Task Group Report on:

High Strength Steel

Prepared by the AISC Committee on Specifications
Ad Hoc Task Group on High Strength Steel

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<u>**Ad Hoc Task Group on High Strength Steel**</u>

Charge

The AISC ad hoc task group on high strength steels will provide guidance to the AISC Specification committee and its Task Committees on how to encourage innovation in the application of high strength steel to construction; specifically, by ensuring AISC specifications are not an unreasonable impediment to adoption and that clarity exists for the process of introducing new steels to the Specification whenever possible, while maintaining the same standards of safety as currently provided by the Specification.

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In addition a large number of additional AISC TC members and guests contributed at the in-person meetings as guests of the TG. Their contributions to this report and efforts are acknowledged and thanked. In particular, Ryoichi Kanno of Nippon Steel and Nancy Baddoo of SCI contributed significantly to the effort. In addition, students Evan Brannon and Waleon Lama contributed to the production of this report and their contributions are gratefully acknowledged.

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EXECUTIVE SUMMARY

Historically, materials innovation is the key driver in civil structures. The progression from masonry and timber to iron and steel is at the core of the Industrial Revolution and created the basis for the infrastructure of modern civilization. Today, as a myriad of new challenges are being faced (resilience, sustainability, robustness, efficiency, more) materials innovation continues. Advances occur even in the low-cost materials that are employed at scale in civil structures such as higher strength and higher performance in concrete and in timber. High strength structural steel, with yield stresses in excess of 65 ksi (450 MPa) and as high as 145 ksi (960 MPa) can be produced and have been employed in buildings world-wide. The ability to tailor the steel in a particular portion of a building to exactly the properties desired now exists. Success in bridges, in automobiles, and other applications readily point to a future where high strength structural steel provides the next generation of strong, resilient, sustainable steel building applications.

As surveyed and reported herein manufacturers are able and willing to supply high strength structural steel to the U.S. market, but do not currently see high/sustainable demand. U.S. structural engineers are bullish on the adoption of high strength steel, but lack of information on price and availability hinders their progress. Success with high strength steel in U.S. bridges has been slow to translate to the building market. Japanese standards, European standards, and new Chinese standards all provide broader and updated provisions for high strength steel than current AISC specifications. AISC specifications have essentially only been developed for steels with $F_y \leq 65$ ksi (450 MPa) and steels that exhibit material characteristics in terms of tensile stress-strain shape similar to mild steel: yield plateau, high elongation, specific tensile-to-yield ratio etc. Adoption of new materials in AISC specifications is slow and does not follow a specific standard, thus stretching out timelines for application of high strength steel and creating uncertainties in final success that inhibit investment and innovation. Additional uncertainties exist around best practices in fabrication, how to best optimize the use of the more expensive high strength shapes, and how to develop successful connections and complete systems.

A comprehensive series of recommendations are provided to address these challenges. We must establish specific performance targets for high strength structural steel in the U.S. from greater than 70 ksi (480 MPa) up to 145 ksi (960 MPa). Using the excellence of the structural designers that participate in the AISC committees we must create the vision, fill-in the details, disseminate the ideas, and lobby broadly for the specific application of high strength steel in modern building systems. Leveraging AISC's dissemination vehicles (Manual, web, MSC, etc.) we must educate engineers on high strength structural steel and create a clear path for information on availability. Working with AISC's fabricators we must establish best practices for fabrication of high strength structural steels. Leveraging the AISC specification committees we need to connect with the other standards-writing bodies around the world, develop new language in our Specification to ease new material adoption, and work across the Specification to improve existing provisions to accommodate higher strength steels in members and connections. Finally, we need to be proactive in establishing research partnerships between academia and industry as well as public and private funding agencies to perform the fundamental research to (a) prove out the high strength steel vision established by the designers, (b) develop new ideas that expand what is possible for steel building systems utilizing high strength steels, and (c) provide the evidence necessary for safely expanding the AISC specifications to accommodate high strength steel.

1 INTRODUCTION

Progress in structural engineering is tied to progress in materials. Engineers leverage new material properties as they are made available and expand what they can provide in their design. The market follows making the new materials more affordable as their use increases and scale is realized. This report explores the possibilities for encouraging innovation in the application of High Strength Structural Steels (HS3s) in U.S. building construction; specifically, by ensuring AISC specifications are not an unreasonable impediment to adoption, and that clarity exists for the process of introducing new steels to the AISC Specification whenever possible, while maintaining the same standards of safety as currently provided.

Steel has benefitted enormously thanks to the materials science and steel processing advances of the last 20+ years. Largely through processes working at the microstructural level, high strength low alloy (HSLA) steels have been improved and entirely new grades of steel have been developed with yield strengths as much as 5× higher than conventional mild steels and tensile elongations greater than 10% and in some cases upwards of 20 or 30% at these high yields. Concomitantly other properties have also been improved including weldability and fracture toughness. Today many of these high strength structural steels, first developed for other markets and applications are finding their way to the construction market – and examples such as 150 North Riverside building completed in 2017, as provided in Figure 1, are beginning to increase.

Figure 1 150 North Riverside, New York, NY structure uses high strength structural steel (HS3) of 70 ksi (485 MPa), completed in 2017 (Finnigan et al. 2015)

High strength steel enjoys applications across a number of markets. For example, a concentrated effort in the automotive market has been greatly expanding available steels as shown in Figure 2. This latest generation of sheet steels are known as Advanced High-Strength Steels (AHSS). Keeler et al. (2017) identify 43 different AHSS grades that have been produced and used since 2002 with yield stress, F_y, as high as 1250 MPa [181 ksi] and ultimate stress, F_u, as high as 1900 MPa [276 ksi]. Some next generation sheet steels even have modestly improved (up to 11% higher) elastic modulus (Fuchs 2013), though this remains somewhat controversial.

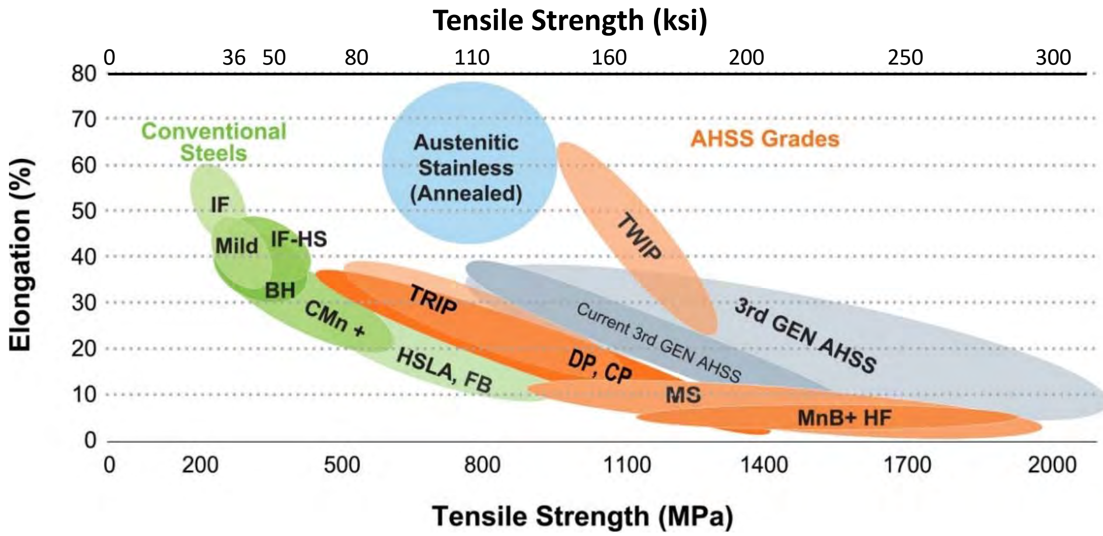


Figure 2 Mechanical performance of available steels (adapted from Keeler et al. 2017, complete abbreviations list in Keeler et al. 2017, selected include HSLA=High Strength Low Alloy, TRIP=Transformation Induced Plasticity, DP=Dual Phase, MS=Martensitic, TWIP=Twinning Induced Plasticity, CP=Complex Phase) Not shown are QST=Quenched and Self Tempering steels which are popular for high-strength structural steel.

While AHSS steels provide a glimpse of future potential for high strength structural steel (i.e. HS3) today a number of high performance HSLA-based steels are already on the market, or already exist as proprietary produced steels with similar properties for use in in bars, plates, tubes, and rolled shapes. It is no longer optimal, nor productive, for civil building construction to remain fixated on such a limited pallet of mild steel grades.

With new steels comes new opportunities, but also new challenges. Traditional civil construction has primarily used one type and grade of steel throughout a structure and relied on shape (or amount) of this material to optimize and achieve efficiency. Figure 2 and successful practice with high strength steel applications suggests the possibility of a complementary approach – where the steel material is also a design choice. Key members in a structure may use different grades of steel to meet a particular need. This approach is now common in automotive production, and has seen limited use in bridge construction, but is a relatively new thought for building construction outside of seismic applications. The engineer/builder that can understand and leverage this new steel material design space can potentially provide higher efficiency/higher performance than traditional construction.

1.1 Task Group Efforts

A task group (TG) was assembled by AISC in the Fall of 2017 under the following charge:

> "The AISC ad hoc task group on high strength steels will provide guidance to the AISC Specification committee and its Task Committees on how to encourage innovation in the application of high strength steel to construction; specifically, by ensuring AISC specifications are not an unreasonable impediment to adoption and that clarity exists for the process of introducing new steels to the Specification whenever possible, while maintaining the same standards of safety as currently provided by the Specification." TG Charge

The TG met and developed a list of possible activities. These activities were summarized as possible work assignments and then prioritized by the TG as reported in Appendix 1. The top 11 activities in priority order (high to low) were:

- Solicit manufacturers and fabricators w.r.t. high strength steels that may require AISC specification modifications for adoption
- Solicit designers regarding need and interest in high strength steel
- Determine explicit (and implicit when possible) F_y and related material (weldability, ductility, strain hardening etc.) limits in AISC 360
- Whether explicit or implicit summarize the role of F_u/F_y, elongation, n, other parameters inherent in AISC Specification provisions
- Given a hypothetical F_y=1000 MPa (145 ksi) steel (with other additional properties known) provide working outline of what steps would be required to introduce such a steel to AISC 360.
- Solicit, compile, and review completed research and standards that would justify expansion of F_y limits for particular steels that are not covered in AISC 360.
- Specifically solicit recent international research on RHS/CHS/Tube and box sections up to 1000MPa (145 ksi) and provide summary of findings to relevant TCs
- Provide recommendations on how to model physical imperfections and residual stresses in higher strength steels
- Provide short term recommendations to the AISC Specification Committee on how to incorporate higher strength steels into the Specification
- Provide long term recommendations to the AISC Specification Committee on how to incorporate higher strength steels into the Specification

This report largely provides the results of the TG's efforts on these tasks. This report is not comprehensive in nature. The TG aimed to provide progress on each of its identified work items and then identify needed activities for the AISC Specification Main Committee, Task Committees, or the AISC Committee on Research.

Given the broad and active discussion in the TG, and efforts from the TG members, the overall interest and need in advancing steel materials in construction is high. Given this fact one of the TG's recommendations is that AISC consider how to continue this conversation/effort in some centralized form as opposed to the more silo-ed efforts of the TCs.

1.2 Terminology: "High Strength" and "High Strength Structural Steel"

The TG charge to address "high strength" steel has a certain ambiguity that must be addressed. The AHSS steels of Figure 2 certainly provide the upper end of what is currently possible, but such extremely new and advanced (costly) steels are not necessary to be considered "high strength" in building construction. In fact, as Table 1 makes clear, the notion of "high strength" is strongly sector dependent. Essentially any steel with F_y greater than 450 MPa (65 ksi) is likely to be considered "high strength" in civil building construction. (This may be contrast with bridges where "high strength" today implies 550 MPa (80 ksi) or higher.)

In the context of this TG report "High strength" steel is further specified as "High Strength Structural Steel" or HS3 – this implies a broad class of steels that provide F_y greater than 450 MPa (65 ksi) and, consistent with the notion of structural steel, delivered thickness at 3/16 in. (4.7mm) or greater – HSLA steels that have undergone careful processing are still at this time the dominant HS3 for rolled shapes. However, the first and second generation AHSS steels (DP, CP, TRIP, TWIP, MS) are of potential interest.

Many of these HS3 have been developed for longer than the 20 year timeframe alluded to in the introduction, and in the literature may appear under other names, e.g., the High Performance Steel (HPS) that was developed for bridge construction starting in 1992 provides an excellent case study for the adoption of higher strength steel in civil construction, as detailed in Appendix 3. In these steels "performance" is highlighted as they sought improved weldability and fatigue performance, not just higher strength. Nonetheless, the effort resulted in specific 345 MPa (50 ksi), 483 MPa (70 ksi) and 690 MPa (100 ksi) F_y grades that are used in domestic bridge construction today, often in hybrid shapes built up from plate.

In recent civil engineering literature the term high strength is often connected to research on 690 MPa (100 ksi) F_y steel and ultra-high strength to 960 MPa (140 ksi) and higher F_y steel. It is worth noting for the U.S. engineer that in the global literature it is common to refer to 690 steel or 960 steel where the F_y is implicitly in MPa – and this may occur without the use of high strength, advanced high strength or ultra-high strength modifiers. The use of all such terms have to be understood in their context – and knowing only the yield stress does not provide the engineer with much information on the actual performance of the steel. As a result, Keeler et al. (2017) always designate by process (DP/TRIP), yield (F_y), and tensile (F_u) a practice that should likely become more commonplace in AISC Specifications in time.

1.3 Acronyms

Inevitably a large number of acronyms were utilized in this report. An attempt is made here to provide a list of the employed acronyms for the readers convenience.

Steel Terms

AHSS	= Advanced High Strength Steel
CHS	= Circular Hollow Section
CP	= Complex Phase
CVN	= Charpy V-Notch test
DP	= Dual Phase
H3S	= High Strength Structural Steel
HPS	= High Performance Steel
HSLA	= High Strength Low Alloy
HSS	= Hollow Structural Section or High Strength Steel. Use HS3 instead
LTB	= Lateral-torsional Bucking
MS	= Martensitic
QST	= Quenched and Self Tempering
RHS	= Rectangular Hollow Section
SBHS	= Japanese High Performance Steel
TC	= Task Committee (of AISC)
TG	= Task Group (of AISC)
TMCP	= Thermo-Mechanical Control Processes

TRIP	= Transformation Induced Plasticity
TWIP	= Twinning Induced Plasticity

<u>Organizations</u>

AASHTO	= American Association of State Highway and Transportation Officials
ACI	= American Concrete Institute
AISC	= American Institute of Steel Construction
AISI	= American Iron and Steel Institute
ASTM	= American Society of Testing Materials
AWS	= American Welding Society
BOMA	= Building Owners and Managers Association
DOE	= Department of Energy
EC3	= Eurocode 3 for the Design of Steel Structures
HUD	= Housing and Urban Development
IABSE	= International Association of Bridge and Structural Engineers
JIS	= Japanese National Standard
MBMA	= Metal Building Manufactures Association
MCA	= Metal Construction Association
MILT	= Japanse Ministry of Land, Infrastructure, Transport, and Tourism
MKA	= Magnusson Klemencic Associates
MSC	= Modern Steel Construction
NIST	= National Institute of Standards and Testing
NSF	= National Science Foundation
RCSC	= Research Council on Structural Connections
SCI	= Steel Construction Institute (UK)
SDI	= Steel Deck Institute
SJI	= Steel Joist Institute

Table 1 Sector dependent definition of "high strength" steel and factors influence use (European Union Research Fund for Coal and Steel, 2011)

High Strength Steels Definition																
v02/06/2016																
		Specified minimum yield strength (MPa)													**Factors affecting use of high strength steels**	
Sector	**e.g Product form**	**235**	**275**	**300**	**355**	**400**	**420**	**450/460**	**500**	**550**	**690**	**890**	**960**	**1100+**	**Advantages**	**Factors limiting use**
Buildings	Pofiles/ open sections	Lo	Lo	Med	Med	Hi	Hi	Hi							Long spans, aesthetics	Deflection, buckling, strain dissipation
Buildings/ stadia	SHS/ RHS/ CHS closed sections		Lo	Lo	Med	Med	Hi	Hi	Hi	Hi					Long spans, weight saving	Welding procedures, welding €
Fondations, quay walls	Sheet piles, piles	Lo	Lo	Med	Med	Hi	Hi	Hi							Limited	Deflection, corrosion allowance
Bridges: road (small and medium spans)	Fabricated girders/ profiles	Lo	Lo	Med	Med	Hi	Hi	Hi							Longer spans	Fatigue (welds), Toughness
Bridges: road (Long span)	Fabricated girders				Lo	Lo	Med	Med	Hi	Hi	Hi				Longer spans, installation weight	Fatigue (welds), Toughness
Bridges: rail	Fabricated girders / profiles	Lo	Med	Med	Hi										Limited (fatigue dominates)	Fatigue (welds)
Pipelines: Onshore	Seam, HFI, spiral welded pipe				Lo	Lo	Med	Med	Hi	Hi	Hi				Higher pressures, welding €	Crack arrestability, uniform elongation
Pipelines: Offshore trunk lines	Seam welded pipe				Lo	Lo	Med	Med	Hi	Hi					Deeper water, launch weight	Buckling, fatigue, compressive collapse
Pipelines: Flow lines / risers	Seamless pipe				Lo	Lo	Med	Med	Med	Hi	Hi				Higher longitudinal service load	Buckling, fatigue welded joints
Pipelines: Tubing / casing	Seamless pipe				Lo	Lo	Med	Med	Med	Med	Med	Hi		Hi	Higher service loads	stress corrosion, buckling
Pressure Vessels	Welded plate		Lo	Lo	Med	Med	Med	Hi	Hi	Hi	Hi				Higher pressures	Toughness
Storage tanks/silos	Welded plate	Lo	Med	Med	Hi	Hi	Hi	Hi							Limited (hydrostatic pressure)	Lowest cost option dominates
Fixed offshore rigs	Welded plate				Med	Med	Med	Med	Hi	Hi					Transport, installation	Fatigue (welds), Corrosion fatigue
Mobile offshore rigs	Welded plate				Lo	Lo	Med	Med	Med	Hi	Hi	Hi			Reduced weight, ease of installation	fatigue, Corrosion fatigue
Bulk container ships	Welded plate	Lo	Med	Med	Med	Hi	Hi								Limited	Fatigue (welds), deflection, crack arrest
Military ships, fast ferries	Welded plate			Lo	Med	Med	Hi	Hi							Higher speed, lower centre of gravity	Distortion
Windtowers/ onshore	Welded plate / profiles/ cold formed tubes		Lo	Lo	Med	Med	Hi	Hi	Hi	Hi					welding €, installation	Fatigue (welds), elastic stability
Windtowers/ offshore	Welded plate		Lo	Lo	Med	Med	Hi	Hi	Hi	Hi					welding €, transport weight	Fatigue (welds), elastic stability
Mobile cranes	Welded tubulars / profiles/ HRC						Lo	Lo	Med	Med	Hi	Hi		Hi	Reduced weight, longer spans	Toughness, fatigue, weld strength
Quarrying & mining	Wear plates/ HRC					Lo	Lo	Med	Hi	Hi	Hi				Reduced weight, wear resistance	Fatigue (welds), Toughness
Yellow Goods	Welded plate/ HRC		Lo	Lo	Med	Med	Med	Hi	Hi	Hi	Hi				Reduced weight, lower fuel cost	Weldability, forming
Notes:																
Lo	= Considered as low strength for the sector															
Med	= Considered as normal strength for the sector															
Hi	= Considered as high strength for the sector															

2 HIGH STRENGTH STEEL FUNDAMENTALS AND REVIEW OF TECHNICAL LITERATURE

2.1 Microstructure, chemistry, and process

Changes in microstructure, chemistry, and processes have led to the creation of structural steels that have yield strength (far) in excess of 65 ksi (450 MPa) and still maintain desirable elongation, ductility, fracture toughness, and weldability. A variety of variations are utilized to achieve this end with high strength low-alloy (HSLA) steels and careful thermo-mechanical control processes (TMCP), e.g. see Ouchi (2001). Specifically quenched and self-tempered (QST) HSLA grades are especially popular for thick plate and rolled shapes (and also enjoy popularity as steel reinforcing bar).

