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Potential for using tubular sections in open web steel

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Abstract
Open web steel joists provide for efficient support of roof and floor systems. During their erection, and before any bridging is in place, they are often highly susceptible to lateral-torsional buckling (LTB) because they are quite slender out of plane. Given that closed shapes, such as tubes, can in general provide excellent torsional resistance, this paper will present a pilot study that investigated the impact of using such shapes within steel joists. Although torsional resistance may have little impact on the capacity of members with smaller unbraced lengths, it was hypothesized that it could have significant potential when longer unbraced length conditions exist. Although the Steel Joist Institute currently specifies only the use of angles and double angles to fabricate joists, the alternative use of hollow structural sections (HSS) was still investigated. Employing the finite element analysis programs MASTAN2 and Strand7, the structural behavior of several joist configurations employing HSS shapes were compared with a standard 32LH06 joists comprised of angles. The results of the analyses indicate that constructing the chords of these trusses from hollow structural shapes can in some cases significantly increase their elastic LTB strength. Most importantly, this study shows that using HSS chords can provide the required torsional stability of unbraced joists, and thereby potentially eliminate bridging provided for erection and wind uplift conditions.

1. Introduction
Open web steel joists are structural trusses that have been used for almost 100 years as an efficient alternative to I-beams (Fig. 1). These trusses are stiff in plane and able to support large vertical loads, but require bridging to provide stability by connecting adjacent joists (shown in yellow in Fig. 1). Indeed, such bridging is an essential component of a complete joist system, and without bridging, the joists may fail in lateral-torsional buckling. In an effort to increase the lateral buckling capacity of the joists, and thereby reduce the amount of bridging required in such systems, this pilot study investigates the use of stiff HSS shapes in place of the routine application of structural angles in joist fabrication.

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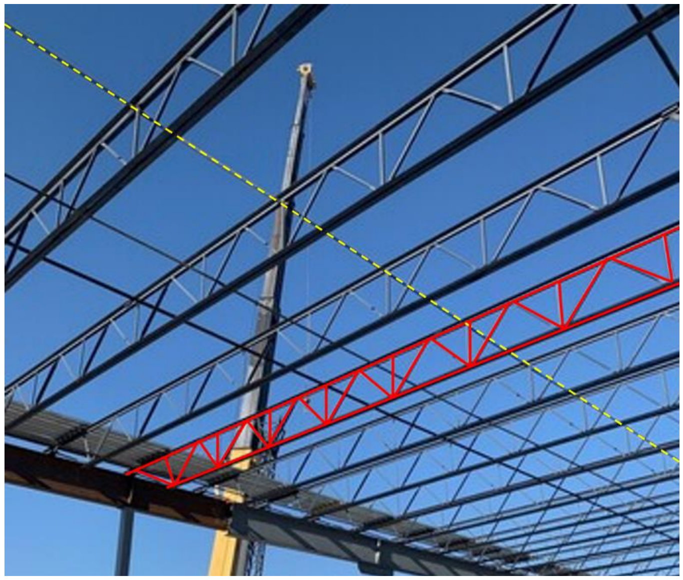

Figure 1: Roof system comprised of open web steel joists in red and horizontal bridging in yellow (SEAA 2020)

1.1 Joist Terminology

The main structural parts of an open web steel joist are the chords, which span the length of the truss on the top and bottom, and web members, which connect the chords at panel points (Fig. 2). Diagonal web members are important for both holding the chords together and transferring loads to the supports via shear action. Vertical web members are used to stiffen the compression chord to resist flexural buckling. When properly sized for strength, web members should have little impact on the overall stability of the joists. In many cases, the behavior of a joist can be treated as simple I-beam with its flanges defined by the chords of the truss. Although this study is based on rather complex finite element models, this simplification is also used in this study to both determine chords size for the joist designs studied and to initially predict their capacities.

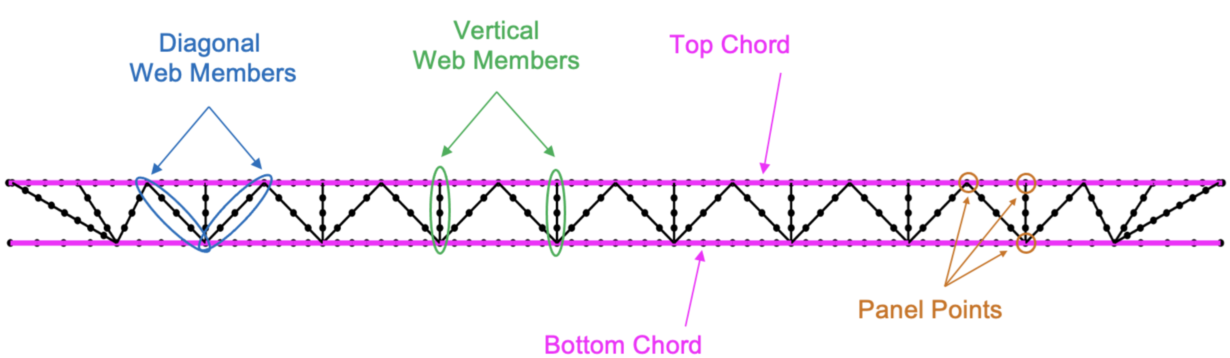

Figure 2: Open web steel joist geometry

1.2 Joist Designs

For this study, a 32LH06 joist is used as a control measure and is compared to three variations of the same joist, which are defined by which members are comprised of tubular sections. The typical joist is assembled with angle and double angle sections in a modified warren truss design with vertical web members. This configuration will be referred to as the Angles Design herein. This design uses double angles for the chords and end web members, and a single angle for the other web members (Fig. 3). Single angle webs are attached between the chords by first being crimped and then welded, while double angle webs are welded to the outside legs of the double angle chords. Most of the webs have slightly different dimensions, as indicated with different colors in Fig. 3. Specific details for this design are provided in Appendix A.

