 6 ft POC beam spacing and approximate 23 ft length were chosen.

The test specimen used simple connections with modifications at the edge beams. Bolted double-angle connections connected all POC A-shape beams to the columns. However, concerns about eccentric loading of the edge beams during concrete topping placement prompted additions of top and seat angles at those locations. These angles were welded to the edge beam flanges and bolted to the columns. The top and seat angles could be unbolted at a later stage, converting the edge beams back to a simply supported condition (Yarnold, 2022; Davis, 2022).

Instrumentation included strain gages and string potentiometers for displacement measurements. The strain gages were placed on the top and bottom flanges of all beams at three different cross sections. One cross section was at beam mid-span; the other two were located 3 ft to either side. The displacement measurements were taken at the beam mid-span location. Each beam had a vertical displacement measurement. One edge beam also had two lateral displacement measurements that could be used to calculate rotation. Additional details of the test setup, instrumentation, and loading can be found in Davis (2022).

Experimental Results

The initial system experiment was designed to capture composite A-shape considerations and behavior at construction and in service. The experiments included Test 1: Precast panel placement, Test 2: Concrete topping slab, and Test 3: In-service and ultimate strength testing. The knowledge gained helped to inform the initial sizing of A-shapes.

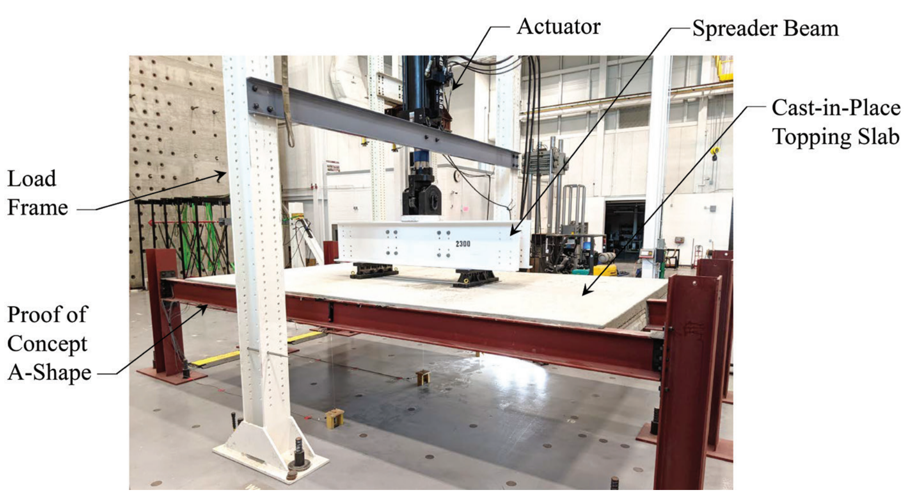


Fig. 4. Test setup for the full-scale experiment.

Test 1: Precast Panel Placement

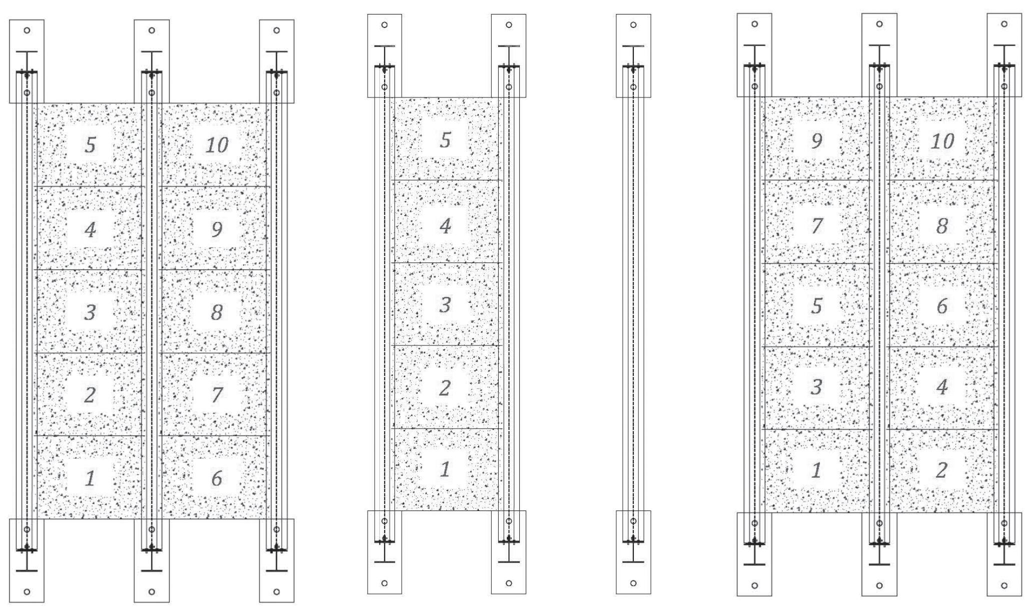
The precast panel placement test explored panel loading for different scenarios and beam boundary conditions. The 8-in.-thick, 4 ft × 6 ft hollow-core precast panels, supported on the beams' bottom flanges, were placed in two different sequences for a simply supported center beam and fixed-fixed edge beams. For loading scenario one, precast panels 1–5 were placed in order in one bay, and then panels 6–10 placed in the next bay, as shown in Figure 5(a). For loading scenario three, panel placement alternated between adjacent bays—that is, panel 1 in one bay, followed by panel 2 in the adjacent bay, and so on, as shown in Figure 5(c). For loading scenario two, the bolts in the top and seat angles were removed to revert back to pinned beam-to-column connections, and then one bay was loaded with panels 1–5, as shown in Figure 5(b).