A large variety of additional methods, which may be characterized as Advanced High Strength Steels (AHSS) also exist as candidates for higher strength and ductility, as summarized in Figure 2. Keeler et al. (2017) characterizes the major difference between these AHSS steels and traditional HSLA steels:

> "The principal difference between conventional HSLA steels and AHSS is their microstructure. Conventional HSLA steels are single-phase ferritic steels with a potential for some pearlite in C-Mn steels. AHSS are primarily steels with a multiphase microstructure containing one or more phases other than ferrite, pearlite, or cementite - for example martensite, bainite, austenite, and/or retained austenite in quantities sufficient to produce unique mechanical properties. Some types of AHSS have a higher strain hardening capacity resulting in a strength-ductility balance superior to conventional steels. Other types have ultra-high yield and tensile strengths and show a bake hardening behavior. "
>
> "All AHSS are produced by controlling the chemistry and cooling rate from the austenite or austenite plus ferrite phase, either on the runout table of the hot mill (for hot-rolled products) or in the cooling section of the continuous annealing furnace (continuously annealed or hot-dip coated products). Research has provided chemical and processing combinations that have created many additional grades and improved properties within each type of AHSS. " (Keeler et al. 2017)

It is important to understand the process that creates the high strength steel under study, not just the $\sigma - \epsilon$ behavior as weldability, fatigue, fracture, influence of high and cold temperature, etc. can all be different even for the same F_y, n, etc. More important for the future is not to treat these additional characteristics as unknowns or concerns, but rather to determine what the desired performance should be for a particular steel in a given structural building application.

2.2 Stress-Strain Relationship and Parameters Beyond F_y

The stress-strain relationship for most high strength structural steels (HS3) presents certain challenges to traditional steel design specifications. Consider the basic differences as illustrated in Figure 3. In addition, an example of an HS3 stress-strain curve of a rolled shape in current production was provided to the TG and is also given in Figure 4, and AHSS curves are provided in Figure 5.

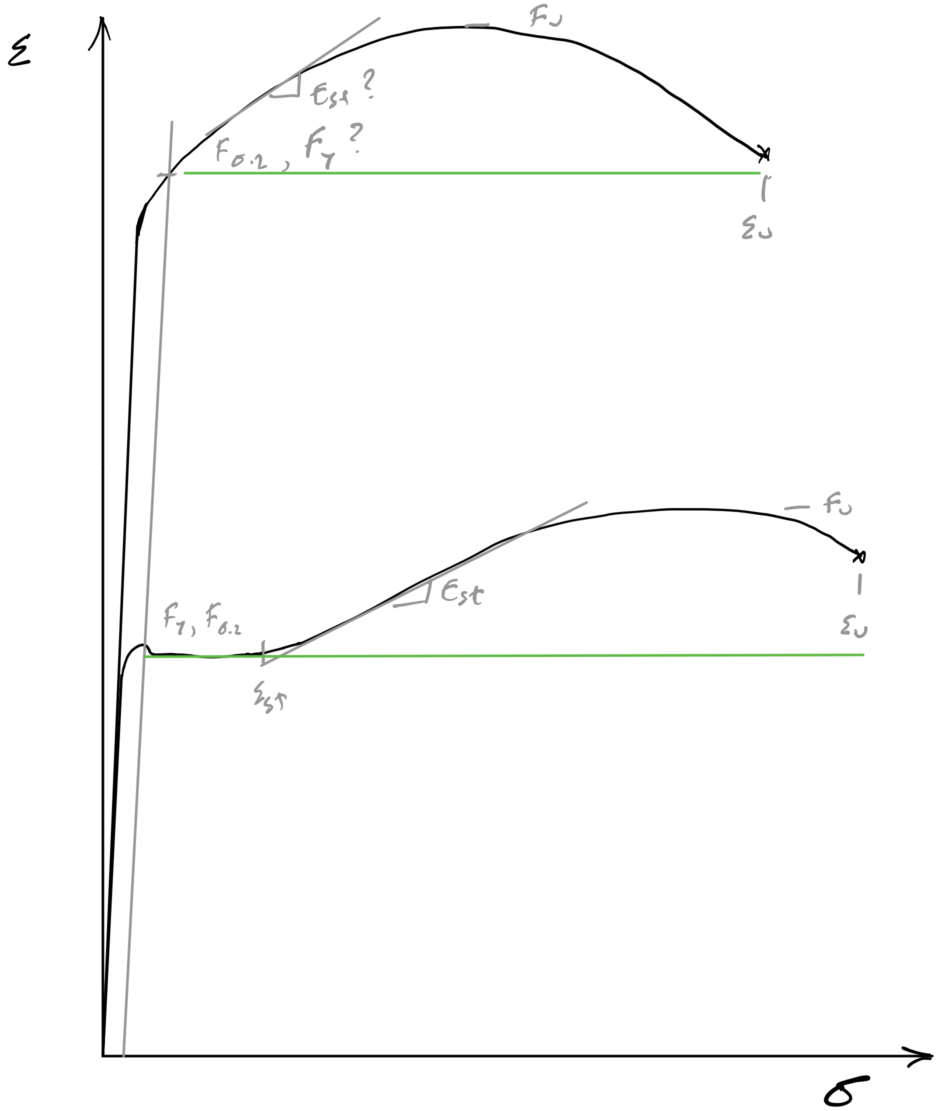


Figure 3 Illustration of typical mild steel and properties and that of a high strength steel

E_o - Initial modulus of HS3 is typically the same as mild steel or in some reported cases slightly higher; depending on the process, anisotropy or orthotropy is possible, but typically not pronounced

Proportional limit – mild steel has a proportional limit very close to 1.0 while stainless steels and aluminum may have proportional limits as low as 0.5. HS3 proportional limit is process dependent, but typically close to 1.0 – depending on the definition of F_y.

Yield plateau – some HS3 have yield plateaus, e.g. many HSLA steels can exhibit a yield plateau; however most do not have a defined yield plateau. The yield plateau is implicitly assumed in plastic design and many traditional structural steel limit states.

F_y – the yield stress for mild steel is not strongly dependent on the method of definition, 0.2% offset, and other methods give similar results – this is not the case for most HS3. Extension of the 0.2% offset is popular, but beyond being a convenience has no real connection to the application of F_y in AISC Specification limit states or other formulas. Careful definition of F_y is needed for successful application of current prediction methods to HS3.

ϵ_{st} – the strain at which strain hardening initiates is an important implicit limit for mild steel. At this strain the material will exhibit increased strength, but typically this strain is also associated with the limit of useful strain for a serviceable structure and width-to-thickness limits, and expected connection strains are tied in part to ϵ_{st}. Common HS3 do not have a definite ϵ_{st} and as a result may correlate poorly to certain implicit limits in the *Specification.* In general, HS3 strain hardens earlier than mild steel, this may be acceptable behavior for strength limit states, but may require careful monitoring for seismic and capacity-based design applications.

E_{st} vs. n – Mild steel has typically been able to characterize strain hardening with a single slope, while other metals tend to use single or multi-stage Ramberg-Osgood parameters, e.g., n. Defining Est in an HS3 may be a difficult exercise and treating HS3 like other metals in terms of n parameters may be more fruitful. From a *Specification* standpoint any ultimate limit states which are deformation dependent need to be addressed in this light.

F_u,ϵ_u – Defining the ultimate stress, strain is the same for mild and HS3 steel, however it is worth noting that engineering stress-strain definitions are used and differences in true stress, strain may be accentuated for HS3. No obvious *Specification* change is needed for F_u-based limit states.

Elastic-plastic idealization – The *Specification* relies on an idealized E-P material definition in many of its limit states (e.g., M_p) . Determining how to make this E-P fit for HS3 steel is not as obvious as it may first seem. Should F_y be fit such that the energy ignored in HS3 is the same for mild steel? Should F_y be fit such that energy absorbed is close to zero up to a target (e.g. ϵ_{st}) strain? ..or other possibilities.

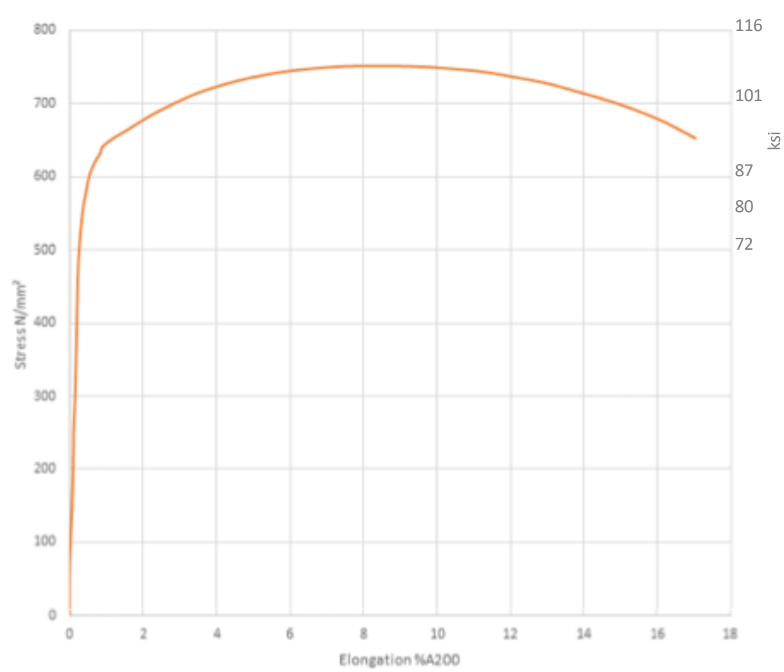


Figure 4 Nominal 550 MPa (Gr. 80) rolled shape in current production

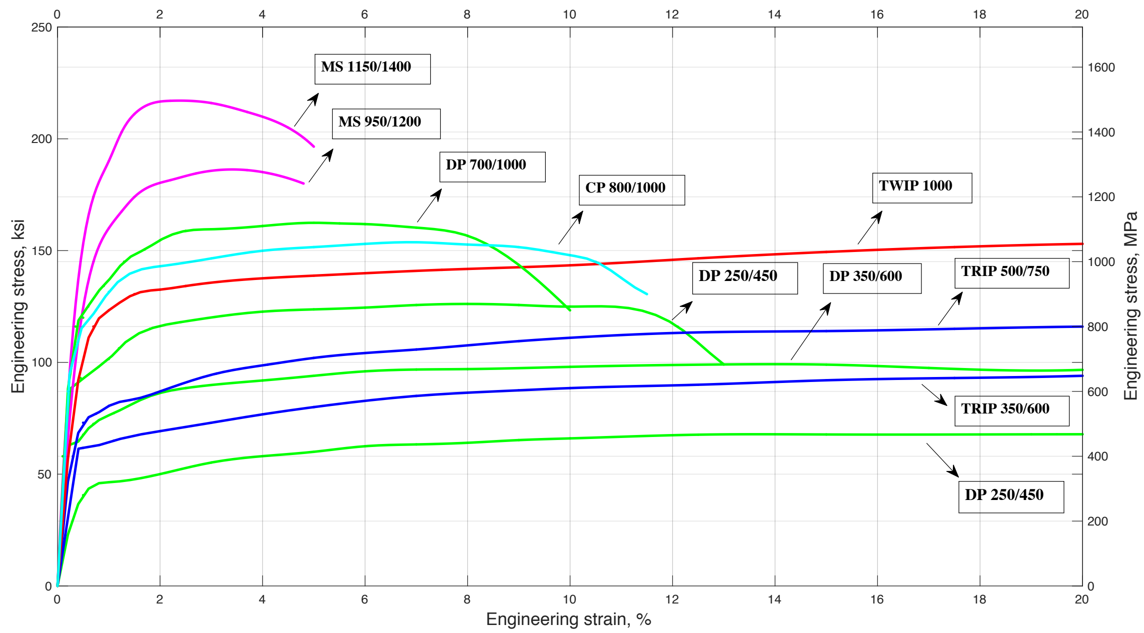


Figure 5 Tensile stress-strain curves across steels (adapted from information provided in Keeler et al. 2017)

2.3 Residual Stresses in HS3 Shapes

Residual stresses influence the strength of steel columns, particularly in the critical inelastic regime where most gravity columns in a building reside. Residual stresses due to differential cooling would be expected to be a function primarily of thermal properties and section thickness and shape – not yield stress. Despite this, common practice (for a variety of reasons) is to express residual stresses as a function of yield stress and thus inherent in this assumption is that residual stresses should increase linearly with F_y. In general this has not been found to be the case, though the data is limited. Some production processes undoubtedly influence residual stresses, but for the most part HS3 appears to have residual stresses that are similar in magnitude to mild steel – and as a result smaller when normalized with respect to F_y.

Early work on a QST F_y=690 MPa (100 ksi) steel in Australia for I- and box-sections formed from plate measured maximum residual stresses of ~150 MPa, or $0.2F_y$ (Rasmussen and Hancock 1995). Spooerenberg et al. (2013) examined residual stresses in QST F_y=450 MPa (65 ksi) rolled structural shapes as summarized in Figure 6. They also provided parabolic residual stress models consistent with Eurocode that were in good general agreement with measured data.

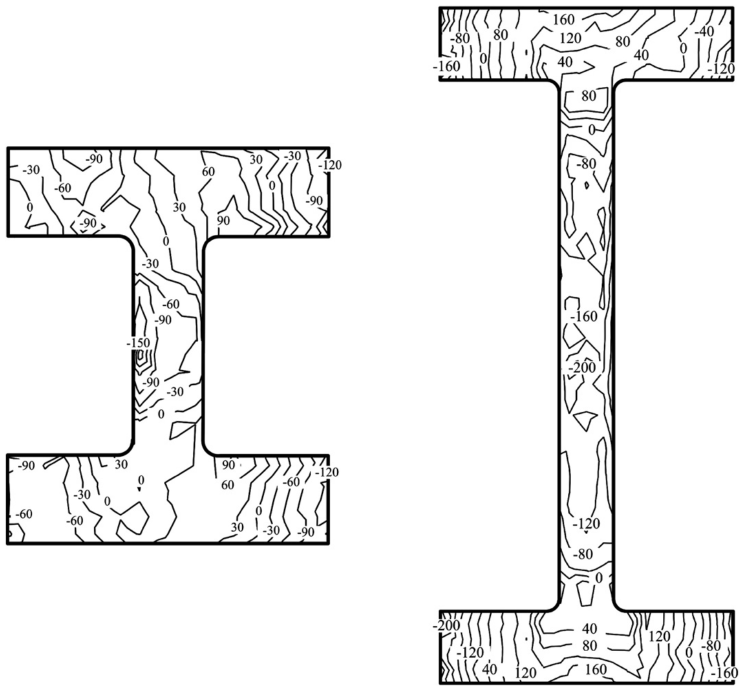


Figure 6 Residual stress in QST Rolled Shapes Fy=450 MPa (65 ksi) nominal from Spoorenberg (2013)
(Note |max| residual stress ~ $0.3F_y$ in the HD 400 x 1202 shape on the left, ~ $0.5\ F_y$ in the HL920x1377 on the right)

In China, Shi and his colleagues have studied residual stresses on high and ultra-high strength steel angles and welded plate sections include I- and box-sections, see for example Figure 7 (Ban et al. 2013). In the summary provided in Shi et al. (2018) residual stress models are recommended for HS3 sections – and it is summarized that HS3 sections have similar distributions of residual stress as non-HS3 sections – and that the magnitude even in welded sections can be significantly below the yield stress (though still fairly high with values as high as 300MPa measured near welds.).