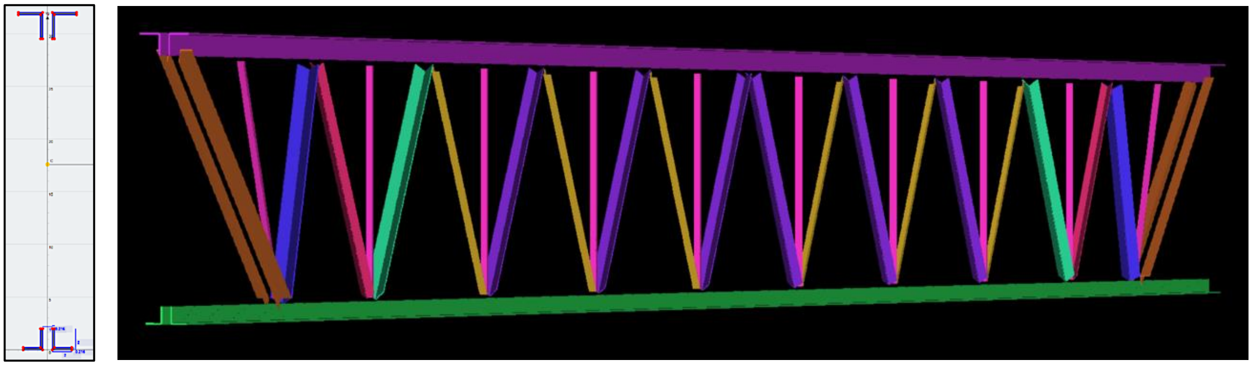

Figure 3: Angles Design chord cross-section and joist rendering, respectively

The first variation of the 32LH06 joist is fully made up from hollow structural steel (HSS) in an unmodified Warren truss design that does not include vertical web members (Fig. 4). This joist is referred to as the HSS Design and is based on a patented design by Armbrust (2017, 2019, 2020). To provide welded connections at the panel points, the web members must be mitered. In this HSS Design, the number of diagonal webs remains the same as that of the Angles Design. The dimensions of the web members are all the same except for the end webs, as indicated by the different colors in Fig. 4. Specific details for this design are provided in Appendix B.

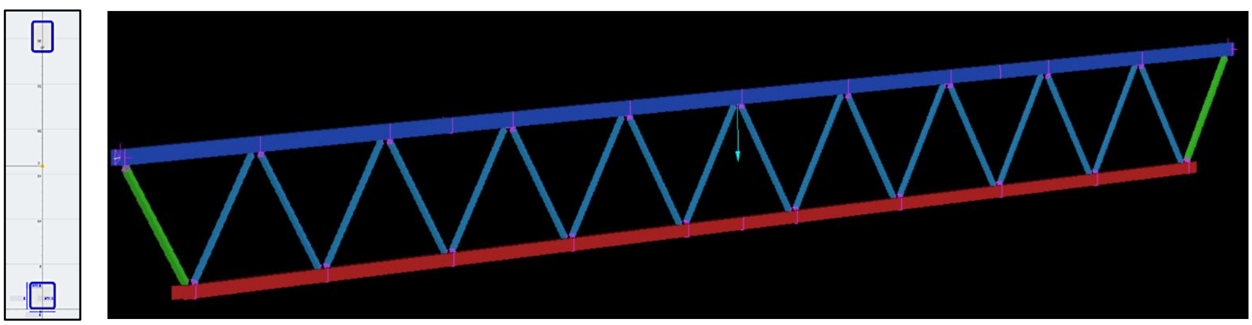

Figure 4: HSS Design chord cross-section and joist rendering, respectively

The next variation of the 32LH06 joist is also an unmodified warren truss using HSS for the chords and employing angles for the web members (Fig. 5). Referred to as the Hybrid 1 Design, this system uses the same chord members as the HSS Design, but provides a potentially more economical option of using angles in the web that theoretically, could provide similar capacity. Unlike the HSS Design, the web members are attached to the sides of the HSS chords, and hence, do not need to have mitered connections and thereby allow for easier assembly. Specific details for this design are provided in Appendix C.

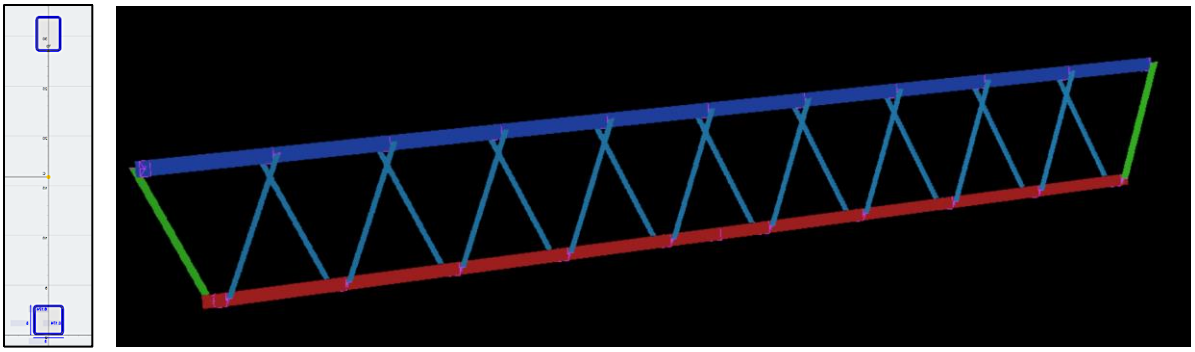

Figure 5: Hybrid 1 Design chord cross-section and joist rendering, respectively

The last variation of the 32LH06 joist studied is assembled with double HSS sections in the chords and angles for webs in a modified warren truss design with vertical web members (Fig. 6). This design will be referred to as the Hybrid 2 Design and was created in attempt to optimize the lateral-torsional buckling capacity by increasing both the out-of-plane flexure and torsional stiffnesses. The Hybrid 2 is most likely a less economical design than the Hybrid 1 because it uses additional HSS sections. All web members are identical to the original typical joist design. Specific details for this design are provided in Appendix D. It is noted that this joist design was developed later into this study than the other three joist designs, and comparative results are therefore not provided in all cases.