Loading scenario two provided the most significant results. The center and edge beams experienced heavier torsional demands than in the other loading scenarios. The measured and theoretical combined bending and torsional stresses were compared, as were the edge beam rotations. Maximum measured and predicted stresses were 5.01 ksi and 5.29 ksi, respectively. The measured edge beam rotation of 1.62° compared favorably to the theoretical rotation of 1.76°. The theoretical stresses and rotations were conservative, due in part to the assumption of pure "pin" connections at the beam ends.

Test 2: Concrete Topping Slab

In preparation for casting of the concrete topping slab, spray expanding foam and silicone caulk were used to seal voids and joints between elements, such as the joint between the precast panel and bottom beam flange. Plywood formwork was built and fitted to the test specimen to achieve 1½ in. of cover over the top beam flanges. A grid of No. 4 Grade 60 reinforcing steel at 16 in. on center was hand tied and supported on 3 in. chairs and the center beam top flange. The slab was cast using a concrete dump bucket attached to the laboratory's overhead crane. During curing, the slab was sprayed with water every 6 hr. There was no need to cover the slab in the laboratory's temperature-controlled environment (Yarnold, 2022).

Concerns motivated deformation and capacity limit state checks for the construction loading. Measured out-of-plane rotation of the edge beams during the concrete pour was approximately 53% of the 4° limit. The researchers noted that while the rotations were "better than that anticipated," the edge beams would likely be stiffened for the torsional demands in a real building. Lateral torsional buckling (LTB) of the center beam exhibited a demand-to-capacity ratio of 0.45. Here, the researchers noted that the predicted capacity did not consider the stabilizing effect of the load placed at the bottom flange rather than at the top of the beam (Yarnold, 2022; Davis, 2022). The results suggested potential improvements and greater efficiencies in A-shape designs.



(a) Scenario one (b) Scenario two (c) Scenario three

Fig. 5. Order of panel placement for loading.

Test 3: In-Service and Ultimate Strength Testing

With the test frame shown in Figure 4, 220 a kip hydraulic actuator, and a spreader beam, a four-point bending test was applied at the center test beam. The load was applied with two steel pin assemblies attached to the spreader beam and spaced 6 ft apart. This created a constant moment region in the center of the beam. A loading rate of 2 kips per minute was used for evaluation of the in-service and ultimate strengths. The in-service loading was the equivalent of 100 psf for the floor system. The ultimate loading was equivalent to 500 psf.

The in-service loading results confirmed the viability of A-shapes for composite floor systems. Strain measurements were relatively low and corresponded to the expected neutral axis location for a fully composite section. Deflections were also low. For the design service load of 100 psf, the measured midspan deflections were on the order of the beam span/3000. The deflections easily satisfied a limit of span/360 (Yarnold, 2022; Davis 2022).

The ultimate strength test results suggested options for design. At a loading corresponding to 500 psf, there was a loss of composite action between the concrete and steel for the center beam. The load redistributed to the edge beams and was sustained for 20 min until unloading. Load at failure corresponded to 54% of the composite capacity. Mechanical connections or other means to increase the steel-concrete bond could be considered in design. Such means might be warranted for longer spans and/or loads, but the strength increase would come at the cost of construction speed and efficiency (Yarnold, 2022).