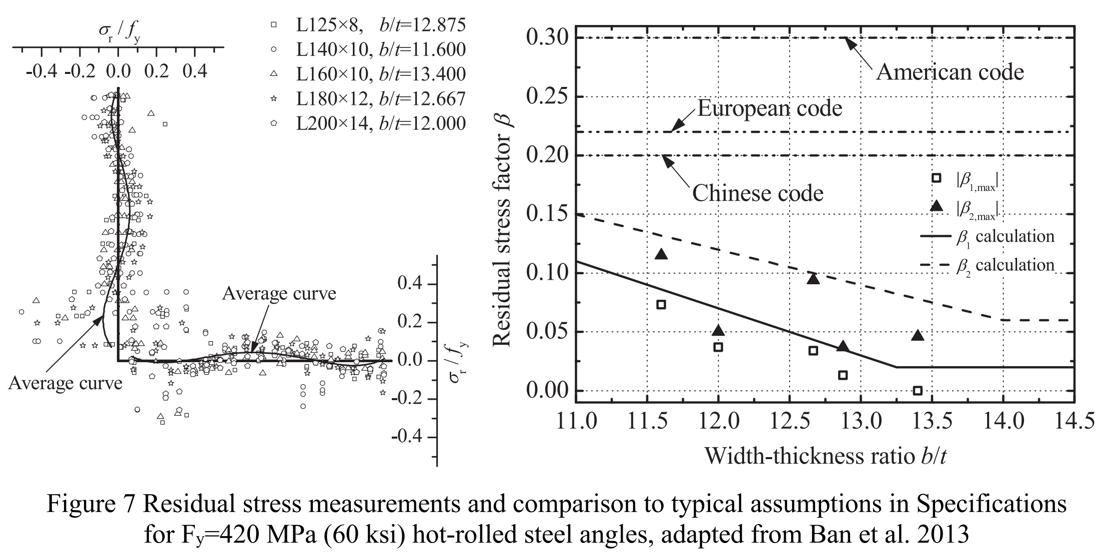


Figure 7 Residual stress measurements and comparison to typical assumptions in Specifications for F_y=420 MPa (60 ksi) hot-rolled steel angles, adapted from Ban et al. 2013

The primary conclusion that one may draw from these studies on residual stresses in HS3 sections is that simplified rules for residual stresses, normalized to F_y based on results in mild steel, should not be used in studies to explore or develop the predicted strength of HS3 sections. In fact, residual stresses are likely to be lower when normalized to Fy than in mild steel – and as a result strength as predicted against empirical column or beam curves may be inefficiently predicted for HS3.

2.4 Geometric imperfections in HS3 shapes

For shapes produced to ASTM A6 using HS3 there is no expectation that the geometric imperfections will be different from those produced from mild steel. However, as an aside, it is worth noting that in cold-formed high-strength sheet steels that the elevated F_y has led to greater challenges with springback and an initial period of adjustment for producers, but in the end traditional tolerances were met. Ban and Shi (2018) summarized 80 HS3 column tests (F_y from 460-960 MPa) on welded box- and I-sections and while many of the specimens had imperfections less than L/1000 more than ¼ did not and several of the specimens approach L/100. Production may have to be monitored with care to meet traditional geometric imperfections standards in HS3 shapes.

2.5 Inelastic Rotation in High Strength Steel Beams F_y>65ksi?

For AISC 360-16 in cases where plastic hinging is anticipated and rotation capacity is required to redistribute moment (for example, B3.3 and Appendix 1.3.2a), F_y is not permitted to exceed 65 ksi (450 MPa). This limit has its origin in the research that established plastic analysis and design, where high-strength steel members (F_y > 65 ksi (450MPa)) were not thoroughly studied.

During the code cycle developing AISC 360-16, a proposal was submitted to increase the current limit of 65 ksi (450 MPa) on F_y (in cases where plastic hinging is anticipated and rotation capacity is required to redistribute moment) to 70 ksi. In support of this proposal, test data for small-scale bend tests were submitted as evidence of material ductility, but the Task Committee decided that evidence of large-scale member-level ductility (rotation capacity) would be necessary to justify the proposed increase in F_y.

The ability of an I-shaped flexural member to the reach its plastic moment, M_p, and sustain it through some plastic rotation depends on a variety of parameters, including: material – yield stress, yield-to-tensile ratio (YR), strain at tensile stress (e_u), strain hardening modulus (E_{st}); section – flange and web local slenderness ratios ($b_f/2t_f$ and h/t_w); and member – lateral slenderness (L_b/r_y), moment gradient. Rotation capacity of flexural members is commonly defined as $R = (\theta_u/\theta_p - 1)$. Where, θ_p is the rotation corresponding to M_p assuming linear elastic behavior up to that point, and θ_u is the rotation at which the moment resistance drops back to M_p after exceeding M_p.

Important differences in material properties between conventional-strength and high-strength steels have been documented in prior research, for example in studies of HPS70W (Barth et al 2000) and HSLA-80 (Green et al 2002). Representative properties reported in these studies are summarized in Table 2 and Table 3, respectively. As yield stress increases, YR increases, and E_{st} and displacement ductility (e_u/e_y) decrease. These material-level trends have been identified to potentially negatively impact member-level ductility (i.e. rotation capacity).

Table 2 *Material properties for A36, A572 Gr. 50 and HPS70W steels (adapted from Barth et al 2000).*

	F_y (ksi)	YR	E_{st} (ksi)
A36	36	0.55-0.62	650
A572 Gr. 50	50	0.77	<720
HPS70W	70	0.83	<280
	F_y (MPa)	YR	E_{st} (MPa)
A36	250	0.55-0.62	4500
A572 Gr. 50	345	0.77	<4900
HPS70W	480	0.83	<1930

Table 3 *Material properties for A36 and HSLA-80 steels (adapted from Green et al 2002)*

	F_{yn} (ksi)	F_{ym} (ksi)	F_{um} (ksi)	YR	E_{st} (ksi)	e_y	e_u	e_u/e_y
A36	36	41	64	0.64	492	0.0034	0.1909	57.0
HSLA-80	80	86	97	0.91	209	0.0049	0.0810	16.7
	F_{yn} (MPa)	F_{ym} (MPa)	F_{um} (MPa)	YR	E_{st} (MPa)	e_y	e_u	e_u/e_y
A36	250	283	441	0.64	3390	0.0034	0.1909	57.0
HSLA-80	550	593	669	0.91	1440	0.0049	0.0810	16.7

For F_y and F_u subscript *n* refers to nominal, *m* to measured

Based on plate buckling theory and experimental data, AISC 360-16 defines compact section slenderness ratios (λ_p) for flexural members (Table B4.1b) that are nominally intended to ensure the development of M_p with R > 3. (More stringent slenderness limits are specified in the AISC *Seismic Provisions*, AISC 341-16, where higher rotation capacity is needed to develop stable cyclic response under earthquake demands.) In AISC 360-16, the flange λ_p is:

$$\lambda_{pf} = \frac{b_f}{2t_f} \leq 0.38\sqrt{\frac{E}{Fy}}$$

and the web λ_p is:

$$\lambda_{pw} = \frac{h}{t_w} \leq 3.76\sqrt{\frac{E}{Fy}}$$

Following early studies on plastic design that considered steel with F_y = 36 ksi, Adams et al (1965) focused on extending plastic design to steel with F_y = 50 ksi. Subsequently, Iyengar et al (1976) concluded that "A572 Gr. 65 steel exhibits mechanical properties in the inelastic region similar to those of structural carbon steel," hence the standard compactness criteria and unbraced length limits were permitted up to F_y = 65 ksi. Although these limits are a function of F_y, their applicability to high-strength steel (F_y > 65 ksi) is not confirmed, and prior research has indicated that these compact section criterion may not be appropriate for some high-strength steel grades (e.g. Green et al 2002).

McDermott (1969) conducted one of the earlies studies of high-strength flexural members with focus on plastic behavior and ductility of A514 steel. Several compact sections per the current criteria did achieve M_p and obtain $R > 3$, but this was not a consistent result. In flexural tests of HSLA-80 I-sections, Green et al (2002) also obtained R > 3 in some cases, but not on a consistent basis, and in these specimens the webs were significantly below the compact section limit. Dexter et al (2002) obtained R > 3 in a flexural test of a girder with F_y = 70 ksi, but the girder was singly-symmetric with a large compression flange designed to locate the neutral axis at the top of the web, simulating a positive moment region in a composite section. Hartnagel (2003) tested two compact HPS70W girders where R > 3 was not achieved. Yakel (2002) also found that compact plate girders with F_y = 70 ksi did not develop R > 3. Prior research has also demonstrated that closely-spaced braces are required to prevent lateral-torsional buckling and enable stable inelastic flexural response at plastic hinges, and – all else equal – a moment gradient improves rotation capacity compared to uniform moment.

Results from these and other prior studies are summarized in Appendix 5. Focusing only on the beams tests under uniform bending which fail in local buckling, i.e., adequately braced against LTB the rotational capacity is plotted against the normalized element slenderness in Figure 8. The element local buckling criteria λ_{pf} and λ_{pw} do not guarantee a minimum R of 3 independent of the steel material grade. See the recent report on local buckling limits for further discussion (AISC Local Buckling Ad Hoc Report 2019). In addition, high strength steels tend to have lower rotational capacity than mild steel. Further work on the Table B4.1 limits and their application to plastic design may be warranted, but regardless, higher strength steels cannot be immediately grouped in with existing mild steels. Additional work is needed to justify the application of higher strength steels – and one can expect that the details of the mechanical $\sigma - \epsilon$ response, not just F_y, are important.

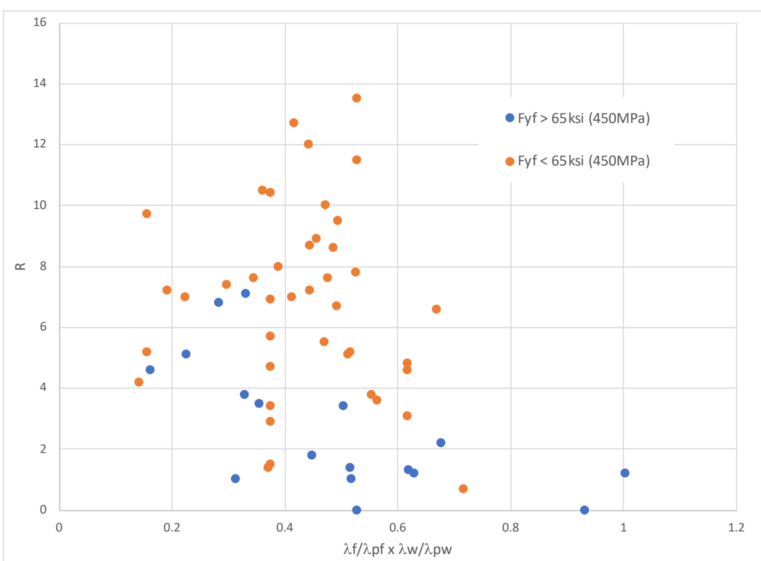


Figure 8 Rotation capacity for uniform bending tests on I-shaped sections

2.6 Additional Review of High Strength Steel (HS3) literature

In the development of this report the TG also performed an initial, general, review of high strength structural steel. The summary which is provided in this section is not comprehensive, but does provide a record of materials reviewed by the TG.

2.6.1 HS3 Flexural Applications

Beg and Hladnik (1996) tested ten HS3 I-beams (Fy = 101.5ksi (700 MPa)) with varying flange slenderness to analyze the local stability and supplemeneted their work with nonlinear analysis to study web-flange interaction in local buckling. Their results suggested that for a welded I-beam in bending, a slenderness limit of 40ϵ could be obtained to separate a slender and semi-compact cross-section, which is considerably more generous than the current slenderness limit of 30ϵ found in the EC3 code for flanges. Their work also showed that the strength and ductility of I-beams in bending is significantly impacted by the stability interaction between the flange and web of the member and revised slenderness limit expressions were proposed.

Bradford and Liu (2017) performed FE analysis to investigate lateral-torsional buckling of high strength steel beams when subjected to a uniform bending moment, as well as the effects of residual stress on the HS3 beam. Previous tests had shown the impact of residual stresses on inelastic LTB (Bradford and Liu 2016). Their modeling showed that higher strength steel beams performed better against code predictions than conventional steel at intermediate slenderness ranges where residual stresses were influential (at low and high slenderness the differences were small). While the absolute magnitude of residual stresses is approximately equal in high strength steel shapes, the relative magnitude (normalized to F_y) is lower.

Lee et al (2013) investigated the effect of flange slenderness on the flexural strength and rotation capacity of I-shaped beams fabricated from 116ksi (800MPa) steel. Lee found that the high strength steel specimens were more than adequate for strength but lacked the magnitude of rotation capacity needed for plastic design. The lack of a defined yield plateau in the material stress-strain response was cited as a potential cause of the reduced rotation capacity. They also examined the use of welded transverse stiffeners to the tension flange of their specimens. The welds suffered brittle fracture under load – indicative of a need for further work with the specific high strength steel studied.

Shi et al (2018) performed flexural tests on I-beams fabricated from 67 ksi (690 MPa) and 129 ksi (890 MPa) steel subjected to a uniform moment. Their study examined noncompact and slender (Class 3 and Class 4 shapes). Comparing their results with current codes, Shi et al found that AISC 360-10 overestimated the ultimate moment for non-compact specimens and underestimated it for slender specimens. In addition, Eurocode 3, GB 50017-2013, AIJ LSD2010 and AS 4100-1998 all gave conservative predictions.

2.6.2 HS3 End plates

Girão Coelho and Bijlaard (2006) and Girão Coelho and Bijlaard (2007) studied end-plate connections made with high-strength steel (67ksi, 100ksi, and 139ksi) to analyze the nonlinear behavior of this type of connection. The study resulted in validating the Eurocode 3 approach for higher-strength steels, and perhaps more importantly in the demonstration that the rotation capacity of specimens using high-strength steel can satisfy high-deformation demands, surpassing what is expected of mild steel grades with the proper material selection and design.

2.6.3 HS3 Bolted Connection

Wang et al (2018) performed an experimental investigation on bearing-type bolted connections with two bolts located perpendicular to the direction of loading to observe the failure modes of tearout and splitting. Twenty-four bolts were fabricated and tested against steel of grades 80 ksi (550 MPa) and 100ksi (690 MPa). Wang et al. found that the end distance plays a major role in determining the ultimate load and deformation capacity, while the bolt spacing and steel grade has minimal effect on the failure mode.

2.6.4 HS3 Columns

Shi et al. (2012) tested 8 high strength steel I-section columns built up from plate of 100 ksi (690 MPa) and 139 ksi (960 MPa) steel. Comparing their tests with typical column curve predictions they found the tests to generally be lower. However, this was found to be due to abnormally high geometric impefections (as high as L/100 in several cases). They validated an FEA model to their testing and then showed that for columns with L/1000 imperfections common buckling curves are conservative. They specifically cite the AISC 360 curve as overly conservative and recommend using the highest buckling curve ("a") if employing Eurocode.

In 2016, Li et al. studied the experimental response of 100ksi (690MPa) steel columns subject to axial compression, using the slenderness ratio (ranging from 30 to 70) as a variable. As anticipated, all specimens failed in a global buckling mode; a couple of specimens, that were heat-straightened during fabrication, showed remarkably lowered capacity. The authors conclude that current codes, including the Chinese code, the Eurocodes, and AISC360-10 underestimate the capacity of 100ksi columns. Using the "a" buckling curves in BS50017 and Eurocode 3 appears to provide the best agreement with the test data.

2.6.5 HS3 Frames

Hu et al. (2017) recently performed cyclic tests on six full-scale single-bay two-story frames. Four frames consisted of varying combinations of high strength (67ksi) and conventional strength (50ksi) steels, one used just ultra-high-strength steel (129 ksi), and one just conventional steel (50ksi). The goal of the research was to observe the response of high strength structural frames under cyclic loading, including the energy dissipation and plastic deformation capacities of each combination. In their research, the most recurring phenomena were local buckling at the base of the columns, partial fracture of the beam-column connections, and twisting of the beam due to torsion. Hu et al. found that, at the end of the tests, the frames with high strength columns and high strength beams, as well as those with high strength columns and conventional strength beams had the largest deformation capacities, even though the frame with conventional strength columns and beams had the largest plastic deformation at the end of the first cycle of the 4% drift ratio loading phase. This was due to the lack of strength degradation found in the high strength members. In addition, the frames with high strength steel members ultimately had greater cumulative energy dissipation. In fact, the frame with high strength columns and beams had 34% more cumulative energy dissipation than the frame with high strength columns and conventional beams. However, as expected, the high strength steel frames had less plastic displacement prior to the onset of severe buckling of the column bases than the conventional steel frame. Overall, the ultimate drift ratio capacity satisfied the requirements for highly ductile behavior for every frame with high strength steel columns.

2.6.6 HS3 seismic and cyclic testing

The majority of research on high strength steels has focused on monotonic response. However, cyclic response and the behavior under seismic action is of interest and current thinking has been summarized by Li et al. (2015). High strength structural steels have good potential for removing mass, a key element in seismic response. The potential for mixing grades and performance of steel in seismic systems was noted by Li et al. and examples provided. High strength structural steel may be used for critical force controlled elements while high performance steels (high elongation, toughness, etc.) may be selected in deformation controlled elements and/or special energy dissipating systems. In Shi et al. (2012) the authors tested 17 specimens of high strength steel subjected to various cyclic loading patterns using 67 ksi (460MPa) steel material. They concluded that these specimens experienced ductile failure after being subject to different cyclic loading patterns, which indicates that high strength steel can achieve good ductility and significant energy dissipation capacity.