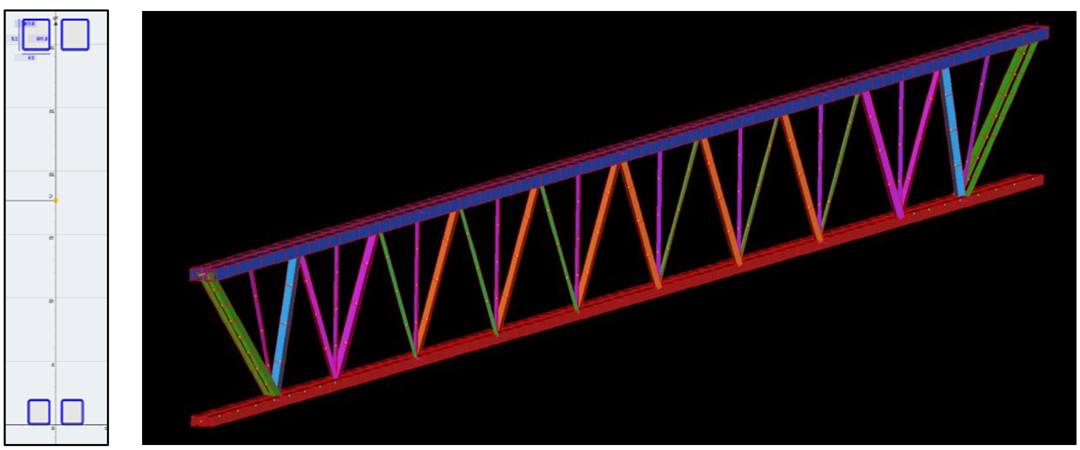

Figure 6: Hybrid 2 Design chord cross-section and joist rendering, respectively

1.3 Finite Element Software

In this study, two finite element analysis programs, MASTAN2 and STRAND7, were used to analyze the various joists. MASTAN2 is a free educational finite element analysis program developed by Ziemian, et al. (2020) that can model sophisticated line elements in three-dimensional space. Local and distortional buckling effects are neglected. STRAND7 is a commercial finite element analysis program (2021), which has similar functionalities to MASTAN2 with regards to line element modeling, but also provides for shell element modeling that can account for local and distortional buckling. Although this program can be more versatile, STRAND7 shell element analyses do take significantly more time to perform than MASTAN2 and STRAND7 line element analyses.

2. Background

Double angle cross-sections can provide acceptable flexural stiffness, but as open sections are torsionally quite flexible. In contrast, the closed HSS cross-sections can provide the same flexural stiffness, but orders of magnitude more torsional stiffness. To provide some initial insight to the impact on performance when using HSS chords in open web steel joists, one could estimate the lateral-torsional buckling capacity of a joist using Eq. 1 (AISC 2016)

$$M_{cr} = C_b \frac{\pi}{L_b} \sqrt{E I_y G J + \left(\frac{\pi \mathrm{E}}{L_b}\right)^2 I_y C_w} \tag{1}$$

where C_b is the moment gradient factor, L_b is the unbraced length, E is the modulus of elasticity (29,000 ksi), I_y is the moment of inertia about the minor axis, G is the shear modulus of elasticity (11,200 ksi), J is the St. Venant torsion constant, and C_w is the warping constant, which can be estimated using Eq. 2 (AISC 2016):

$$C_w = \frac{I_y {h_0}^2}{4} \tag{2}$$

where I_y is the moment of inertia of the member about its minor axis and h_0 is the height between flanges (chords).

While these equations are technically meant for doubly symmetric (e.g. I-shape) members, it can provide insight into how changes to the cross-section properties may impact the capacity of open web steel joists. It is important to note that Eq. 1 does not include any possibility for material nonlinear behavior. This equation also assumes a uniform cross section along the length of the truss with no initial imperfections. For the purposes of estimating joist capacities with these equations, the effects of the web members and the differences in the top and bottom chords are therefore assumed negligible.

Each joist's total self-weight and the cross-sectional properties of the chords are provided in Table 1. Although self-weight and chord cross-sectional area are not considered in Eq. 1, it is important to note that the models being compared in this study do use approximately the same amount of steel overall, as well in the chords, thereby ensuring that changes in capacity results are truly a reflection of changes to joist member's chord cross-sectional geometries.

Table 1: Self-weight and cross-section property comparisons between joists[1]

Design	S.W. (lbs)	A (in^2)	I_y (in^4)	J (in^4)	C_w (in^6)	h_0 (in)
Angles	836 [1.00]	3.66 [1.00]	6.68 [1.00]	0.05 [1.00]	1575 [1.00]	30.72 [1.00]
HSS	861 [1.03]	3.76 [1.03]	4.26 [0.64]	7.84 [156]	880 [0.56]	28.75 [0.94]
Hybrid 1	828 [0.99]	3.76 [1.03]	4.26 [0.64]	7.84 [156]	880 [0.56]	28.75 [0.94]
Hybrid 2	894 [1.07]	3.81 [1.04]	13.26 [1.99]	4.76 [95]	2934 [1.86]	29.75 [0.97]

1. Values in brackets are property values as a ratio of the Angles Design value for the same property

The HSS and Hybrid 1 Designs have the same chords, so it is anticipated that their capacities will be roughly identical. The HSS and Hybrid 1 Designs have a minor axis moment of inertia and warping constant of almost half that of the Angles Design. On the other hand, the HSS and Hybrid 1 Designs have a torsion constant that is over 150 times larger than the Angles Design. Although the HSS and Hybrid 1 torsion constants are much higher than the Angles torsion constant, the lower moment of inertia and lower warping constant suggests that there may be a critical unbraced length at which the Angles Design will outperform the HSS Design. The use of Eq. 1, however, suggests that this critical unbraced length is not achieved for joist length studies (Fig. 7).

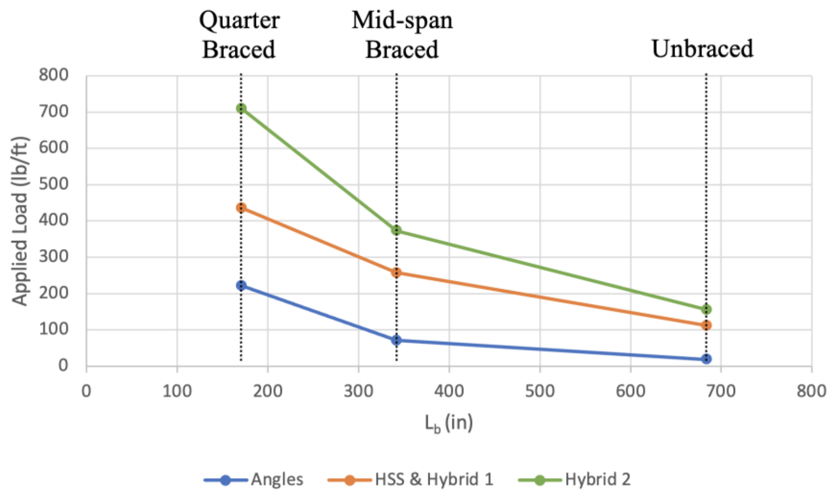

Figure 7: Anticipated joist capacities using Eq. 1 for various unbraced lengths

The Hybrid 2 Design was later chosen with the goal of maintaining the same weight and cross-sectional area, while attempting to maximize the warping constant, torsion constant, and minor axis moment of inertia.