Theoretical vs. Experimental Comparison

Comparisons between the experimental results and theoretical predictions provided additional insights for analysis and design of these floor systems. Assumptions for the theoretical predictions include ideally pinned and fixed to establish bounds for the partially restrained connections, a 25%–50%–25% load distribution across the three beams, a composite beam effective width corresponding to the beam tributary width, and fully composite behavior. The measured strains and displacements for the center beam did show closer to ideal pinned connection behavior, as well as more fixity at the supports for the edge beams. The design assumption of pinned, or simple, connections was deemed to be conservative and appropriate (Davis, 2022). As mentioned previously, the data also confirmed composite action between the concrete and encased steel section at service-level loads. There are no mechanical connections; the composite behavior is developed from bond and friction (Yarnold, 2022). Additional details of the tests and comparisons can be found in Davis (2022).

INITIAL SIZING STUDY FOR RESIDENTIAL FACILITIES

A shallow-depth residential floor system case study was used to investigate the flange and web sizing of A-shapes. The case study utilized the configuration seen in the experiment—A-shapes supporting precast concrete panels and a cast-in-place topping slab. Flange and web geometries were varied for different beam depths, spans, and spacing. Beam depths and spacing ranged from 8 to 12 in. deep and 10 to 26 ft, respectively. In addition, 20 to 30 ft span lengths were considered.

The analytical study followed the loading from the initial construction stages to the building in service. Construction load cases (Load Cases 1, 2a, 2b) were noncomposite, and the in-service loading case was composite (Load Case 3) (Figure 6). The construction loads considered an eccentric loading for precast panel placement (Load Case 1), a uniform flexural demand from the concrete topping and construction live load of 20 psf (Load Case 2a), and an eccentric loading resulting in flexural and torsional demands (Load

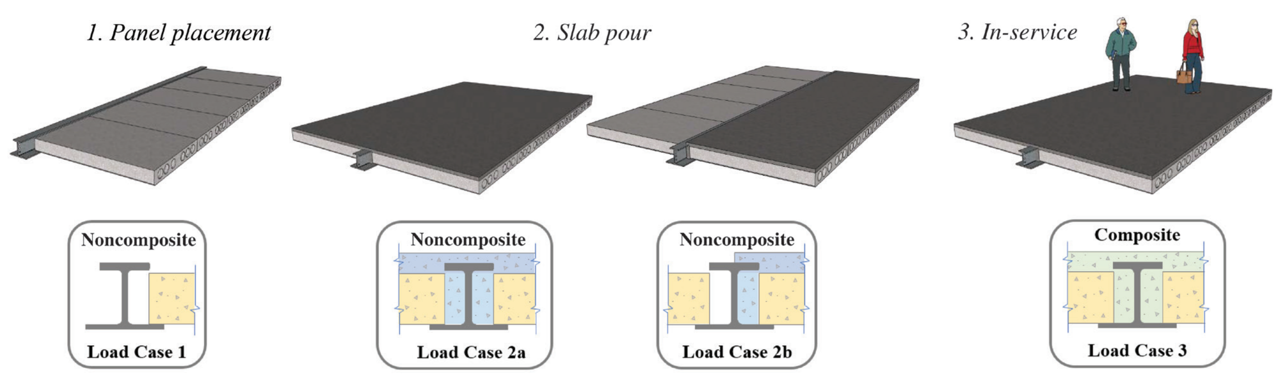


Fig. 6. Load cases for the initial sizing study for residential facilities.