2.6.7 HS3 and Steel-concrete composites

Researchers also focused on the behavior of steel-concrete composite systems, to investigate the benefits of using high-strength steels paired with high-strength concrete materials. Research on composite beams has been performed in Australia by a group of researchers led by Bradford (Ban and Bradford, 2013; Ban and Bradford, 2014; Ban et al., 2016). These papers focused on the flexural strength and rotational capacity of composite beams with high strength materials (both steel and concrete), with steel grades ranging from 34 ksi (235 MPa) and 50ksi (345MPa), through 67ksi (460MPa), 73ksi (500MPa), 80ksi (550MPa), 90ksi (620MPa) all the way to 139ksi (960MPa). In 2013, Ban and Bradford used finite element analysis to perform a parametric analysis on the flexural behavior of composite beams with high-strength steel, validated upon several sets of experimental results, focusing on the influence of the yield strength of steel and of the degree of shear connection. In a work along similar lines, Ban and Bradford (2016) used once again finite element analysis to study the flexural capacity of composite beams made with high-strength steel sections, focusing on the effects of the yield strength of steel and of the residual stresses. The researchers propose a reduction factor to be used in the adaptation of currently used rigid-plastic analysis approaches, from which the prediction of the flexural strength of composite beams with high-strength steels can be made accurate. Ban et al. (2016) found that the available rotation capacity is contingent upon the location of the neutral axis within the composite beam system. The smaller the distance of the neutral axis, the greater the rotation capacity of the composite beam. Another important factor that impacts the available rotation is the span-to-depth ratio. In order to determine the most influential parameters that affected available rotation capacity in composite beams, Ban et al. created a 3D finite element model, and then validated the model against over 1300 beams analyzed by independent researchers. They used this model to construct an empirical equation that was consistent in predicting available rotation capacity of high strength composite beams.

The flexural behavior of composite beams made of high-strength steel was also recently investigated by Shamass and Cashell (2017). The researchers focused on finite element analysis of composite beams accounting for geometric and material non-linearity, as well as for the non-linearity caused by the shear connectors. A parametric study was conducted focusing on the influence of material properties, shear connection, distribution of shear connectors, and beam geometry. Validation was performed on several sets of experimental results on composite beams, using both mild steel and high-strength steel. The materials considered in the parametric study were 67ksi and 100ksi steel, and 6ksi, 7.5ksi, and 9ksi concrete, with shear connection ratios ranging from 0.5 to 1.7. The researchers developed reduction factors that can be used with Eurocode design approaches, resulting in safe and economical designs.

2.6.8 HS3 and Fire

There is limited research on the performance of high strength steel under extremely high temperatures, so Chen et al (2006) performed tests using both steady and transient-state test methods to better understand the mechanical properties of high strength steel in the presence of elevated temperatures. Results showed that mild strength steels and high strength steels had similar retention factors and elastic modulus for temperatures in the range of 22 - 540 degrees Celsius, but not for temperatures greater than 540 degrees Celsius. Measured thermal elongation were less than predicted by international standards (including the U.S.) and EC3-1-2 codes. Measured yield strength retention factors were higher than predicted by international standards. Winful et al (2017) conducted a parametric study of the flexural behavior of high strength square and rectangular hollow section columns at temperatures up to 800 degrees Celsius. Overall, their analysis determined that the Eurocode buckling curves capably predict the buckling resistance for the high strength HS3 columns, but tend to be overly conservative.

3 POTENTIAL FOR HIGH STRENGTH STRUCTURAL STEEL (HS3)

3.1 Survey and Perspective of Steel Designers

The AISC Adhoc Task Group on High-Strength Steel performed a survey in February 2018. There were 27 participants and participants were allowed to skip questions. Altogether there were five questions followed by comments or suggestions by the survey takers. Follow up interviews were conducted by those who provided their names and contact information. A full report of the survey results including charts for each question and a list of companies that participated is found in Appendix 2. The following is a summary of the survey findings:

- 100% of the participants believe they could use AHSS (material with yield strength ranging from 65 ksi (460 MPa) to 145 ksi (1000 MPa) or higher) in their practice and project design.
- The top responses for which shapes/materials should be offered in high strength steel were: W-shapes (96% agreed), plate (74% agreed) and HSS (70% agreed). The rest of the shapes/materials that fell below 50% were: pipes (44% agreed), bars or rods (30% agreed), headed studs (11% agreed), angles (11% agreed), joists (7% agreed), and channels (4% agreed).
- Trusses and composite columns are more desirable when it comes to applications of advanced strength steel (Fy > 65 ksi). Their desirability was 63% and 56% respectively. One person commented that thick plates for fabrication of large cross-section, built-up column or brace sections would be more desirable. The other participate commented that any cases where self-weight is a big part of design loads and deflections can be managed.
- High rise buildings (> 20 floors), arenas, conventions centers, and stadiums had the highest responses to structures that could use high strength steel (Fy > 65 ksi) with a response of 87% or higher. Theaters/culture centers, bridges, industrial facilities, medium rise buildings (7-20 floors) had an average of 67% response and hospitals, transmission structures, schools/education facilities, low rise buildings (1-6 floors) had an average response of 21%. One participate commented that "special structures could use high strength steel" and the second commenter wrote that "any structures where self-weight is a big part of load and deflections can be managed could use high strength steel".
- The following three categories: "cost information", "market availability" and "production application" (steel grade and its intended applications) were checked off by 20 participates regarding to what information is missing in order to use high strength steel. Whereas the categories "ASTM specifications" and "limitations on use" tied with 14 responses. Two people did mark off "other". One believes "testing showing adequate deformation capacity in sub assemblages" (e.g., beam/column joints, brace to column connections, etc.) is missing in order to use high strength steel. Another comment was that "technical knowledge in mass distribution" is missing in order to use high strength steel.

Questions 6 and 7 were open end questions where people who took the survey were able to give their opinions. See below for the question and responses. Not every participant answered questions 6 and 7.

Q6. What other problems do you see that prevent high strength steel to be used on your projects (other than those listed in the above questions)?

Responses:

1. Weldability, ductility, toughness concerns would all need to be addressed.
2. Material availability is the biggest question. Most projects are driven by schedule. Availability and cost information is necessity.
3. Reliable welding technologies; higher strength bolted connections
4. In projects where stiffness drives the design (high-rise columns, long span trusses, etc.), steel elements are chosen based on area, not strength
5. Wherever strength is needed and serviceability is manageable
6. No improvement in elastic modulus
7. Education to steel fabricators, erectors, and special inspectors. It's not enough for engineers to be able to specify them. If others involved with the product are resistant to its use, it will adversely impact its adoption in the built environment

Q7. Please provide any other comments or suggestions that you believe is pertinent to the Task Committee efforts.

Responses:

1. I think there is a lack of clear understanding of the availability of high strength, thick plate material in the market. There are producers who say they can produce plate products well beyond the current ASTMs, but the Structural Engineer needs reliable information on material properties, availability, cost, and other design considerations at an early stage, in order to be able to implement.
2. For seismic regions (say Seismic Categories C and D) it should only be used in elements intended to remain elastically or practically elastically.
3. As more and more high strength material is used, must further emphasize serviceability limit states in design.
4. The committee should take a look at the NIST and Pankow foundation reports on developing criteria for HS reinforcing bars in concrete.
5. We are successfully implementing grade 65 steel for columns, braces and truss members in a high-rise residential project. Switching from grade 50 to 65 saved the owner 100 tons of material.
6. Need the research and design guides, cost info and availability
7. Most structural steel design is limited by deflection concerns. AHSS will not address this.

3.2 Survey Follow-Up Interview Results

Task group members interviewed survey takers who provided their name and contact information. Notes from these interviews follow:

- **Report from Bill Scott**

  Socrates Ioannides (Structural Affiliates International) commented that high strength steel would be favorable in heavy columns and trusses he uses to transfer loads. He is not so favorable on use of high strength steel in hybrid girders as deflection generally controls long span girders. 75 ksi steel is available internationally so now's the time to address this issue.

  Pre-heat and post-heat for welding are an issue when going to high strength material. Bill reports that he uses blankets after welding high strength steel to slow cooling. The issues involved with welding high strength steel will have to work their way through AWS.

  AASHTO is pushing the strength limits for plate—he would look to them for provisions for high strength steel. The question is how high in strength can we go before the Specification won't work. We will need to do testing to determine where that limit is.

- **Report from Tom Poulos**

  Carol Drucker (Drucker Zajdel Structural Engineers) replied that she has been able to successfully push 70 ksi steel in NYC projects. One of the plusses to higher strength material is that it could reduce shop labor when high strength electrodes are used. Grade 65 plate has been used; however, thickness of this material is currently limited to 1-1/4". Bolts, welds and plate material will all need to be stronger to match higher strength members. Bolt strength beyond A490 will be needed, but the 200 ksi bolts currently allowed by the Specification are not allowed to be used outside. It would make sense to apply a set of rules for different grades of material. For example, on a project you could use different diameter bolts for low, med. and high strength materials. The goal is to keep confusion to a minimum and avoid a situation where a coupon test is needed for inspection.

  Additional feedback received from co-workers: High strength material would be beneficial for trusses where strength controls the design and to provide a shallower design. High strength is also desirable for building columns where KL/r controls the design. Composite columns are another application that would benefit using high strength materials. With higher strength material you have the potential to span more floors. Tom mentioned that he had experience with 100 ksi lifting bars on a recent project.

- **Report from Ramon Gilsanz**

  Joe Mugford (Gilsanz Murray Steficek): Transfer girders, gravity columns are the two main applications. Pipe piles that presently have proprietary high strength rebar could now have high strength jackets instead of rejected high strength pipe from petrochemical applications. Curious about the shape of the post yield curve. He saw similarities with the success of high strength concrete. Concerned about the cost differential with more common steels

Walterio Lopez (Rutherford + Chekene): Thinks is a very good idea and if the steel producers can do this for a reasonable price it would be ideal. He thinks it would be easier to comply with the strong column weak beam. Everyone wants to be more resilient; hence, less damage and as a result more strength will helps us achieve that. As ASCE7-16 is more codified for nonlinear analysis it requires bigger Pu loads that translate into bigger shapes more heavy shapes; Higher strength will reduce the shape weight and also help with welding requirements.

R. Hamburger (Simpson Gumpertz & Heger): I understood that you think that high strength steel has a good future mostly in axially loaded members. Your structures are mostly deflection controlled so the higher strength does not help. So, what you want is steel with a higher Young's Modulus.

3.3 Survey and Perspective of Steel Manufacturers

The AISC Adhoc Task Group on High-Strength Steel preformed a survey of steel manufacturers supplying structural steel to the U.S. market in May of 2018. There were 5 participants and participants were allowed to skip questions. Altogether there were eight questions followed by comments or suggestions by the survey takers. A full report of the survey results including charts for each question is found in Appendix 4. The following is a summary of the survey findings:

- Manufacturers have an interest in bringing specific F_y = 450 MPa (65ksi), 480 MPa (70ksi), 620 MPa (90ksi) , 690 MPa (100ksi), and 760 MPa (110ksi) products to market today, and in the future there is wider interest in 690 MPa (100 ksi) products.
- Even in this small sample manufacturers already, or are likely to produce, high strength steel coil, plate, bar, tubes (hollows), and rolled shapes for structural application.
- Engineers/contractors should contact the producer directly for pricing and availability. (For materials that go through distribution (service centers) this is an obvious challenge.) That said, the rough outlines of the IABSE (2005) article with respect to preliminary pricing are (hesitantly) confirmed. [This question alluded to the survey results from the previous section where cost and availability were identified as primary issues by design engineers – an issue which is complex to readily resolve.]
- The necessity to have ASTM standards is not seen as an impediment to adoption by the surveyed manufacturers. [TG discussions prior to the survey indicated hesitation with always requiring an ASTM standard, but the surveyed manufacturers were sanguine about this need.]
- Composite (concrete) design up to 100ksi steel is seen as an obvious extension, well aligned with advances in steel for rebar, and an important part of the future of that form of construction.

Follow-up with one manufacturer supplying in the U.S. civil construction market provides an anecdotal summary of the current state in construction: For rolled shapes they are topping out at F_y = 460MPa (65 ksi) at this time, aligned with existing ASTM standards, and do not see enough demand yet to go higher. For tubes/hollows they have production capacity and interest in F_y = 690 MPa (100 ksi) and higher, but not seeing demand and are not a large player in the particular market. For plate they have production and interest up to F_y = 960 MPa (145 ksi), but currently not supplying into the construction market at that yield.

High strength steel exists for structural steel applications and manufacturers can supply with steels that are far above the performance commonly used today in civil building construction; however, producers have found construction market customers typically only have vague requirements and it can make it difficult to align need with available HS3 materials.

4 CURRENT USE OF HIGH STRENGTH STRUCTURAL STEEL

As Table 1 indicates all major sectors of steel application include potential "high strength" steel options. To that end, an exhaustive examination of the current use of high strength structural steel is impossible and not aligned with the TG objectives. However, specific domestic and international experiences in the use of high strength steel with potential lessons for U.S./domestic adoption are of interest and summarized herein.

4.1 Domestic

4.1.1 High Strength Steel in Buildings

A broad summary of the role of steel strength and production technology in the development of steel buildings in the U.S. is provided in Finnigan et al. (2015). More specifically, Finnigan et al. trace the introduction of HSLA steels with higher strength, but reduced ductility, and then the advantages realized by TMCP processes in creating improved grain structures, and the extensions that provide todays in-line produced QST steels with high strength and ductility. Recent U.S. building examples employing high strength steels include One World Trade Center completed in New York in 2014, 150 North Riverside employing QST with F_y=65 ksi (450 MPa) and F_y=70 ksi (485 MPa) completed in New York in 2017 and 217 W. 57th St. using QST with F_y=485 MPa (70 ksi) completed in Chicago in 2020. In addition to the projects listed in Finnigan (2015) the TG is also aware of the following additional projects using F_y=70 ksi (485 MPa) in the project: One Manhattan West, New York, NY; and 425 Park Ave, New York, NY. In addition looking more broadly at North America projects utilizing F_y=70 ksi (485 MPa) steel include: The Britt Residences, Toronto, ON; Bay Adelaide East, Toronto, ON; Brookfield Place, Calgary, AB; 707 Fifth, Calgary, AB; Telus Sky, Calgary, AB; and 400 W Georgia, Vancouver, BC.

4.1.2 High Performance Steel in Bridges

In the U.S. the development and application of High Performance Steel (HPS) can be traced back to a consortium in 1992 that developed new high performance (particularly weldability and toughness) steel grades 50W, 70W and 100W which have F_y = 345 MPa (50 ksi), 480 MPa (70 ksi), and 690 MPa (100 ksi), respectively, and the 1998 Transportation Equity Act for the 21st Century which encouraged and partially funded the use of the new grades in highway bridges. As detailed in Appendix 3 these steels met a particular need and it has been a long process from development to use. Today the 70W (480 MPa – 70 ksi) grade has reasonably widespread adoption in plate steel built-up bridge girders, and hybrid girders with 50W webs and 70W flanges are recognized for their benefits and also applied in designs.

The HPS story (Appendix 3) provides a number of potential lessons for successful adoption of innovation in civil construction for steel:

- Focus on specific grades and specific needs
- Utilize industry, engineering, government, and academic research in the development
- Work directly with early adopters to try to increase chance of success
- Create incentives that reward early adopters

- Don't expect 1:1 replacement while higher F_y provides significant benefits – re-engineering is required to reap the rewards. This is true for the shapes/members employed, the connections, and the fabrication.

4.1.3 Aside: High Strength Steel Rebar

Over the years, there has been an increasing demand for the use of higher-strength materials in reinforced concrete design. The potential benefits of using high-strength steel and concrete include reductions in material quantities, reinforcement congestion, and construction schedule. At present time, reinforcing bars Grade 80 (550) with specified yield strengths of 80 ksi (550 MPa) or greater are regarded as high-strength reinforcement.

In the early 1900s, reinforcing bars Grade 33 (230), 40 (280), and 50 (350) were standard. In the 1950s, Grades 60 (420) and 75 (520) became available and were adopted into the 1963 edition of the ACI Code, *Building Code Requirements for Structural Concrete* (ACI 318). In 1971, the ACI Code placed a limit of 80 ksi (550 MPa) on the design yield strength of longitudinal reinforcement for non-seismic applications and 60 ksi (420 MPa) for seismic applications. These limits were maintained in the 2014 edition of the ACI Code with the exception of allowing Grade 100 (690) for confining reinforcement since the 2005 edition.

The most commonly referenced reinforcing bar specification in the United States is ASTM A615, *Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement*. However, ASTM A706, *Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement*, is usually specified for seismic applications or other special applications where weldability or ductility is important.

In 1968, ASTM A615 was introduced to replace ASTM A15, A408, A431, A432, and A305. The initial version of ASTM A615 covered bars in three different grades, 40 (280), 60 (420), and 75 (520). In contrast, ASTM A706 was introduced in 1974 and covered only Grade 60 (420) reinforcement. In 2009, both ASTM A615 and A706, added high-strength reinforcement Grade 80 (550).

The 2019 edition of ACI 318 (ACI 318-19) introduced special requirements on mechanical properties and bar deformations of high-strength steel bars Grades 80 (550) and 100 (690) to allow their use in seismic-force-resisting systems (special moment frames and special structural walls). These requirements supplement those in ASTM specifications and address minimum uniform elongation and minimum base radius-to-height ratio of the deformation lugs for all grades of ASTM A706 reinforcement. Other special requirements specify a minimum tensile-to-yield strength ratio for all grades of ASTM A706 and A615 reinforcement.

ACI 318-19 allows the use of ASTM A706 reinforcement Grades 60 (420) and 80 (550) in special moment frames and up to Grade 100 (690) in special structural walls. All non-seismic applications of ACI 318-19 allow the use of Grade 100 (690) reinforcement with a few limitations.

4.1.4 Aside: Automotive steels

Keeler et al. (2017) provide a must-read for the application of high strength steels in automotive design. Consider the twin needs of increasing fuel efficiency and increasing crashworthiness. Fuel efficiency standards can be most easily met through mass reduction. Safety and crashworthiness goals can be best met from strategic use of materials, shapes, and structures.

Both lend themselves to the potential benefits of strategically using high strength steel in automotive bodies. For example in the modern automotive body provided in Figure 9 each color is a different steel. The red designates an AHSS steel with extremely high F_y – providing a safety region where side impact does not impinge into the driver while the purple region is a different AHSS steel with extremely high elongation and energy dissipation – providing a region where front impact can absorb energy and not transmit that energy to the occupants.