In general, finite element analysis results are as accurate as the refinement used to model them. On the other hand, the more complex the model, the longer the modeling and analysis periods. To be able to run sufficient studies on the joists, models made of line elements, shell elements, and a combination of line and shell elements were compared using a linear eigenvalue buckling analysis. Although not a full geometric and material nonlinear analysis, linear buckling analyses were considered a good starting point, with more sophisticated analyses coming in future research. For the purposes of this study, the elastic critical buckling values determined from the shell-element models will be used as the basis for comparison. Based on the error analysis results presented in Fig. 8, the line element models are shown to provide sufficient accuracy for this work. The Hybrid 2 Design is not included because it was developed after this comparison was completed.

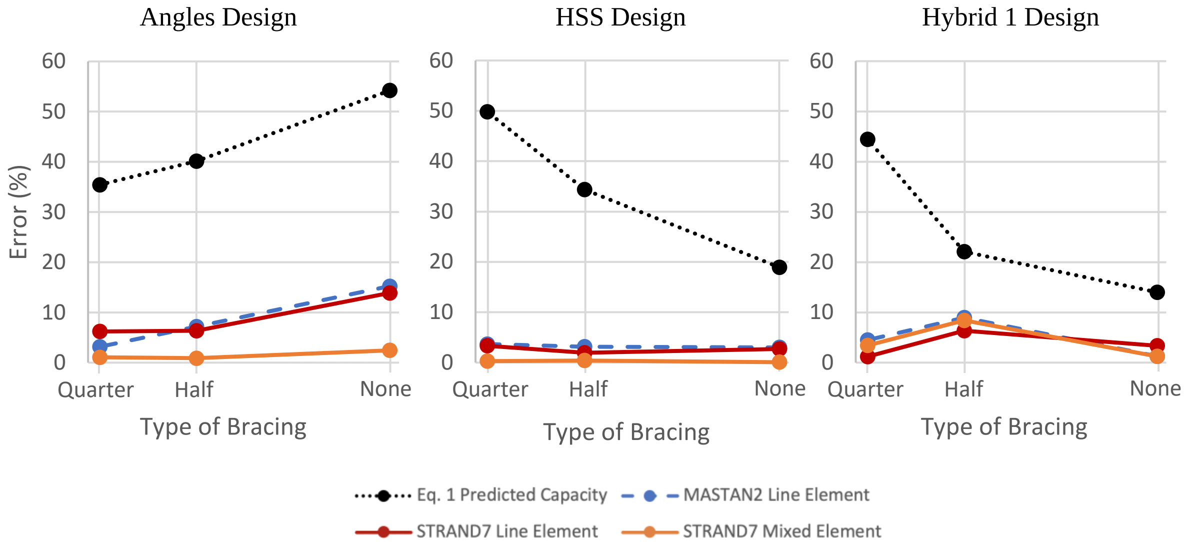

Figure 8: Comparison of linear buckling analysis for different finite element models

3. Analyses Details

Three different load cases were studied representing the main ways the joists may be loaded during their lifespan (Fig. 9). These loading conditions include a point load representing a worker disconnecting a joist from a crane during initial erection, distributed gravity loading representing deck live and dead loads transferred to the joist along its length, and wind induced uplift for roof joist systems. All three of these load cases were analyzed with and without self-weight.

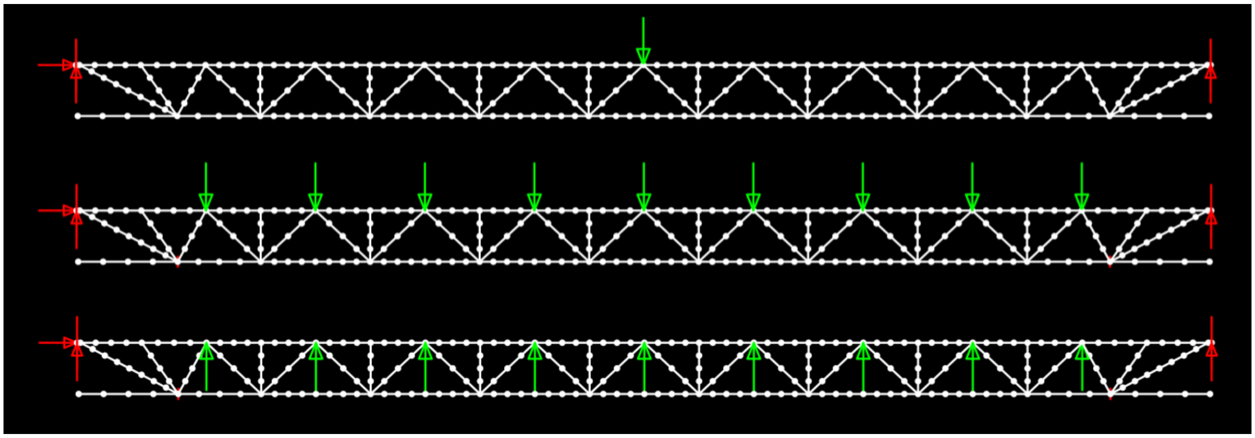

Figure 9: Point load, distributed gravity load, and distributed uplift load, respectively

Three types of analyses were used for the finite element models. For all load cases, a linear buckling analysis (LBA) was performed. With the exception of the uplift load case, geometrical nonlinear analyses (GNIA) in addition to geometric and material nonlinear analyses (GMNIA) were also performed. The initial system imperfection in the GNIA and GMNIA included a sine wave with an out-of-plane imperfection of 0.1 percent of the unbraced length. Imperfections between the joist panel points were not included.

Given that the GNIA is elastic, an instability is identified by the out-of-plane lateral deflections of the joist significantly increase as the joist undergoes lateral-torsional buckling. For the purposes of this study, a limit state was chosen for this analysis when the rate of the incremental lateral deflection reaches 0.02 in/lb. By defining this limit state, comparisons between GNIA and GMNIA results can be made to indicate the potential impact of yielding on joist design capacity.

4. Results

After completing LBA, GNIA, and GMNIA efforts for the various load cases, both the capacities and the factors impacting joist capacity can be better understood.

4.1 Point Load

For the point load case, the use of HSS in all three variations of the typical joist design provided substantial capacity increases as seen in the GMNIA with self-weight results provided in Table 2. As expected, the Hybrid 1 has a very similar capacity to the HSS Design, thereby indicating that the web members were not controlling the system instability. Given that the elastic and inelastic analysis results are quite similar, it appears that the unbraced joists were long enough that in each design, the buckling was fully elastic without any yielding. The initial imperfection has a similar impact within 5% on all designs. The self-weight has a much larger capacity impact on the Angles Design than any other joist.