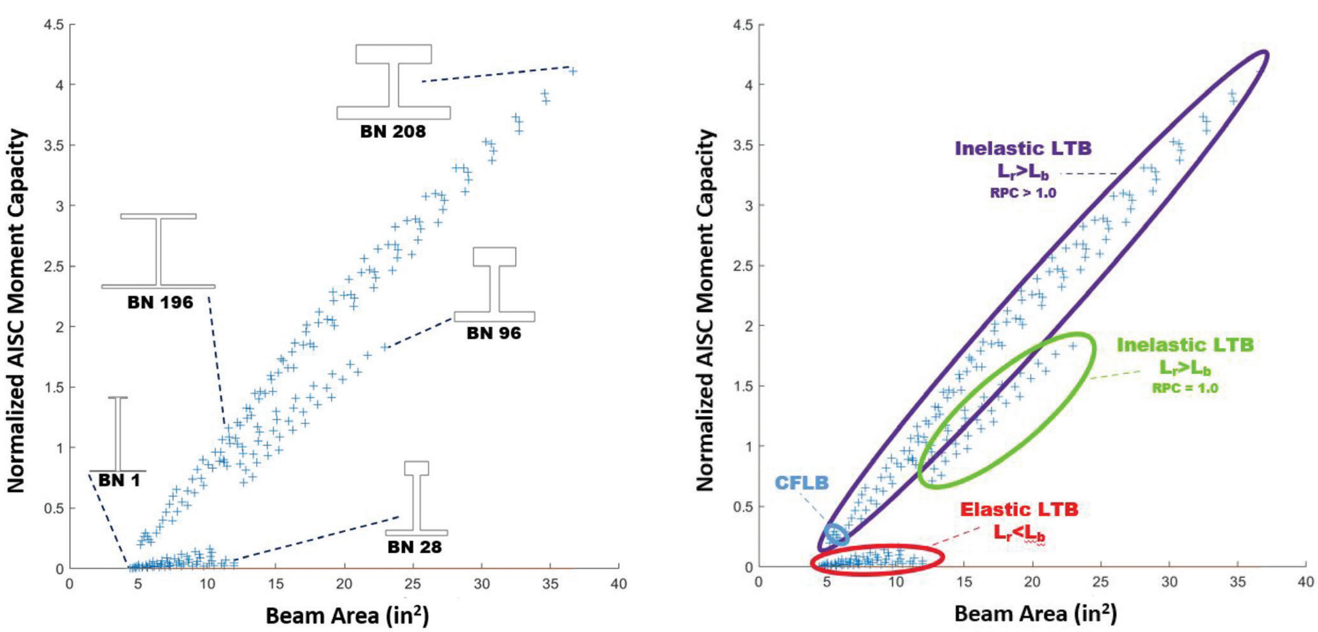
Case 2b). The beam end supports are modeled as pinned connections. The out-of-plane rotation limit was set at 4°, and beam span/240 is the service dead plus live load deflection limit. For the building in service, the composite A-shape beams were designed for a live load of 100 psf and a live load deflection limit of beam span/360. Following AISC Design Guide 11, *Vibrations of Steel-Framed Structural Systems Due to Human Activity* (Murray et al., 2016), the beams were also designed for a floor vibration live loading of 8 psf and a walking excitation loading of 65 lb (Stoddard, 2022).

A collection of A-shape cross sections was evaluated for the construction and in-service design parameters. Cross-sectional properties were dictated by the previously mentioned manufacturing constraints and seat width for the precast panels. Top flange areas were set equal to the bottom flange areas. Web thicknesses were constrained to at least 0.5 in. or half of the top flange thickness. Bottom flanges were required to be 4 in. wider than the top flange to provide a 2 in. seat for the panel. The collection was developed by varying the top flange geometry and adjusting the other parameters.

Controlling limit states varied by load case. Out-of-plane rotation controlled the A-shape designs for Load Case 1. The service dead and live load deflection limit controlled for Load Case 2a. Stability [i.e., lateral-torsional buckling (LTB)] was a controlling limit state for Load Cases 2a and 2b. Normalized moment capacities versus cross-sectional area are shown in Figure 7. Highlighted in Figure 7(a) are the ranges of moment capacity for the varying A-shape proportions. Figure 7(b) summarizes the controlling limit states of elastic LTB, inelastic LTB, and compression flange local buckling (CFLB). Meanwhile, vibration was the major design consideration for Load Case 3.

FUTURE RESEARCH

Future work is focused on refining the A-shape recommendations by revisiting limitations and conservative assumptions to improve the modeling, update sizing, conduct new experimental validation studies, and generalize the results for design. Limitations included the use of normal-weight concrete for one topping slab depth and one compressive strength. The team will investigate the potential advantages for variations in compressive strength and use of lightweight concrete. The team will also explore increased strength and stiffness versus higher dead load for different topping slab thicknesses. The rotation limit of 4.0° requires further investigation and justification. Conservative assumptions, including those made for initial vibration calculations, will be refined. Modeling and sizing of A-shapes will be updated accordingly, as experimental validation studies further expand knowledge about behavior of the recommended shapes. The research team will synthesize results from the analytical and experimental studies to produce design aids with supporting documentation.



(a) A-shape proportions *(b) Limit states*

Fig. 7. Normalized moment capacity vs. beam area organized by A-shape proportions and limit states.

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