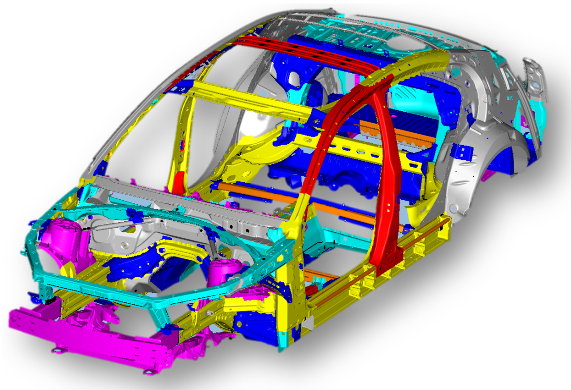


Figure 9 Body in white for a modern in-production automobile, each color is a different grade of sheet steel, red and purple are AHSS steels, individual stamped parts are joined by resistance welding and adhesives, Krupitzer (2014)

It is worth noting that these steels were developed directly with automotive customers with specific objectives on formability, joinery, and end performance. Keeler et al. (2017) delve deeply into issues around manufacturing and joinery that will be critical to successful steel fabrication in buildings with higher strength steels. Fabrication and weldability are all solvable problems, but don't expect current processes to work without modification. Even basic tooling in the shop has to evolve to handle working with higher strength steels. The use of the right steel, in the right amount, in the right shape, in the right location is the story of the application of high strength steels in automobiles and portends a level of design sophistication that will be new to building construction.

Today evidence exists that these steels are moving slowly into construction and infrastructure. For example, a sheet steel with F_y > 145 ksi (960 MPa) and favorable weathering properties is being used for piling in solar array construction and being considered for highway guard rails.

4.2 Tube Applications Domestic and Worldwide

Hollow structural shapes have led other structural shapes in early adoption of higher strength steel applications with 690 MPa (100 ksi) material available and codified in Europe and 960 MPa (140 ksi) material developed, available, and in the codification process. Due to this early adoption, they are summarized separately here.

In Europe, hollow structural sections, both circular hollow sections (CHS) and rectangular hollow sections (RHS), are used frequently in construction. (Note the common acronym for hollow structural shapes – HSS, has not been used here as HSS refers to High Strength Steel in much of the literature). CHS have commonly been used with minimum yield strengths up to 690 MPa (100 ksi), while RHS, used less commonly in Europe, are still typically specified as 345 MPa (50 ksi), but are sometimes found up to 450 MPa (65 ksi). In European construction, hollow sections are either left bare or filled with concrete and often the material used is heat treated after being cold-formed. High strength CSS/RHS in Europe are typically hot-formed.

Since the material is actually being produced there, Europe has done quite a bit of research on CHS/RHS sections and their connections. The research has shown that in Eurocode the connection strength to a section with a higher yield strength (> 345 MPa) is downgraded by a "material factor" to account for larger connection deformations. Additionally, the ratio of yield to tensile strength is usually capped as this can get quite high for high strength steels. The next edition of Eurocode 3 will contain rules for CHS/RHS up to 690 MPa (100 ksi).

Domestically, in addition to ASTM A500/A1085/A1065, CHS/RHS are also produced to an HSLA specification that meets a minimum yield strength of 550 – 760 MPa (80 – 110 ksi). This material is not typically available in service centers so mill quantities may apply making their current usefulness in construction limited. Also domestically, one manufacturer has recently assisted in the development of an ASTM standard for cold-formed welded high strength carbon steel, low-alloy RHS (ASTM A1112). This manufacturer is also producing CHS/RHS using a direct-form method and can achieve yield strengths exceeding 690 MPa (100 ksi). This product is currently being used for vehicles and trailers, ladders, booms and agricultural equipment. With the adoption of ASTM A1112, specification in construction projects should be easier as there is now an accepted standard building officials can refer to for high strength CHS/RHS.

4.3 Europe

Examples of applications of high strength steel in building and bridge construction in Europe are briefly summarized in SCI (2019) and Shi et al. (2018), and include:

> "Pont Citadelle in Strasbourg, FR, spans 180 m and includes extensive use of S460 [65 ksi] for the structural members (2017). The four primary roof trusses for the Friend's Arena, Stockholm, SE, used S460 [65 ksi] tubes for the top chord, S690 [100 ksi] and S900 [130 ksi] U sections for the bottom chord and diagonals made from S900 [130 ksi] rods. The trusses were 13% lighter and 15% cheaper than using S355 [50 ksi] (2012). Muiderberg railway bridge, NL, used S460 [65 ksi] box sections for the arches and main girders (2016). Northern Spire, UK, a 336 m long, 2 span cable stayed bridge, used S460 [65 ksi] for the lower flanges in the main deck girders (2018). " SCI (2019)
>
> and
>
> "The Sony Centre in Berlin, Germany, which uses S460 [65 ksi] and S690 [100 ksi] steel, the Rhine bridge at Dusseldorf-Ilverich, Germany, with S460 [65 ksi], the Millau Bridge in France, with S460 [65 ksi] steel, the Fast 48 Military Bridge in Sweden, with S960 [140 ksi] and S1100 [160 ksi] steels, the composite bridge near Ingolstadt, Germany, with 690 MPa [100 ksi] steel, and the Mittadalen hybrid bridge girder in Sweden, with 690 MPa [100 ksi] steel." Shi et al. 2018

A number of other examples exist, but the list is intended to provide clarity that 450 MPa (65 ksi) structural steels are in regular production and use, and higher 690 MPa (100 ksi) and much higher 960 MPa (140 ksi) are being employed in strategic projects.

4.4 Japan

Japan has about a 60-year history of using high strength steels in the construction market, where high strength steels are roughly defined as structural steels with nominal yield strengths larger than 400 MPa (58 ksi) or nominal maximum strengths larger than around 600 MPa (87 ksi). The application started first in bridges in the 1960s and then to buildings in the early 1990s (Kanno

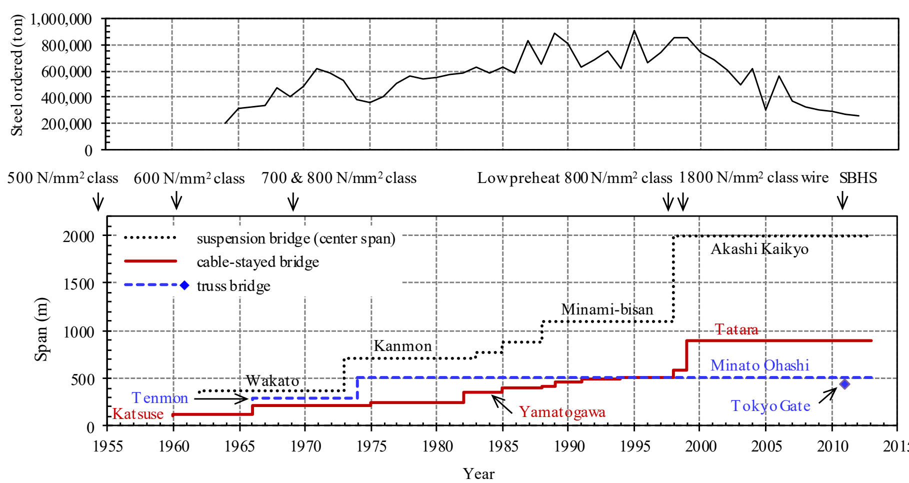
2016). At present a wide variety of high strength and performance steels are used in the construction market in Japan.

For bridges, Figure 10a shows timelines of the steel amounts ordered for bridge construction, the maximum spans of suspension, cable-stayed and truss bridges, and the major developments in steel materials (adapted from Kanno 2016). In addition to higher strength, advancements in toughness and weldability were sought, achieved, and applied in the Akashi Kaikyo Bridge and led to the formal development of a high-performance steel known as “SBHS”. SBHS with yield strengths of 400 MPa (58 ksi), 500 MPa (72 ksi) and 700 MPa (101 ksi) were developed and standardized in Japan in 2008.

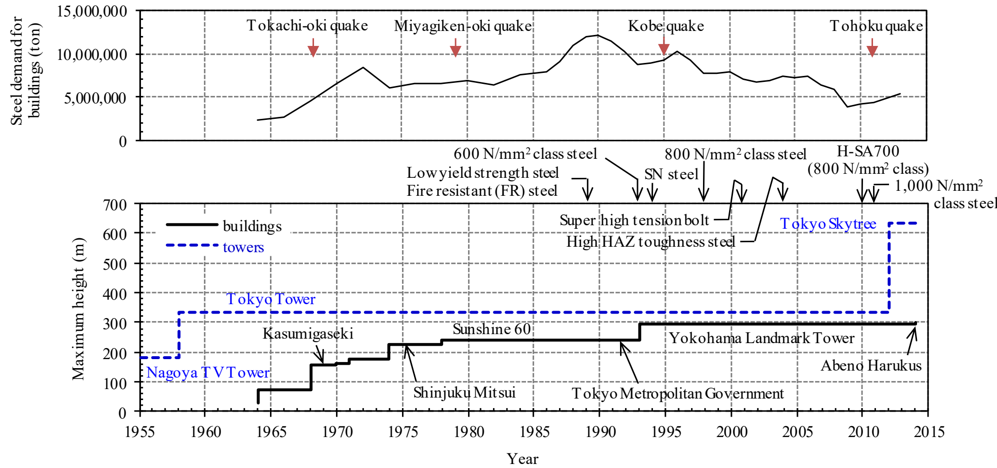
For buildings, Figure 10b shows timelines for the steel demand, and the maximum heights of Japanese buildings and towers (adapted from Kanno 2016, 2017). Noteworthy steel material developments and earthquakes are also indicated in the figure. Due to the high seismic activity in Japan, the application of high strength steels in building frames has lagged behind bridges (approximately 30 years!). Steel with a tensile strength of 600 MPa (F_y = 430 or 440 MPa) was first used in the Yokohama Landmark Tower in 1993 and then in the Keyence Headquarters in 1994. Furthermore, in 1998 steel with a tensile strength of 800 MPa (F_y = 620 MPa) was applied to the Kokura Station Building. Note that both of these high strength steels (600 and 800 MPa) differ from the steels used for bridges. They have special properties for achieving larger inelastic deformation capacity for seismic design such as low yield ratio and low yield strength variation. Around 2010, following the development of a passive control system typically paired with buckling restrained braces, steels with a tensile strength of 800 MPa (F_y = 700 MPa) and subsequently 1,000 MPa (F_y = 880 Mpa), which is the strongest steel ever to be used in a building, were put into use. These high strength steels did not meet the requirements previously set in Japan, since the building’s seismic resistance was secured largely by dampers such as buckling restrained braces and the similar devices.

The development of high strength steel can also be viewed through systematic improvement in steel strength for plate as Figure 11 provides (adapted from Kanno 2017). The application of high strength steel advanced in bridges and then later expanded to use in buildings. Steel with a tensile strength of 800 MPa was successfully applied in the 1960s in bridges because bridge design was primarily based on elastic design. The application of high strength steel in buildings was delayed due to a safety concern about earthquakes. After a change in design methodology for buildings from elastic to inelastic design in 1981, the requirements for structural steel for buildings were stipulated to increase the inelastic deformation capacity of members and frames. Following this, progress was rapidly made during the 1990s in the application of high strength steel with tensile strengths up to 800 MPa, which had different properties from those used in bridges. Interestingly, the maximum strength of the steel used in buildings surpassed that of bridges to 1,000 MPa in the early 2010s once a passive control system with dampers like buckling restrained braces became common. This is because inelastic deformation capability is mainly attained by dampers; thus the frame itself can be design to remain roughly elastic. In this situation, columns can enjoy a great benefit of using high strength steels by making their sizes small. As seen in the history in buildings, design methodology and frame system against earthquakes were major driving forces in buildings for the development and application of high strength steels.

Underpinning these advances in strength and performance are advances in steel production technologies (Homma 2014, Nishioka and Ichikawa 2012). Traditionally high strength steels were produced using techniques like quenching and temper treatment. As such, the steel contained relatively high carbon and strength hardening alloys such as Si, Mn, Ni, Cr, Mo and B, which deteriorated the weldability. Advances of high strength steels were realized in Japan through the advancements of; 1) steel cleanliness technology, 2) metallurgy for microstructure control, and 3) thermo-mechanical control process (TMCP) technology. Among them, the TMCP technology and the related metallurgy significantly contributed to realizing a wide variety of steel properties without adding much alloys. The TMCP technology is the production technology that can achieve both high strength and high fracture toughness under, in principle, an on-line process. Such property control is achieved primarily by refining the microstructure through the optimum control of chemical composition, heating, rolling, cooling and micro-alloying elements such as Nb and Ti. Since satisfying high strength, high toughness and high weldability are quite difficult to achieve by traditional processes, the TMCP technology became a breakthrough technology that opened the door to the wider application of high strength steels in steel structures. As shown in Figure 12, TMCP technology is a combination of “controlled rolling”, which results in fine grains mainly by introducing many dislocations as new grain sites (nucleation sites), and “accelerated cooling”, which promotes phase transformation at lower temperature while suppressing grain size growth. Using this technology, high performance steels can be produced efficiently with smaller amounts of C and alloy elements. Figure 13 shows a difference and chronological change in microstructures of steel. It shows that ordinary steel has a grain size of about 20μm, whereas the TMCP steel has a grain size of around 5μm.



(a) Timelines of steel ordered and maximum spans for bridges (Kanno 2016)



(b) Timelines of steel demand and maximum heights for buildings and towers (Kanno 2016)

Figure 10 Timelines of Japansese steel demand and performance

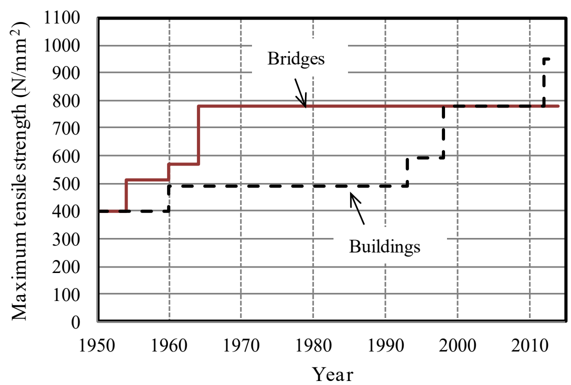


Figure 11 Timeline of maximum tensile strength of plate in Japan (Kanno 2016)

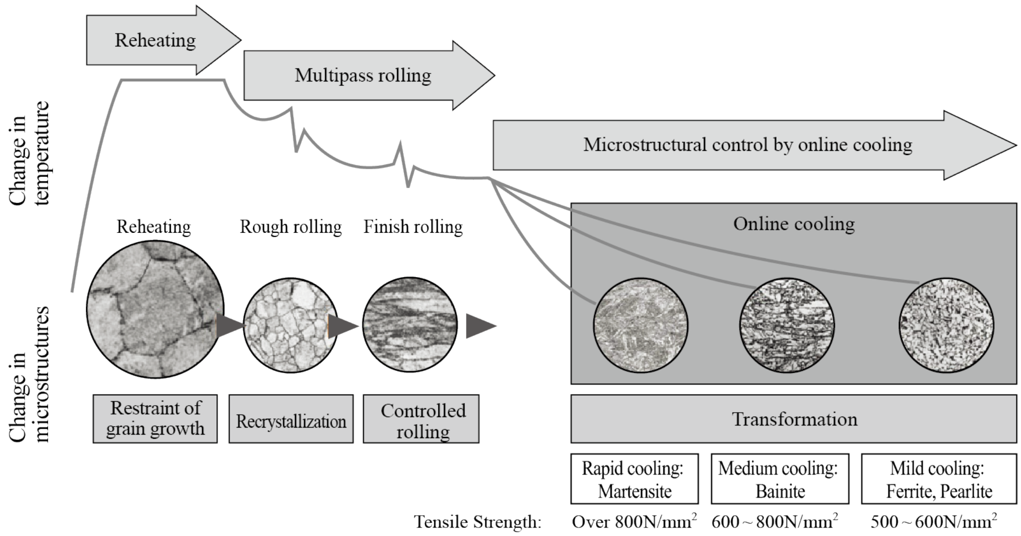
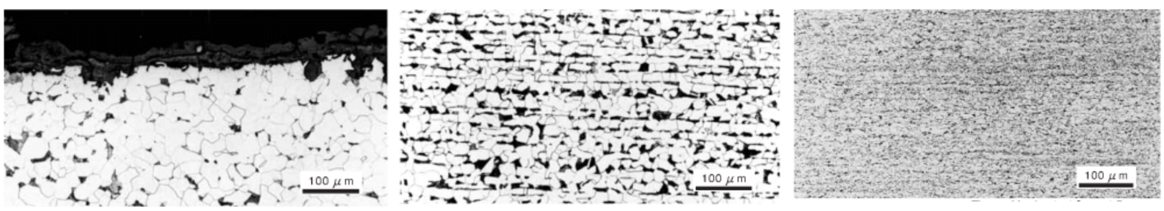
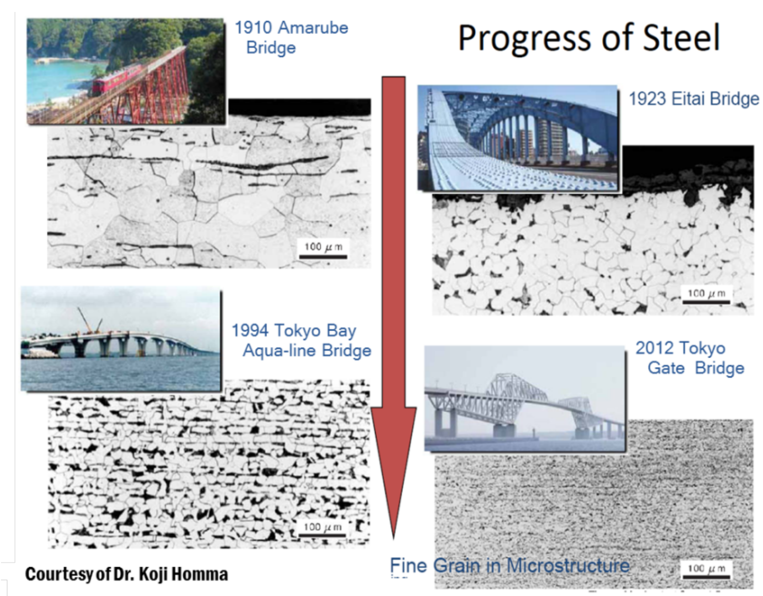


Figure 12 Outline of thermo-mechanical control process (TMCP) (Kanno 2016)



Steel by ingot-making Ordinary as-rolled steel TMCP steel

(a) Comparison of microstructure



(b) Chronological change in microstructure in bridges

Figure 13 Comparison of typical steel microstructure over time in Japanese applications (Homma 2014)

4.5 China

Application of high strength steel in Chinese building construction has followed with some of the larger signature projects:

> "In China, HSS [high strength steel] structures have undergone rapid development in recent years, and Q460 steel (fy = 460 MPa [66 ksi]) was adopted in China for the National Stadium ("Bird's Nest"), the CCTV headquarters, the Phoenix International Media Center and the Shenzhen Bay Sports Center… With advantages in economy, environmental protection and energy efficiency, HSS structures represent one of the important development trends in steel structures." Shi et al. 2018

Significant research and code development has also been undergone in China.