Table 2: Point load[1]

	Limit State Capacity[2]		Failure Capacity			Capacity Impacts		
Design	GNIA (kips)	GMNIA (kips)	LBA (kips)	GMNIA (kips)	GMNIA self-weight (kips)	Yielding (%)	Initial imperfection (%)	Self-weight (%)
Angles	0.9 [1.0]	0.9 [1.0]	1.1 [1.0]	1.0 [1.0]	0.6 [1.0]	0	9	43
HSS	2.3 [2.7]	2.3 [2.7]	2.9 [2.6]	2.8 [2.8]	2.3 [4.0]	0	5	17
Hybrid 1	2.2 [2.7]	2.2 [2.7]	2.8 [2.5]	2.6 [2.6]	4.2 [3.8]	0	7	17
Hybrid 2	3.9 [4.6]	3.9 [4.6]	4.7 [4.3]	4.3 [4.3]	3.9 [6.8]	0	9	10

1. Values in brackets are the Angles capacity ratio
2. Capacity when the incremental lateral deflection reaches 0.02 in/lb.

The capacities of these joists depend purely on the cross-sectional geometry of the joists. Inclusion of self-weight effects reduces each joist capacity by approximately the same amount because all designs have similar weights. Percentagewise, the consistent capacity impact from self-weight influences lower capacity joists more because the self-weight is a larger percentage of the total joist capacity.

4.2 Unbraced Distributed Gravity Load

For the unbraced distributed gravity load case, the three HSS variations of the typical joist, again, provide substantial capacity increases, as seen in the GMNIA with self-weight results provided in Table 3. Similar to the point load case, the Hybrid 1 has a very similar capacity to the HSS Design, and the unbraced joists were long enough that in each design, buckling occurs without any yielding. The initial imperfection has a larger impact on the distributed gravity load case compared to the point load design. With exception of the Hybrid 2 Design, all joists are influenced by the geometric nonlinear effects more significantly than in the point loading case. In this loading case, the Angles Design is impacted slightly more than the Hybrid 1 and more than twice as much as the Hybrid 2. Again, the self-weight has a much larger capacity impact on the Angles Design than any other joist – for the same reason mentioned above.

Table 3: Unbraced distributed gravity load[1]

Design	Limit State Capacity[2] GNIA (plf)	Limit State Capacity[2] GMNIA (plf)	Failure Capacity LBA (plf)	Failure Capacity GMNIA (plf)	Failure Capacity GMNIA self-weight (plf)	Capacity Impacts Yielding (%)	Capacity Impacts Initial imperfection (%)	Capacity Impacts Self-weight (%)
Angles	25 [1.0]	25 [1.0]	34 [1.0]	29 [1.0]	17 [1.0]	0	15	42
HSS	68 [2.8]	68 [2.8]	99 [2.9]	82 [2.9]	68 [4.1]	0	17	17
Hybrid 1	67 [2.7]	67 [2.7]	91 [2.7]	79 [2.7]	64 [3.8]	0	14	19
Hybrid 2	121 [4.9]	121 [4.9]	154 [4.5]	142 [4.9]	128 [7.6]	0	7	10

1. Values in brackets are the Angles capacity ratio
2. Capacity when the incremental lateral deflection reaches 0.02 in/lb.

As in the point load case, the capacities of these joists again depend purely on the cross-sectional geometry of the joists and inclusion of self-weight effects reduces each joist capacity by their own weight.

4.3 Mid-span Braced Distributed Gravity Load

For the mid-span braced distributed gravity load case, the HSS in all three variations of the typical joist design continued to provide capacity increases, as seen in the GMNIA with self-weight results in Table 4. The HSS and Hybrid 1 Designs capacities differ more than in the unbraced case, mainly because yielding is now impacting the performance of the Hybrid 1 Design significantly more than any of the other joist designs. The initial imperfection (now modeled as a sine wave between brace points) has a lesser impact in this case than the other joist designs. Self-weight also has less of an impact on the Angles Design than the unbraced case, and does not impact the buckling capacity of the Hybrid 2 Design as much as the other joist designs.

Table 4: Mid-span braced distributed gravity load[1]

Design	Limit State Capacity[2] GNIA (plf)	Limit State Capacity[2] GMNIA (plf)	Failure Capacity LBA (plf)	Failure Capacity GMNIA (plf)	Failure Capacity GMNIA self-weight (plf)	Capacity Impacts Yielding (%)	Capacity Impacts Initial imperfection (%)	Capacity Impacts Self-weight (%)
Angles	82 [1.0]	82 [1.0]	111 [1.0]	93 [1.0]	80 [1.0]	0	16	14
HSS	151 [1.8]	149 [1.8]	192 [1.7]	170 [1.8]	156 [1.9]	1	10	9
Hybrid 1	153 [1.9]	131 [1.6]	186 [1.7]	132 [1.4]	117 [1.5]	14	15	11
Hybrid 2	277 [3.4]	274 [3.3]	352 [3.2]	295 [3.2]	280 [3.5]	1	15	5

1. Values in brackets are the Angles capacity ratio
2. Capacity when the incremental lateral deflection reaches 0.02 in/lb.

The large yielding impact on the Hybrid 1 Design is likely a result of how the web members are attached to the chords. In the unbraced case, the alternating position of the webs does not impact the direction in which the joist will undergo lateral-torsional buckling. With mid-span bracing, both unbraced lengths have an imbalance in the number of webs on each side of the joist chord, thereby creating more of an incentive for the joist to undergo lateral-torsional buckling in the direction of the imperfection.

4.4 Quarter Braced Distributed Gravity Load

For the Hybrid 2 Design, yielding occurred in two web members shown on the deflected joist shape (shown in blue in Fig. 10) with plastic hinges in the web members (yellow bowties in Fig. 10). The deflected shape shows more drastic deflections after the webs yielded, thereby showing nonuniform behavior along the entire length of the joist, and significantly impacting the GMNIA results. To keep this study focused on how the chords impact joist capacity, yield strength of these two web members were artificially increased to avoid such premature member web yielding.