4.6 Australia

High strength steel has also seen application in Australia. Examples from Pocock (2005) also included in Shi et al. (2018) include "Star City in Sydney, Australia, with 650 and 690 MPa steels, [and] the Latitude in Sydney, Australia, with 690 MPa steel". These applications are consistent with fundamental research on 690 MPa Australian-produced QST steels summarized for example in such work as Rasmussen and Hancock (1995). Current code development is summarized in Chapter 5.

5 STANDARDS ADOPTION PATH FOR HIGH STRENGTH STEELS

5.1 Overview

In general the use of a given steel material in the AISC Specification is supported by (a) a materials standard, (b) a product standard, and (c) execution within the AISC Specification, i.e., the appropriate application standard, which is supported by research, and often past practice, justifying the applicability of the design guidance provided to the engineer.

Typically a new material for structural steel would need to (a) develop or expand an existing materials standard with ASTM, (b) modify the product standard, ASTM A6 to be inclusive of the new material, and then (c) work with AISC to (i) include their ASTM material standard in the Section A3 list of AISC 360 and (ii) support that inclusion of said materials standard will not compromise any of the existing provisions in AISC 360. For (a) and (b) there exists a defined process, for (c) the producer must participate in the AISC standards process – and the path to adoption is unclear and involves numerous task committees and long timelines.

The materials standard would typically need to include:

- Material composition (chemistry)
- Process category (e.g. DP, QST, etc.)
- Mechanical properties in tension (these can be quite specific, in general complete $\sigma - \epsilon$ behavior needs to be measured, longitudinal tension measurements are generally adequate, but for some steels/processes transverse may be required. Limiting values for yield stress (F_y) ultimate stress (F_u), ultimate elongation strain (ϵ_u), as well as modulus (E), strain hardening modulus (E_{st}), Ramberg-Osgood parameters (e.g., n), and limits on ratios of yield-to-ultimate etc. min/max are all regularly required).

The product standard is expected to be ASTM A6, modifications may be required if the composition, process, or other behavior of the new steel is unique. At a minimum the materials standard must be referenced. The ASTM A6 product standard includes

- Ordering, identification, packaging, rejection procedures and quality
- Chemistry (and analysis) and metallurgical structure
- Materials and manufacturing
- Dimensions and tolerances
- Testing criteria, inclusive of tension tests, CVN
- Weldability and heat treatment

Each of the preceding would need to be reviewed in light of the new material, but it is not expected that major changes would be needed, nor desired.

The application standard: e.g. AISC 360 covers all expected limit states for structural steel in building structures. Detailed considerations for high strength steel are detailed in Chapter 6. For a steel that is fundamentally not "equivalent" to an existing approved steel, and realistically for any material at 460 MPa (65 ksi) or greater there is likely to be questions on the applicability of many sections in AISC 360. Research/documentation (not necessarily testing) would potentially be needed to address these issues. The potential work needed is extensive, and includes:

- Material: $\sigma - \epsilon$, residual stresses, imperfections, fire/retention E, F_y, F_u factors
- Member: Stub column, long column, fully braced beam, unbraced (long) beam, multi-span beam/moment redistribution, short beam/shear, patch load, multi-axial load consistent with intended application, concrete composite member tests as appropriate
- Connection: Work with RCSC - bolted lap shear with varied limit states, work with AWS welded T-stub and more – align connection testing with intended application
- Fatigue: align testing with intended application
- Fire: material retention factors, component thermal and mechanical response and alignment with existing Specification, assembly tests consistent with intended application
- Seismic: Material variability bias, Cyclic performance in intended application, development of appropriate detailing as needed for system being considered

The extensive nature of the potential effort to bring in a new material grade, particularly at a higher strength can be a severe impediment to innovation. Further, one will find as new simulations or tests are performed that weaknesses in certain aspects of the existing standards will be revealed, even for traditional materials, and new burdens and challenges may emerge as general adoption is pursued for the new material.

Even questions as simple as who should be allowed to do the research or testing for a product under consideration can be complicated. Historically research supporting the materials standards were often, but not exclusively, conducted by the mills, and research for the product application were conducted in academic laboratories. Certified third-party labs are now more commonly acceptable in the U.S., and the use of uncertified academic labs can create questions in the standardization process. Thus, even where to conduct the needed research can be a potential barrier to adoption.

Alternative pathways to speed structural steel materials adoption are needed. The following section provides an overview of the basic materials adoption in AISC, followed by other U.S. steel standards including AASHTO and AISI in order to examine potential alternatives. In addition, a survey of international standards and processes for adoption of high strength structural steels is provided to explore possibilities of new processes and methods in the U.S.

5.2 Current AISC Approach to Materials Adoption

5.2.1 ASTM Designated Steel

For a new steel the preferred path is the development of an ASTM standard. If an ASTM standard for structural application is developed AISC may consider that standard for adoption. The commentary to AISC 360-16 explains that "There are hundreds of steel materials and products. This Specification lists those products/materials that are commonly useful to structural engineers and those that have a history of satisfactory performance." Based on this criteria AISC 360-16 states that "Structural steel material conforming to one of the following ASTM specifications [Table 4] is approved for use under this Specification: "

Table 4 Approved ASTM Materials in AISC 360-16

(a) Hot-rolled structural shapes	
ASTM A36/A36M	ASTM A709/A709M
ASTM A529/A529M	ASTM A913/A913M
ASTM A572/A572M	ASTM A992/ A992M
ASTM A588/A588M	ASTM A1043/A1043M
(b) Hollow structural sections (HSS)	
ASTM A53/A53M Grade B	ASTM A847/A847M
ASTM A500/A500M	ASTM A1065/A1065M
ASTM A501/A501M	ASTM A1085/A1085M
ASTM A618/A618M	
(c) Plates	
ASTM A36/A36M	ASTM A572/A572M
ASTM A242/A242M	ASTM A588/A588M
ASTM A283/A283M	ASTM A709/A709M
ASTM A514/A514M	ASTM A1043/A1043M
ASTM A529/A529M	ASTM A1066/A1066M
(d) Bars	
ASTM A36/A36M	ASTM A572/A572M
ASTM A529/A529M	ASTM A709/A709M
(e) Sheets	
ASTM A606/A606M	
ASTM A1011/A1011M SS, HSLAS, AND HSLAS-F	

5.2.2 Non-ASTM Designated Steel or ASTM steel not listed in body of AISC 360-16

Development of an ASTM standard is the preferred path; however AISC 360-16 does provide a possible secondary path for non-ASTM designated steels, or steels following an ASTM standard that is not explicitly listed in Table 4. The path is not explicit, such as in AISI S100-16 which lays out specific criteria, rather the commentary to Section A3 in AISC 360-16 needs to be invoked:

> "This Specification lists those products/materials that are commonly useful to structural engineers and those that have a history of satisfactory performance. Other materials may be suitable for specific applications, but the evaluation of those materials is the responsibility of the engineer specifying them. In addition to typical strength properties, considerations for materials may include, but are not limited to strength properties in transverse directions, ductility, formability, soundness, weldability including sensitivity to thermal cycles, notch toughness, and other forms of crack sensitivity, coatings, and corrosivity. Consideration for product form may include material considerations in addition to effects of production, tolerances, testing, reporting and surface profiles. " pg. 16.1-259 Commentary AISC 360-16

In addition the commentary further states that

> "*Hot-Rolled Structural Shapes*. The grades of steel approved for use under this Specification, covered by ASTM Specifications, extend to a yield stress of 100 ksi (690 MPa). " pg. 16.1-259 Commentary AISC 360-16

As discussed in Section 5.1, only one ASTM grade covering HPS 100W for bridge applications, actually constitutes a 100 ksi (690 MPa) product, although an HSLA QST grade for rolled shapes up to 80 ksi (550 MPa) is included. Thus, the commentary provides limited justification for inferring that AISC 360-16 is valid up to F_y=690 MPa (100 ksi).

The task group was not in agreement on whether or not the requirement to develop an ASTM standard was a significant hurdle to innovation. An example was given where a material/process met all applicable ASTM standards except thickness and the cognizant ASTM committee was either unwilling to expand its scope due to limited applicability, or moved too slow for project practicalities and as a result an ASTM path was essentially unavailable. It was also noted that the ASTM process is in general expensive and lengthy. AISC should consider if it can provide more explicit alternatives or guidance to encourage innovation.

5.3 AASHTO

AASHTO (2017) provides a list of materials (Table 5) that are allowed and in its commentary states that deprecated versions of these same materials may be allowed "with approval of the Owner". No direct mention is made of the use of other steels. Note, AASHTO has recently balloted QST grades and 50CR (now in A709) may be added to Table 5 in the near future.

Table 5 Allowable AASHTO steel material

AASHTO Designation	M 270M/ M 270 Grade 36	M 270M/ M 270 Grade 50	M 270M/ M 270 Grade 50S	M 270M/ M 270 Grade 50W	M 270M/ M 270 Grade HPS 50W	M 270M/ M 270 Grade HPS 70W	M 270M/ M 270 Grade HPS 100W	
Equivalent ASTM Designation	A709/ A709M Grade 36	A709/ A709M Grade 50	A709/ A709M Grade 50S	A709/ A709M Grade 50W	A709/ A709M Grade HPS 50W	A709/ A709M Grade HPS 70W	A709/ A709M Grade HPS 100W	
Thickness of Plates, in.	Up to 4.0 incl.	Up to 4.0 incl.	Not Applicable	Up to 4.0 incl.	Up to 4.0 incl.	Up to 4.0 incl.	Up to 2.5 incl.	Over 2.5 to 4.0 incl.
Shapes	All Groups	All Groups	All Groups	All Groups	N/A	N/A	N/A	N/A
Minimum Tensile Strength, F_u, ksi	58	65	65	70	70	85	110	100
Specified Minimum Yield Point or Specified Minimum Yield Strength, F_y, ksi	36	50	50	50	50	70	100	90

5.4 AISI approach

The American Iron and Steel Institute (AISI), through AISI S100, the *North American Specification for the Design of Cold-Formed Steel Structural Members*, has provisions for what it refers to as applicable steel and other steels. This approach provides a much broader path for materials adoption, but comes with its own complications.

Applicable steels are steels based on specifications providing mandatory mechanical properties and requiring test reports to confirm those properties. Applicable steels are grouped by their minimum elongation requirements over a two-inch (50-mm) gage length, as follows:

- Steels with a specified minimum elongation of ten percent or greater (elongation ≥ 10%) can be used without restriction under the provisions of AISI S100 provided the ratio of tensile strength to yield stress is not less than 1.08 and the minimum elongation is greater than or equal to either 10 percent in a two-inch (50-mm) gage length or 7 percent in an eight-inch (200-mm) gage length standard specimen tested in accordance with ASTM A370 or ASTM A1058.
- Steels with a specified minimum elongation from three percent to less than ten percent (3% ≤ elongation < 10%) are permitted to be used provided that the available strengths of structural members and connections are calculated in accordance with AISI S100 (excluding welded connections) using a reduced yield stress 0.9 F_{sy} in place of F_{sy}, and a reduced tensile strength of 0.9 F_u in place of F_u.
- Steels with a specified minimum elongation of less than three percent (elongation < 3%) are permitted to be used only for multiple web configurations such as roofing, siding, and floor decking provided a reduced specified minimum yield stress is used in bending calculations, yield stress limited to 75% of the specified minimum yield stress or 60 ksi, and tensile strength is limited to 75% of the specified minimum tensile strength or 62 ksi. Alternatively, the suitability of such steels for any multi-web configuration can be demonstrated by load tests, but not for the purpose of using higher loads than can be calculated using AISI S100.

Other steels are permitted, provided the following requirements are met:

- The steel conforms to the chemical and mechanical requirements of one of the listed specifications or other published specification, and F_y and F_u are the specified minimum values as given in the specified reference specification.
- The chemical and mechanical properties are determined by the producer, the supplier, or the purchaser, in accordance with the specified reference specification including all general requirements standards cited therein.
- The coating properties of coated sheet are determined by the producer, the supplier, or the purchaser, in accordance with ASTM A924/A924M.
- If the steel is to be welded, its suitability for the intended welding process is established by the producer, the supplier, or the purchaser, in accordance with AWS D1.1, AWS D1.3 or CSA W59, as applicable.

Other steels must also meet the permitted uses and restrictions based on ductility above. However, an exception is provided for purlins, girts, and curtain wall studs if minimum local elongation in a 1/2-inch (12.7 mm) gage length across the fracture is 20 percent, and minimum uniform elongation outside the fracture is three percent.

AISI S100 defines a published specification as requirements for a steel listed by a manufacturer, processor, producer, purchaser, or other body, which (a) are generally available in the public domain or are available to the public upon request, (b) are established before the steel is ordered, and (c) as a minimum, specify minimum mechanical properties, chemical composition limits, and, if coated sheet, coating properties.

In cases where the identification and documentation of the production of the steel have not been established, then the manufacturer of the cold-formed steel product must also establish that the yield stress and tensile strength of the master coil are at least 10 percent greater than specified in the referenced published specification.

The Commentary on AISI S100 provides the rationale for these provisions along with detailed references to the underlying research. AISI S100 with Commentary is available as a free download at: www.aisistandards.org. Printed version available as part of the 2 volume AISI D100-17 CFS Design Manual at: www.steel.org.

5.5 International Approaches and Standardization of High Strength Steel

5.5.1 Europe

The basic state of standardization for high strength steels is summarized in SCI (2019) and provided here:

> **"***Product standards:* European product specifications exist for construction products made from steels up to S960 [140 ksi] strength (Table 6). Hot rolled sections are available in S460 [65 ksi] steel from ArcelorMittal. Open sections in higher strengths are made from welded plate. Both cold formed and hot finished hollow sections are available in strengths up to S960 [140 ksi]. Longer procurement times are required for HSS sections.
>
> *Design standards:* Design rules for steels up to S700 [101 ksi] are currently available: EN 1993-1-12 gives supplementary rules for steels above S460 [65 ksi] and up to S700 [101 ksi]. These rules will be included in the second generation version of EN 1993-1-1. Work is now underway on a new version of EN 1993-1-12 which will cover steels of strength up to S960 [140 ksi].
>
> *Execution standards:* The execution standard EN 1090-2 claims to cover steels up to and including grade S960 [140 ksi]. However, it gives no rules for steels stronger than S700 [101 ksi] and even the rules for S420 [65 ksi] to S700 [101 ksi] steels have not been rigorously tested, particularly in terms of the impact of these requirements on the cost and quality of steel fabrications." SCI (2019)

Table 6 European material specifications for steel (SCI 2019)

Standard		Steel grade	Steel quality
EN 10025-2	Non-alloy structural steels	S275, S355	JR, J0, J2, K2
EN 10025-3	Normalized/normalized rolled weldable fine grain structural steels	S275, S355, S420, S460	N, NL
EN 10025-4	Thermomechanical rolled weldable fine grain structural steels	S275, S355, S420, S460	M, ML
EN 10025-6	Flat products of high yield strength structural steels in the quenched and tempered condition	S460, 500, 550, 620, 690, 890, 960	Q, QL, QL1
EN 10210[1)]	Hot finished structural hollow sections of non-alloy and fine grain steel	Non alloy S275, S355	JRH, J0H, J2H, K2H
		Fine grain S275, 355, 420, 460	NH, NLH,
EN 10219[1)]	Cold formed welded structural hollow sections of non-alloy and fine grain steels	Non alloy S275, S355	JRH, J0H, J2H, K2H
		Fine grain S275, 355, 420, 460	NH, NLH,
1) The next revision of EN 10210 and EN 10219 will include steels up to S960.			

5.5.2 Japan

Japan has been enjoying a wide variety of high strength and performance steels especially in building market, where structural steels with a nominal yield strength of up to 880 MPa (127 ksi) have been produced and applied. One reason for this is due to a unique building material approval system which has long been utilized in Japan. This approval system has been established and conducted under the Building Standards Law and its relevant rules (called "BSL" hereafter) that regulate building designs including materials in Japan. This section overviews the BSL material approval system.

Building materials used in Japan: Building materials such as steels, concrete and high strength bolts that are used in important structural members and elements must conform to Article 37 Item 2 of the BSL in Japan. As far as steel is concerned, the material must be included in the Japanese Industrial Standards (JIS) or be approved in advance by the Minister of Land, Infrastructure, Transport and Tourism in Japan. Steels for building use in the JIS have nominal yield strengths equal to and less than 325 MPa. Therefore, when it comes to higher strength steels, a designer must submit design calculation documents together with the corresponding Ministry approval certificates to local building officials before a project is initiated. The Ministry approval is generally obtained by steel manufacturers and they provide the steels together with the approval certificate.

This approval system has provided an environment where various manufactures can develop and provide their own advanced materials, meeting changing market needs in an efficient and timely manner. If this system did not exist, a time consuming approval procedure would be necessary for new materials to be included in a national specification (the JIS) which is equivalent to the ASTM (American Society of Testing and Materials). Note that any steels not included in the JIS are subjected to this Ministry approval, meaning that not only high strength steels but also high performance steels in terms of weldability, ductility, seismic performance, etc. can be approved. The steels shown in Table 7 are examples of the approved materials. They have not only high strengths but also low yield ratios and high toughness.

Procedure for material approval in Japan: An applicant such as a building material manufacturer should submit an application form for a material approval to a performance evaluation organization, together with required documents on its overview and scope, material specifications, statistical data and test descriptions, and quality control management. The performance evaluation organization is designated by the Ministry of Land, Infrastructure, Transport and Tourism (MILT) and there are currently 11 organizations in Japan for material approval. After evaluation through multiple technical meetings in an established peer review committee, the organization issues a performance evaluation report. Then the applicant submits an application form and the report together to the MILT. Accordingly, the Ministry approval certificate is issued to the applicant by the MILT. The procedure is outlined in Figure 14. Note that the right holder for the material approval is limited only to the applicant to which the Ministry approval certificate is issued. Even for the same material, each manufacturer must obtain the approval certificate in a separate action.

Other approval systems in Japan: Other approval systems than the one for new materials exist such as structural performance evaluation for high-rise building with over 60 m in height, buildings with base-isolation systems, and so on. The similar procedures to that for material approval should also be needed for the approval. In the structural performance evaluation, a series of time history analyses with various excitations are required to examine the seismic safety.

Table 7 Examples of the approved steels in Japan by Nippon Steel Corporation

Designation	Yield strength (N/mm^2)	Tensile strength (N/mm^2)	Yield ratio (%)	Charpy impact test		
				Thickness (mm)	Tem-perature (°C)	Absorbed energy (J)
BT-HT355B	355 ～ 475	520 ～ 640	≦ 80	40<t	0	≧27
BT-HT385B	385 ～ 505	550 ～ 670	≦ 80	16≦t	0	≧70
BT-HT440B	440 ～ 540	590 ～ 740	≦ 80	19≦t	0	≧47
BT-HT630B	630 ～ 750	780 ～ 930	≦ 85	9≦t	0	≧47
BT-HT400C	400 ～ 550	490 ～ 640	≦ 90	16<t	0	≧70
BT-HT500C	500 ～ 650	590 ～ 740	≦ 90	19≦t	0	≧70
BT-HT700B	700 ～ 900	780 ～ 1000	≦ 98	12<t	-20	≧47
BT-HT880B	880 ～ 1060	950 ～ 1130	≦ 98	9≦t≦12	0	≧53

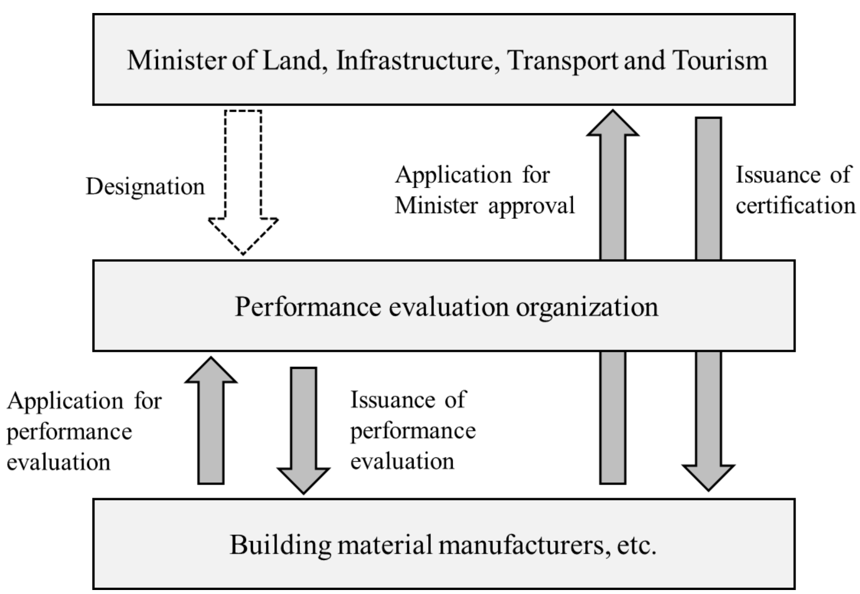


Figure 14 Procedure for Ministry approval of a steel in Japan

5.5.3 Australia

Australian and Australian-New Zealand Standards allow for the use of HSLA QST (Quenched and Tempered) Structural Steels up to 700 MPa (101 ksi): - AS4100 amended in 2012, AS/NZS2327 published in 2017, and AS/NZS5100 Part 6 published in 2017. Working groups for Australian standards have initiated improved slenderness limits for 900 MPa (130 ksi) yield structural members.

5.5.4 China

Development of a high strength steel standard is reported in Shi et al. (2014, 2018). The standard leverages an extensive amount of experimental and simulation-based research conducted in China. The aim of the standard is to cover structural steel applications using materials with F_y from 420 MPa (65 ksi) to 960 MPa (140 ksi). The basic approach of the draft standard was to take the best of Eurocode, the AISC Specification, and the legacy Chinese code to create this new standard. Updated residual stress patterns, column curves, connection strengths, and seismic design parameters are all to be included. A draft standard is under public review in China currently and a final version should be published in the near future.

6 REVIEW OF AISC 360-16 FOR HIGH STRENGTH STEEL ADOPTION AND PRELIMIANTY EXAMINATION OF AISC 341

This Chapter provides a brief review of the AISC 360 Specification from the perspective of what sections would need to be revisited when considering adoption of a high strength structural steel that is currently outside of AISC 360's scope.

6.1 Materials

In concept, a wide range of steel is permitted by AISC 360 with F_y up to 100 ksi (690 MPa) - Table 4 lists all referenced material standards. However, this seemingly broad range of applicability can be sharply limited by the referenced ASTM standards. For instance, for rolled shapes, A992 is preferred and only F_y=345 MPa (50 ksi) is designated – the complete list is provided in Table 8. From Table 8 we can conclude that AISC 360-16 allows rolled shapes from mild and HSLA carbon steel of 36 ksi (290 MPa) or 50 ksi (345 MPa), and extends up to 55 ksi (380 MPa) for C-Mn steel and as high as 80 ksi (550 MPa) for HSLA QST steel.

Table 8 Designated ASTM grades for rolled shapes in AISC 360-16

	F_y ksi	**36**	**42**	**50**	**55**	**60**	**65**	**70**	**80**	**100**
ASTM	**Type F_y MPa**	**250**	**290**	**345**	**380**	**415**	**450**	**485**	**550**	**690**
A36	Mild	X								
A529	CMn			X	X					
A572	HSLA CbV		X	X	X	X*	X*			
A588	HSLA W	X	X	X						
A709	Bridges Mild	X*		X*						
A709	Bridges QST			X*			X*	X*		
A709	Bridges HPS			X*				X*		X*
A913	HSLA QST			X		X	X	X	X	
A992	Mild and HSLA			X						
A1043	Low Y/T	X		X						

* designates grade that is only listed as applicable to bridges in the ASTM standard

In AISC 360 there are a few explicit limits on F_y for specific situations and many embedded (implicit) F_y dependencies. The limits / dependencies are not primarily about strength, but about other properties that may change as strength increases (e.g. ductility, strain hardening, residual stress). Chapter 2 of this report provides a brief overview of variations in steel stress-strain properties and the impact that these material-level variations have on member-level behavior.

In the following AISC 360-16 was reviewed to identify primary locations where F_y is constrained explicitly or where there are implicit dependencies on F_y. The following sections summarize this review under the categories: members, systems and connections. Within the members category, compression members are examined in more detail to illustrate the issues that would need to be addressed as part of incorporating high strength steel into AISC 360 Chapter E. Required work for beams would be similar in concept.

6.2 Members

6.2.1 Local Stability

Local buckling limits (*Specification* B4.1) are functions of F_y that were developed in the context of existing structural steel (roughly $F_y \leq 50$ ksi (345 MPa)). Limits may need to be adapted for higher strength steel with different residual stress and strain hardening characteristics. For example the discussion in Section 2.5 demonstrates potential deficiencies in the compact section λ_p limits for HS3 beams. A recent AISC ad hoc TG report on local buckling width-to-thickness limits provides additional discussion on this issue (AISC ad hoc w/t report 2019). Further, as Chapter 2 makes clear, the definition of F_y for gradual yielding steels needs to be implemented with care when applied to HS3.

6.2.2 Tension Members

There are no significant dependencies on F_y for design of tension members (*Specification* Chapter D). However, while "yielding in the gross section" may be a clear strength limit state in mild steel, for steel without a defined yield plateau the strength may change even over strains/deformations within the serviceable range. Work on stainless steel structural members as part of the development of AISC 370-22 has indicated that materials with very high F_u/F_y ratios, as is also possible in some high strength steels, may need to consider deformation limits in addition to traditional yield in the gross and fracture in the net section strength limit states.

6.2.3 Compression Members

Compression members (*Specification* Chapter E) require careful evaluation in the context of high strength steel. Topics include:

- The column curve: The current AISC *Specification* column curve (Equations E3-2 and E3-3) is based on the SSRC 2P column curve (Figure 15). SSRC 1P and SSRC 3P were also developed, where SSRC 1P was recommended as more appropriate for $F_y > 90$ ksi (Table 9). Efforts to adopt new steels with higher F_y should consider a column curve that accurately represents the compression strength, multiple column curves, multiple resistance factors, or alternative means and methods to take advantage of the actual strength for what will likely be costly material.
- Buckling modes: torsional and flexural-torsional buckling modes (*Specification* E4) appropriate the column curve to translate elastic buckling behavior into ultimate strength (considering residual stresses and initial imperfections). Compared to the extensive supporting research on flexural buckling, the ultimate strength provisions for torsional and flexural-torsional buckling are not as rigorously supported. Efforts to adopt new steels with higher F_y should carefully consider torsional and flexural-torsional buckling behavior, particularly as higher F_y members are expected to become more slender, globally and locally. (Work in cold-formed steel shapes has shown that the use of the flexural column curve for flexural-torsional buckling can be overly conservative if the torsion end boundary conditions restrain warping – this has been definitively shown for angles – and may require investigation to maximize the benefit of costly HS3 applications, particularly in trusses).
- Single angles and built-up members (*Specification* E5 and E6): strength depends on semi-empirical relationships that need to be examined in the context of higher F_y.

- Slender elements (*Specification* E7): the approach for strength reduction due to local plate behavior should be studied in the context of higher F_y compression members. The effective width method is adaptable for higher strength steel and variations in residual stresses and initial imperfections. Again, for HS3 applications more sections are likely to utilize these provisions than currently – thus additional care is needed.

Table 9 Column Curve Selection (Bjorhovde 1972 and 1988, Ziemian 2010)

Fabrication Details		Axis	Specified Minimum Yield Stress of Steel (ksi)				
			≤ 36	37 to 49	50 to 59	60 to 89	≥ 90
Hot-rolled W-shapes	Light and medium W-shapes	Major	2	2	1	1	1
		Minor	2	2	2	1	1
	Heavy W-shapes (flange over 2 in.)	Major	3	2	2	2	2
		Minor	3	3	2	2	2
Welded Built-up H-shapes	Flame-cut plates	Major	2	2	2	1	1
		Minor	2	2	2	2	1
	Universal mill plates	Major	3	3	2	2	2
		Minor	3	3	3	2	2
Welded Box Shapes	Flame-cut and universal mill plates	Major	2	2	2	1	1
		Minor	2	2	2	1	1
Square and Rectangular Tubes	Cold-formed	Major	N/A	2	2	2	2
		Minor	N/A	2	2	2	2
	Hot-formed and cold-formed heat-treated	Major	1	1	1	1	1
		Minor	1	1	1	1	1
Circular Tubes	Cold-formed	N/A	2	2	2	2	2
	Hot-formed	N/A	1	1	1	1	1
All stress-relieved Shapes		Major and Minor	1	1	1	1	1

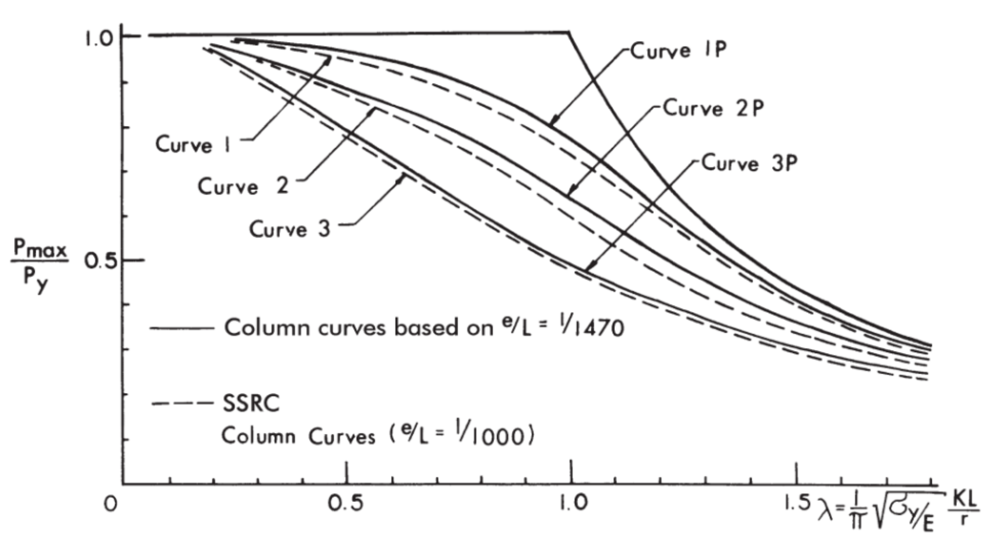


Figure 15 Column Curves (Bjorhovde 1972, Ziemian 2010)

Application of HS3 in bridges has shown the advantage of hybrid shapes (For example in a 3-plate I-section the use of different F_y for the web and flange). If such shapes are used in building construction they are likely to act as beam-columns, and thus column strength provisions (or at least clear guidance) for hybrid shapes are likely to be needed in the future.

6.2.4 Flexural Members

Flexural members (*Specification* Chapters F and G) require careful evaluation in the context of high strength steel. Flexural and shear strength provisions were developed for existing structural steel (roughly $F_y \leq 65$ ksi (450 MPa)). Inelastic local buckling and lateral-torsional buckling strength provisions have embedded assumptions about residual stress characteristics. R_{pg} derivation contains assumptions that relate to conventional steel (F_y = 36 ksi (250 MPa)). Vertical buckling limits contain assumptions about residual stress characteristics. These all need to be evaluated for impact of high strength steel with different residual stress and strain hardening characteristics. However, for flexural members AISC 360 benefits from its close relationship with the AASHTO bridge standard which has been employing 70W and 100W steels with success in bridge girders.

6.2.5 Interaction

Provisions for design of members subjected to combined forces and torsion (*Specification* Chapter H) are largely dependent on *Specification* Chapters E, F and G. Interaction equations should be re-evaluated in the context of high strength steel.

6.3 Systems

This section covers topics that are more related to system-level issues than to individual members (although some topics are also pertinent at the member level).

6.3.1 Stability

Stability (*Specification* Chapter C) is the most critical system-level design consideration. In the Direct Analysis Method, stiffness adjustment (*Specification* C2.3) accounts for inelasticity including the effect of partial yielding of the cross section, which may be accentuated by the presence of residual stresses. This stiffness reduction was developed in the context of existing normal strength structural steel (roughly $F_y \leq 65$ ksi) and may need to be adapted for higher strength steel with different residual stress characteristics. Design by advanced analysis (*Specification* Appendix 1) also includes stiffness reduction (*Specification* 1.2.2b).

For inelastic analysis where ductility is required at plastic hinge locations for moment redistribution, the limit $F_y \leq 65$ ksi is currently imposed (*Specification* B3.3 and 1.3.2a). Existing data does not justify increasing this limit (see Chapter 2).

6.3.2 Fatigue and Fracture

The current fatigue provisions (*Specification* Appendix 3) were developed in the context of existing structural steel (roughly $F_y \leq 65$ ksi) and need to be evaluated in the context of high strength steel. As explained by Duane Miller:

"The current model as contained in Appendix 3 suggests that the steel strength is not a significant factor in terms of predicting fatigue behavior. In most cases, the allowed stress ranges are independent of the strength of the steel. However, there is at least one example where the allowed stress range for higher strength steel is lower than for lower strength steel (albeit only a minor decrease). However, the implication of this is that it is possible (and I'd opine probable) that higher strength steel will actually have lower fatigue resistance. One more comment on this topic: while it is generally true that that strength of the steel does not affect the fatigue resistance of welded (or fabricated) steel, the advantages we seek to gain from higher strength steel (i.e., lighter structures or structures with more capacity) inevitably increase the stress range and decrease the fatigue life."

"(Need) to identify the potential changes that will be needed in toughness requirements as steel strength increases. Two factors need to be considered: first, higher strength steel is usually loaded to a higher stress level, which increases facture concerns. Secondly, and particularly for weld metal with F_y > 100 ksi, it becomes increasingly challenging to obtain weld metal with good notch toughness. The issue of potential fracture concern issues should be added to the check list of items to be investigated." (Duane Miller, Personal Communication, June 7, 2018)

6.3.3 Fire

The current provisions for structural design under fire conditions (*Specification* Appendix 4) contain material models for elevated temperature that are limited to $F_y \leq 65$ ksi based on past research and existing data.

Based on existing research it should be expected that the type of steel mild, HSLA, DP, etc. and the details of the processing can definitively influence response under fire – thus it should be anticipated that retention factors will be a function of ASTM grade and not just F_y, nor the same for all steels.

6.3.4 Other

The following topics do not have significant dependence on F_y: design for serviceability (*Specification* Chapter L), fabrication and erection requirements (*Specification* Chapter M), QC/QA (*Specification* Chapter N), design for ponding (*Specification* Appendix 2), evaluation of existing structures (*Specification* Appendix 5), member stability bracing (*Specification* Appendix 6), alternate methods of design for stability (*Specification* Appendix 7) and approximate second-order analysis (*Specification* Appendix 8).

6.4 Connections

The impact of high strength steel on local connection limit states (*Specification* Chapter J) should be studied. More research, and possibly material development is needed to establish the response of bolted or welded connections to HS3 materials. Welded connections are one area where new developments are obviously needed. Welded connections in higher strength steel will require appropriate weld metal.