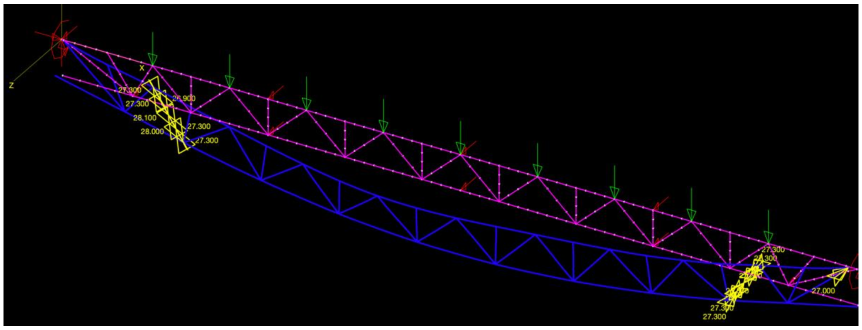

Figure 10: Web member yielding of the quarter braced Hybrid 2 Design

For the quarter braced distributed gravity load case, and as shown in Table 5, the Hybrid 2 Design is the only joist variation that provides a capacity increase as seen in the GMNIA with self-weight results. Yielding of the chords has a large impact on both hybrid designs, with initial imperfections have similar impacts on all designs. In this load case, self-weight is not as significant as the loading cases for any of the designs.

Table 5: Quarter braced distributed gravity load[1]

	Limit State Capacity[2]		Failure Capacity			Capacity Impacts		
Design	GNIA (plf)	GMNIA (plf)	LBA (plf)	GMNIA (plf)	GMNIA self-weight (plf)	Yielding (%)	Initial imperfection (%)	Self-weight (%)
Angles	281 [1.0]	268 [1.0]	356 [1.0]	288 [1.0]	273 [1.0]	4	15	5
HSS	226 [0.8]	221 [0.8]	288 [0.8]	240 [0.8]	225 [0.8]	2	15	6
Hybrid 1	212 [0.8]	189 [0.7]	280 [0.8]	201 [0.7]	186 [0.7]	11	17	7
Hybrid 2	668 [2.4]	488 [1.8]	858 [2.4]	504 [1.8]	490 [1.8]	27	14	3

1. Values in brackets are the Angles capacity ratio
2. Capacity when the incremental lateral deflection reaches 0.02 in/lb.

The yielding in the hybrid designs is likely a result of the web member designs. The web members for the Hybrid 1 model were proportioned based on their cross-sectional area for stiffness rather than their strength capacity, and the web members for the Hybrid 2 were designed for an overall joist capacity.

4.5 Impact of Unbraced Length

The sensitivity of joist capacity to changes in unbraced length was also studied. Fig. 11 shows a comparison of the predicted joist capacities from Eq. 1 to the GMNIA results that do not include self-weight. While capacities predicted by Eq. 1 should be fairly close to the more accurate capacities determined from the GMNIA results, the Angles Design provides higher GMNIA values, while the other three models provide significantly lower GMNIA capacities. This illustrates the importance of including material nonlinear behavior when estimating HSS joist capacities, especially when unbraced lengths are reduced.

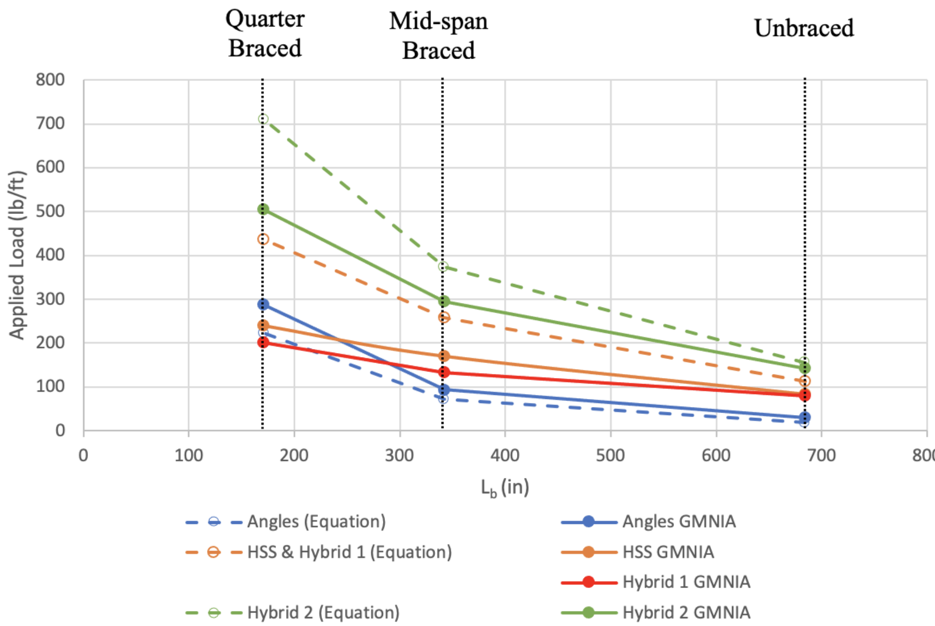

Figure 11: GMNIA capacities compared with anticipated results

By comparing the distributed gravity load GMNIA results including self-weight for the various unbraced lengths, the impact of providing vertical web members becomes apparent (Fig. 12). The Angle and Hybrid 2 Designs, which both include vertical web members show a greater rate of increase in capacity as the unbraced length decreases when compared to the other designs. This also suggests that the web members may have a more significant impact on joist performance than initially assumed.

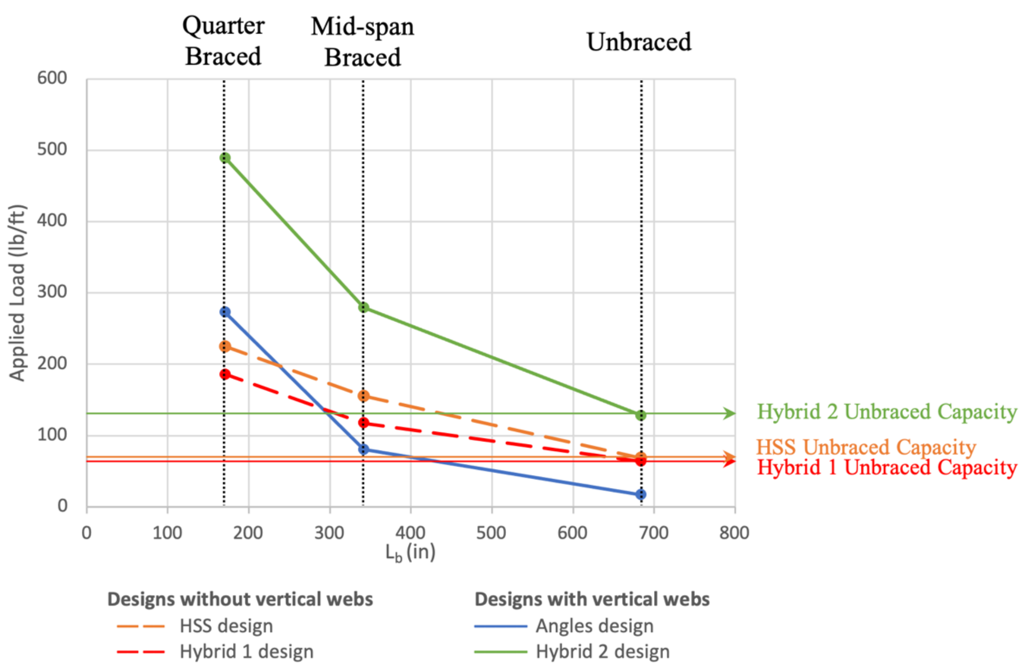

Figure 12: GMNIA distributed gravity load capacity as a function of unbraced length with self-weight

This portion of the study also shows that only the Hybrid 2 Design could eliminate some bridging as indicated by the unbraced capacities provided in Fig. 12.

4.5 Distributed Uplift Load

The uplift loading condition, while important, was not evaluated as comprehensively as the distributed gravity load analysis for this study. Only an LBA study was performed, but the impact of including self-weight was still analyzed. Unlike the previous loading cases, self-weight is counteracting the applied loads, and therefore increases the uplift capacities of the joists. The Hybrid 2 was not included in this study due to its late entry in this study. All uplift load cases including bridging on the bottom chords, regardless of how the joist was braced for gravity loading.

The HSS and Hybrid 1 Designs outperformed the Angles joist in all cases (Table 6). In the unbraced case, the HSS and Hybrid designs provide approximately 150% of additional capacity and in the quarter braced case, they provide approximately 30% additional capacity. Both the HSS and Hybrid 1 Designs also allow for some reduction in required bridging (Fig. 13).

Table 6: Distributed uplift load[1]

	Without self-weight			With self-weight		
Design	Unbraced (plf)	Mid-span braced (plf)	Quarter braced (plf)	Unbraced (plf)	Mid-span braced (plf)	Quarter braced (plf)
Angles	54 [1.0]	115 [1.0]	265 [1.0]	70 [1.0]	130 [1.0]	280 [1.0]
HSS	158 [2.9]	240 [2.1]	355 [1.3]	174 [2.5]	255 [2.0]	370 [1.3]
Hybrid 1	158 [2.9]	236 [2.1]	347 [1.3]	174 [2.5]	251 [1.9]	363 [1.3]

1. Values in brackets are the Angles capacity ratio

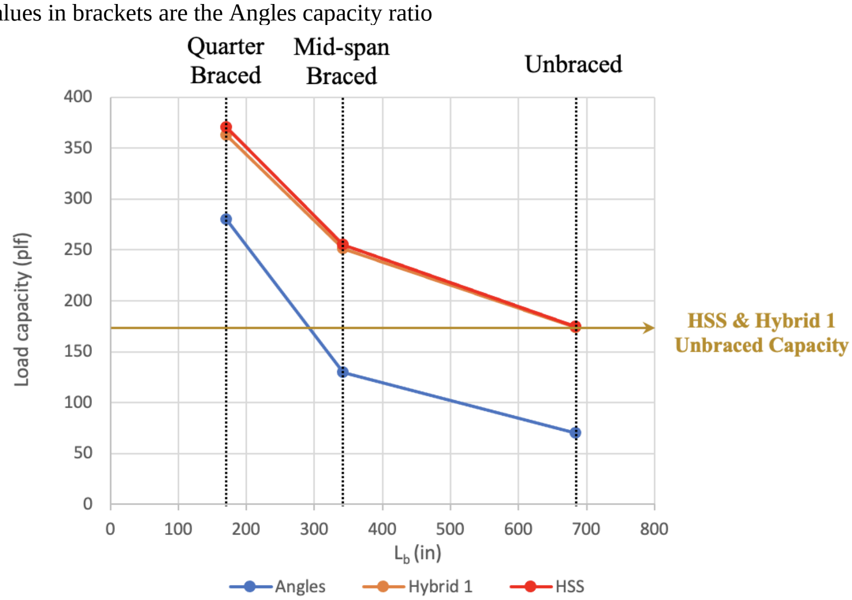

Figure 13: LBA uplift capacity as a function of unbraced length with self-weight

5. Conclusions

As consequence of open web steel joists having low out-of-plane flexural and torsional stiffnesses, they can often be highly susceptible to lateral-torsional buckling – especially during erection before any bridging may be in place. Given that closed shapes, such as hollow structural sections, can in general provide excellent torsional resistance, this paper presented a pilot study that investigated the impact of using such shapes in place of structural angles used to fabricate such joists. Employing two finite element analysis programs, MASTAN2 and Strand7, which provided for linear buckling and nonlinear buckling analyses using line and/or shell elements, the structural behavior of several joist configurations employing HSS shapes were compared with a standard 32LH06 joist comprised of angles.

Based on the analysis results obtained in this study, it was shown that the use of HSS in the chords could substantially increase the LTB strength of a joist, and potentially reduce the amount of bridging required, but not necessarily fully eliminate its need. While HSS chords provide higher load capacities during the erection of a joist and when loads are distributed over long unbraced lengths, the unbraced capacity may not exceed the capacity of the standard Angles Design when unbraced lengths are more reasonable. For example, it was observed that when bridging is provided at the span quarter points, the joist's strength limit state is defined by in-plane yielding, in which both HSS and structural angles provide the similar strengths. With this in mind, it appears that the main benefit of using HSS sections in the chords is the increased capacity provided during the joist erection process – a situation that could benefit construction safety (Armbrust, 2020).

When joist capacity is primarily controlled by the chords, a joist comprised of all members as HSS only slightly outperformed a design with HSS for chords and angle for webs (Hybrid 1 and Hybrid 2 Designs). Given that angle webs can be more readily welded to the sides of the HSS chords, and thereby allow for a less complex fabrication process, a more economical design may be obtained while still achieving the benefit of increased strength. In fabricating joists with HSS chords and web members comprised of single angles, it was further observed that the potential economy gained may be somewhat offset by corresponding reductions in strength due to the resulting out-of-plane eccentricities produced by the web angles being offset at the joist panel points.

Overall, it appears the use of HSS in the chords can provide greater stability during erection, and to lesser but still significant degree throughout the loading history of a joist. Based on a 32LH06 unbraced joist subjected to a midspan concentrated load, a joist with an HSS compression chord could support a load of approximately four times that of an equivalent standard joist with double angle chords. Although most likely less economical, it was shown that a joist with double HSS compression chords can provide approximately seven times the capacity of an equivalent joist with a double angle chord. It is anticipated that additional in-depth research aimed at the more efficient use of HSS in the chords could result in significant benefits to the open web steel joist industry.

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Appendices

A. Angles Design

SPAN = 57'-0"
TOP CORD (2) = 2.5" x 2.5" x 0.212" w/ 1" GAP
BOTTOM CORD (2) = 2.0" x 2.0" x 0.216" w/ 1" GAP
W2 = 1.5" x 1.5" x .109 DOUBLE ANGLE
W3 = 2.0" x 2.0" x 0.187" CRIMPED
W4 = 1.25" x 1.25" x 0.109" CRIMPED
W5 = 1.75" x 1.75" x 0.155" CRIMPED
W6 = 1.0" x 1.0" x 0.109" STRAIGHT
W7 = 1.5" x 1.5" x 0.155" CRIMPED
W8 = 1.0" x 1.0" x 0.109" STRAIGHT
W9 = 1.5" x 1.5" x 0.155" CRIMPED
W10 = 1.0" x 1.0" x 0.109" STRAIGHT
W11 = 1.5" x 1.5" x 0.155" CRIMPED
V1 = 1.0" x 1.0" x 0.109" STRAIGHT
V2 = 1.0" x 1.0" x 0.109" STRAIGHT
WEIGHT = ? lb

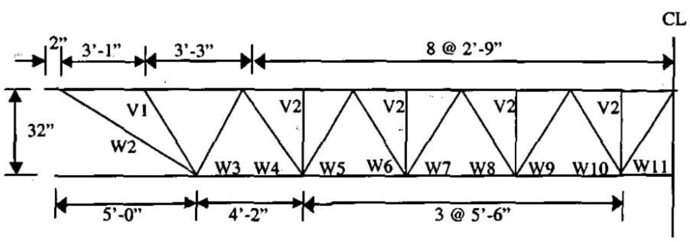


Figure 14: Geometric Properties of the Angles Design (Schwarz, 2002)

B. HSS Design

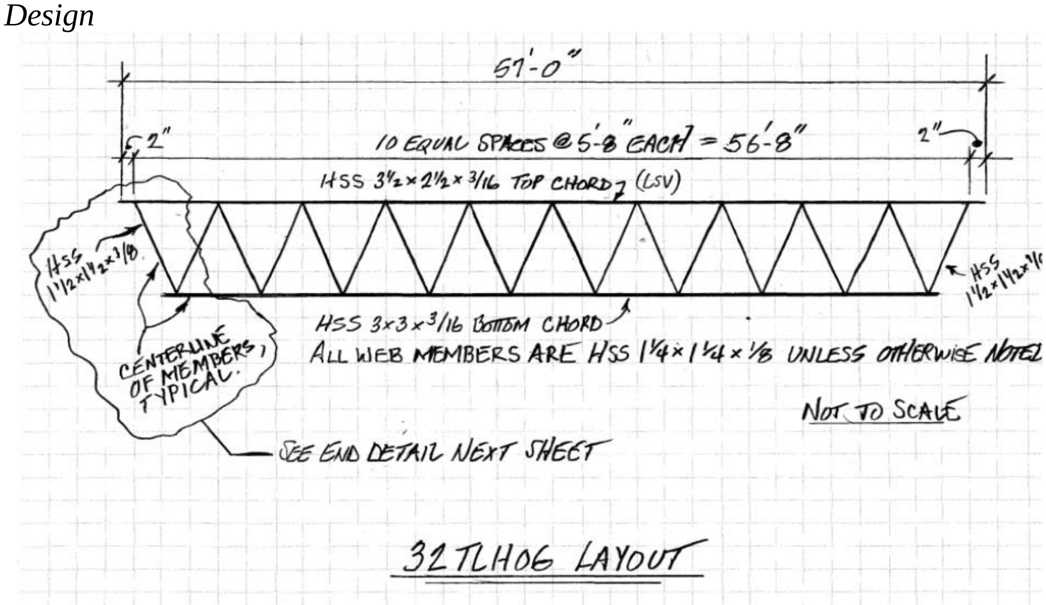
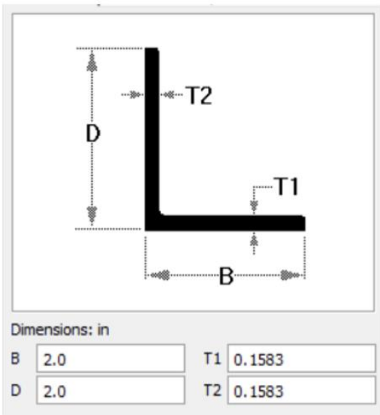
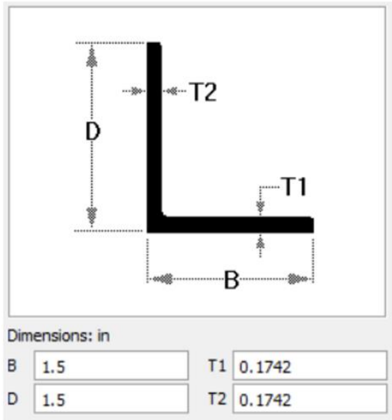


Figure 15: Geometric Properties of the HSS Design (Armbrust, 2020)

C. Hybrid 1 Design



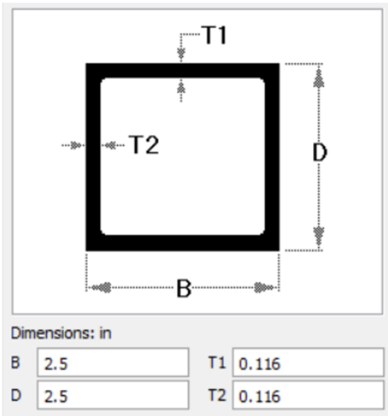
End web section



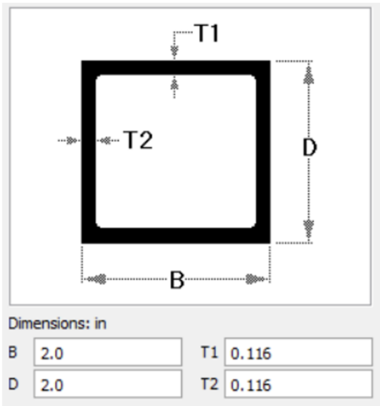
All other web sections

Figure 16: Web member changes to the HSS Design for the Hybrid 1 Design

D. Hybrid 2 Design



Top Chord dimensions for 1 of 2 identical double HSS sections spaced @ 1”



Bottom Chord dimensions for 1 of 2 identical double HSS sections spaced @ 1”

Figure 17: Chord member changes to the Angles Design for the Hybrid 2 Design