> "As the strength of the base metal increases, particularly beyond 120 ksi, it will be more and more difficult to make welded connections with "matching" strength metal. The use of "undermatching" weld metal, with appropriate weld and joint designs, will likely be a bigger issue. This may in turn affect localized connection limit states. The welding-related challenges of higher strength steel cannot be minimized, and I'd opine this may be a major issue." (Duane Miller, Personal Communication, June 7, 2018)

RHS/CHS and box-section connections (*Specification* Chapter K) need to be evaluated for high strength steel since current provisions have been developed in the context of normal strength steel and associated material limits are imposed ($F_y \leq 52$ ksi and F_y / $F_u \leq 0.8$ in *Specification* Tables K2.1A, K3.1A, K3.2, K4.1A, K4.2A).

6.5 Composite

For design of composite members (*Specification* Chapter I), structural steel is limited to $F_y \leq 75$ ksi (*Specification* I1.3) based on scenarios studied in research. See Section 2.6.7 for more information.

6.6 Discussion of AISC 341-16 and Other Standards

The *Seismic Provisions* govern the design, fabrication and erection of structural steel members and connections in a seismic force-resisting systems (SFRS). The *Provisions* are applied in conjunction with the *Specification*, and all requirements of the *Specification* are applicable unless otherwise stated in the *Provisions*. Similar to the *Specification*, a wide range of steel is permitted by the *Provisions* with F_y up to 50 ksi in general and 70 ksi in specific cases (Table 6.1).

Table 10 ASTM Designated Steels in AISC 341-16

Element type	**ASTM designation**
Shapes	A36/A36M, A529/A529M, A572/A572M [Grade 42 (290), 50 (345) or 55 (380)], A588/A588M, A913/A913M [Grade 50 (345), 60 (415), 65 (450) or 70 (485)], A992/A992M
CHS/RHS	A53/53M, A500/A500M (Grade B or C), A501/A501M, A1085/A1085M
Plates	A36/A36M, A529/A529M, A572/A572M [Grade 42 (290), 50 (345) or 55 (380)], A588/A588M, ASTM A1011/A1011M HSLAS Grade 55 (380), A1043/A1043M
Bars	A36/A36M, A529/A529M, A572/A572M [Grade 42 (290), 50 (345) or 55 (380)], A588/A588M
Sheets	A1011/A1011M HSLAS Gr. 55 (380)

Material limits are more stringent in the *Provisions* than in the *Specification* since earthquake-resistant design relies on a combination of 1) predictable inelastic behavior in specific members and 2) capacity-based design of the remainder of the system so that nominally elastic behavior is achieved. For 1), significant material and member ductility is required, along with well-defined hardening / overstrength properties so that 2) can be accomplished. For members in which inelastic behavior is expected, $F_y \leq 50$ ksi is generally required, except for in systems classified as Ordinary. However, the *Provisions* provide flexibility (*Provisions* A3.1): "Either of these specified minimum yield stress limits are permitted to be exceeded when the suitability of the material is determined by testing or other rational criteria." For columns in some systems, $F_y \leq$ 70 ksi is permitted.

If high strength steel is to be implemented in SFRS for members in which inelastic behavior is expected, significant research is required to define inelastic behavior and ensure adequate member ductility. Existing literature, as reported in Sections 2.5 and 2.6.6, shows that concerns exist about available post-yielding rotation capacity in HS3 flexural members, which could create difficulties with the current *Provisions* approach of relying on plastic hinging for energy dissipation. Implementing high strength steel in SFRS for members that are not expected to yield and are proportioned per capacity-design is a more straightforward (and likely the most beneficial) path.

7 RECOMMENDATIONS

Based on the work of the TG the following recommendations for high strength structural steel (HS3) adoption were formulated. The recommendations represent a broad set of tasks spanning from education and technology transfer down to detailed research necessary for expanding the AISC Specification. The required effort is large, but in all cases initial steps can be taken in each category that would be productive and aligned with AISC objectives. The TG strongly recommends initiating efforts on all of these fronts.

__Critical Initial Path Items__

CP1: Establish High Strength Structural Steel (HS3) material targets: A small but diverse AISC led group should establish initial desired minimum performance properties for targeted HS3 for use in building construction at 550MPa (80 ksi), 690MPa (100 ksi), and 960MPa (145 ksi). These minimum performance properties provide critical targets for the mills and are necessary for creating a competitive marketplace for HS3. Further, the performance targets can guide the necessary research to improve the Specification to address these grades and speed their future adoption in the Specification. This effort parallels the success of the HPS experience in U.S. bridges, the adoption of HS3 in Japan, and recent high strength steel rebar for concrete in the U.S. These top down decisions are needed for HS3 to flourish.

CP2: Establish potential and create a vision: Leveraging the expertise of the top-level structural steel designers that already participate with AISC, a funded design charette/competition should be engaged to develop the potential of HS3 in building construction. Separate competitions/charettes for new high-rise buildings, new long-span roofs, retrofit building design, seismic building design, design for blast, etc. are all needed. The result should provide high-level information for public consumption and technical information for early adopters. High level champions for HS3 and a publicly consumable vision of the use of HS3 in buildings is needed. Participants should not be overly constrained in developing their vision.

__Education__

E1: Promote cost and availability information: Lack of information on cost and market availability is a strong impediment to HS3 adoption today. AISC should leverage its publications (MSC), the Manual, website (e.g., www.aisc.org/steelavailability/), NASCC conference etc. to better address/directly address HS3 availability. AISC should work with the mills (and/or AISI) and the mills should designate a POC or similar for HS3 to work with AISC on this effort. Increasing the information available to designers on cost and availability is critical.

E2: Summarize successes/case studies: A large number of HS3 applications were identified worldwide in this report. However, U.S. awareness of these successes and their lessons learned is low. AISC should consider funding a study/report/website of the application of HS3 in buildings so that U.S. engineers/builders/developers can understand the reasoning and successes in the application of HS3 and the potential efficiencies that have been realized. The TG noted that past efforts such as the 2017 NASCC session on Moving Forward with High Strength Steel could provide potential starting points.

__Advocacy__

A1: Incentivize demonstration projects - lobbying: HPS use in bridges in the U.S. was specifically incentivized through legislation. AHSS use in automobiles was a direct response to fuel efficiency regulations. HS3 has the potential to be part of steel's solution to efficiency/resilience/sustainability in buildings and should be incentivized for use. AISC should consider teaming with AISI and others and determining how HS3 fits within their legislative and related lobbying priorities.

A2: Partner with funding agencies and organizations: Using past success with HPS bridges, and based on a vision for more resilient, sustainable, efficient, buildings AISC should advocate for HS3 as a needed priority within the industry: AISI, MBMA, SDI, SJI, MCA, RCSC, AWS etc.; with federal agencies: DOE, HUD, NIST, Army, NSF; and with private foundations: Pankow, MKA. The TG recognizes that this is a difficult task, but past success shows that it is absolutely necessary for success and to speed implementation.

A3: Inform other building decision-makers: Create specific advocacy and outreach for the application of HS3 to key decision-makers that are not within the typical AISC sphere of influence; including: general contractors, building owners, developers, BOMA, real estate boards.

Fabricator Needs

F1: Capture fabrication challenges: HS3 steels and shapes place new requirements on fabrication, especially tooling and processes for cutting and welding. AISC should fund a collaboration between leading fabricator(s) and the mills to establish and share best practices in the fabrication of HS3. This information should be disseminated to engineers and fabricators and AISC should consider what role, if any, HS3 fabrication could play in certification (or extensions to certification) in the longer term. Past lessons show that excellence in fabrication is particularly needed for HS3 applications to be successful.

Specification Task Committee Led Efforts

T1: Formally connect AISC to world-wide specification efforts in HS3. Establish which TC takes the lead on HS3 and connect this TC with world-wide efforts. Formally connect the designated TC to its counterpart efforts in Eurocode, China, Japan, and Australia. Fund the TC to develop a summary of world-wide Specification efforts and their impact on AISC specifications in HS3 – the new Chinese standard, Eurocode standards updates, and existing Japanese code all are identified herein as providing information useful for updating the AISC Specification. Funding for travel to formalize the connection amongst the specification entities should also be considered.

T2: Develop a material performance path for the adoption of steel in AISC specifications. Currently AISC 360's criteria for listing a material in the standard is "those products/materials that are commonly useful to structural engineers and those that have a history of satisfactory performance." This standard of care provides simplicity and clarity, but does not encourage innovation. The TG recommends that alternative pathways for material adoption be included in the *Specification*. The pathway may or may not require an ASTM standard, alternative models exist from AISC in the procedures of the AISI Specification, or with 3rd party certification as used in Japan. If performance targets for 550-960 MPa (80 - 145 ksi) steels are established as recommended herein this effort would be simplified and aligned with those targets. At a minimum a checklist should be established.

T3: Develop an execution path for HS3 in AISC 360. Consider a given HS3, what additional simulation and testing should be required for this material to be adopted in AISC 360? As detailed in Section 5.1 herein the list is expansive; and the rejection of the increase in the plastic hinge limit above F_y=65 ksi (450MPa) as detailed in Section 2.5 is a case study in the challenges. Nonetheless, no matter how long the list uncertainty needs to be removed from the process. An AISC TC should be charged with overseeing a funded effort to develop and detail such a list. Key items in this effort include details of any required testing, and guidance on number of samples to be performed and how to judge equivalence to approved steels and satisfactory performance for existing standards. A guidance document, not a specification is the anticipated product from this effort. The ATC115 effort for high strength rebar is a potential model. (Funded project with TC4 and TC10 oversight likely needed at a minimum)

T4: Clarify execution path for HS3 in AISC 341. Materials applications in AISC 341 are far more specific to a given Seismic Force Resisting System. In addition, the existence of the pre-qualified connection path provides another means to integrate HS3 materials innovation into the existing standard. Nonetheless a similar "execution path" effort to that for AISC 360 should be conducted. (Funded project with TC9 oversight)

Research

Develop and fund research in HS3. This effort could be completed in collaboration with the mills, with federal funding agencies and with private foundations. The topic area is vast, the research recommended below is intended to align with the recommendations above.

R1: Develop RFP for justifying HS3 performance criteria: the selected material performance criteria (item CP1) for 550MPa (80 ksi), 690MPa (100 ksi), and 960MPa (145 ksi) will require technical substantiation. What are the critical material criteria for successful member and connection performance in gravity and lateral systems? Simulation can likely be utilized to provide the needed substantiation. A research project is needed to support the team developing the material targets for HS3.

R2: Develop RFP to research design charette/competition solutions: Once a vision of HS3 applications is established by the design charette/competition it will be necessary to show that the new ideas developed by the designers are capable of performing as intended. Whatever is put forth – massive HS3 box columns, thick shallow beams without fire protection, castellated beams, hybrid F_y beam-column moment frames, HS3 boundary elements on composite steel plate shear walls, etc. These novel solutions will require investigation. Research teams that are inclusive of the designer and provide fundamental evidence of desired performance should be pursued.

R3: Develop RFP for improved specification provisions. With HS3 material performance criteria established it is then possible to work systematically through the AISC 360 specification and recommend improvements. Multiple projects are needed to address the wide array of needs, specific TCs could be assigned to work with the project teams. High strength steel topics with direct Specification implications identified herein include:

- imperfections and residual stresses;
- define F_y for steels that do not have a yield plateau. (Use of $F_{0.2\%}$ is common, but may not provide the best agreement, See Section 2 of this report);
- w/t limits, emphasis on rotational capacity of beams;
- compression/columns, emphasis on multiple column curves, F-T buckling, slender element columns;

- flexural/beams, LTB bracing, Local-LTB interaction, hybrid shapes, more;
- shear/beams, TFA, boundary details, hybrid shapes, more;
- connections: welded connections; ductility demands in bolted connections, fatigue
- composite: HS3-concrete composite members and subsystems;
- system stability, Chapter C τ reductions and equivalent system imperfections;
- fire: retention factors; and
- seismic: R_y and R_t factors, SFS applicability.

R4: Open HS3 RFP. Rather than work top-down it can be more impactful and useful to leverage the creativity of the research community directly. An open RFP on the subject of the use of HS3 in steel buildings provides a path to providing AISC with the best proposals aligned with a given need. Asking the proposal to be responsive to the two critical path items: target HS3 material, and designer vision could provide some structure to the solutions. Though less structured, the potential for long-term impactful work is great.

Recommended Action Summary Table

Recommendation	**AISC Lead**	**Funding Need**
Critical Initial Path Items		
CP1: Establish High Strength Structural Steel (HS3) material targets	TC1/Board	$
CP2: Establish potential and create a vision	CoR	$$
Education		
E1: Promote cost and availability information	?	$
E2: Summarize successes/case studies	?	$$
Advocacy		
A1: Incentivize demonstration projects – lobbying	?	$?
A2: Partner with funding agencies and organizations	?	$?
A3: Inform other building decision-makers	?	$$
Fabricator Needs		
F1: Capture fabrication challenges	?	$
Specification Task Committee Led Efforts		
T1: Formally connect AISC to world-wide specification efforts in HS3	TC3/4?	$
T2: Develop path for material adoption of HS3 in AISC	TC10	$
T3: Develop an execution path for HS3 in AISC 360	TC3/4?	$$
T4: Clarify execution path for HS3 in AISC 341	TC9	$
Research		
R1: Develop RFP for justifying HS3 performance criteria	CoR	$$
R2: Develop RFP to research design charette/competition solutions	CoR	$$
R3: Develop RFP for improved specification provisions	CoR	$$$$$
R4: Open HS3 RFP.	CoR	$$$

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Appendix 1: Development and Prioritization of Tasks

The Task Group developed 13 potential activities that were aligned with the objective of understanding and easing adoption of high strength structural steels for construction in October 2017. Members of the task group ranked the activities on a scale of 1 to 5 and then further prioritized the ranking during an online meeting in January of 2018. The final prioritized rankings and the task items are summarized in the following table. This report largely summarizes the results of these activities.

Table 11 List and prioritization of tasks for high strength steel in construction

Survey Rank	Mtg. Rank	Work Item
4.88	1	Solicit manufacturers and fabricators w.r.t. existing and in-development high strength steels that may require AISC specification modifications for adoption – with a goal of decreasing the time to market by knowing the areas to target for improvement in this specification development cycle.
4.00	2	Solicit designers.. Develop an RFP (or similar outreach) that AISC could consider for building structural engineers and architects to explore the potential of 1000MPa (145 ksi) steel systems. Consider design competitions or other forward facing work that could be used to demonstrate the future of steel construction.
4.63	3	Determine all explicit (and implicit when possible) Fy limits in AISC 360 (341 tooX?X) compile and list these limits in one document. (This might be expanded to other properties, since weldability etc. is often a key issue – note TC3 notes indicate L. Fahnestock may have information on this available).
4.38	4	Whether explicit or implicit summarize the role of Fu/Fy, elongation, n, other parameters inherent in AISC Specification provisions.
4.25	5	Given a hypothetical Fy=1000 MPa (145 ksi) steel (with all other additional properties known) provide working outline of what steps would be required to introduce such a steel to AISC 360 (341 too?).
4.50	6	Solicit, compile, and review completed research AND STANDARDS that would justify expansion of Fy limits for particular steels that are not covered in AISC 360-16 (e.g. HPS steels investigated at Lehigh in the 80's and 90's, high strength angles investigated by SGH in the early 2000's, other domestic and international research). Provide recommendations to the TCs on their adoption.
4.38	7	Specifically solicit recent international research on RHS/CHS/Tube and box sections up to 1000MPa (145 ksi) and provide concise summary of findings to relevant TCs regarding Specification changes necessary to adopt such steels. (BWS notes that significant work on China has recently been examining this)
3.25	8	Provide recommendations on how to model physical imperfections and residual stresses in higher strength steels
4.50	9	Provide short term (next cycle) ~~and long term (next couple of Specification cycles)~~ recommendations to the AISC Spec. Comm. on how to incorporate

		higher strength steels into the Spec.
4.50	10	Provide ~~short term (next cycle) and~~ long term (next couple of Specification cycles) recommendations to the AISC Spec. Comm. on how to incorporate higher strength steels into the Spec.
3.86	11	Review ASTM steels and develop a list of candidate high strength steels for AISC 360/(341?) consideration.
3.75	12	Recommend acceptable fracture properties and weldability for high strength steel in construction
2.88	13	Develop an addendum to the AHSS Guidelines handbook for construction. See the current handbook here (http://www.worldautosteel.org/projects/advanced-high-strength-steel-application-guidelines/) The lack of construction application and guidance is a clear barrier to adoption.

Note, tasks 11-13 identified, but not pursued as a priority by the task group.

Appendix 2: Detailed survey results

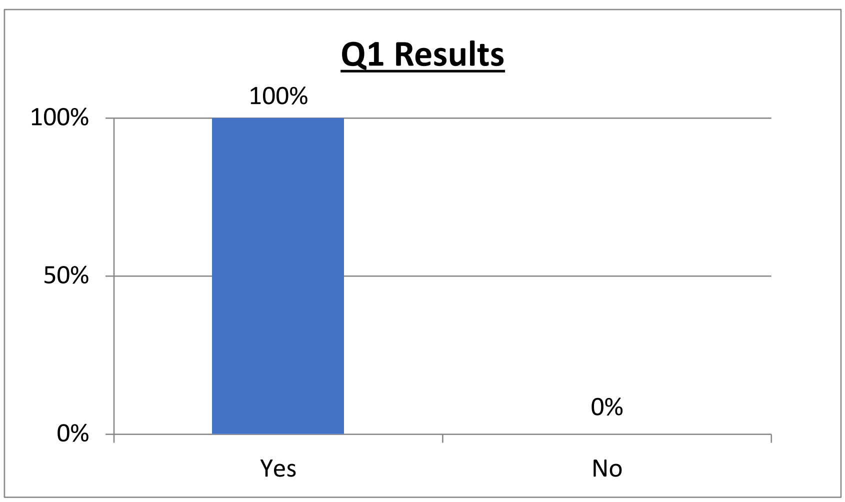
AISC High Strength Steel Ad hoc
Engineer Survey Poll Results—February 2018

Total survey participants: 27

1. Advanced High Strength Steels (AHSS) is material with yield strength ranging from 65 ksi (460 MPa) to 145 ksi (1000 MPa) or higher.

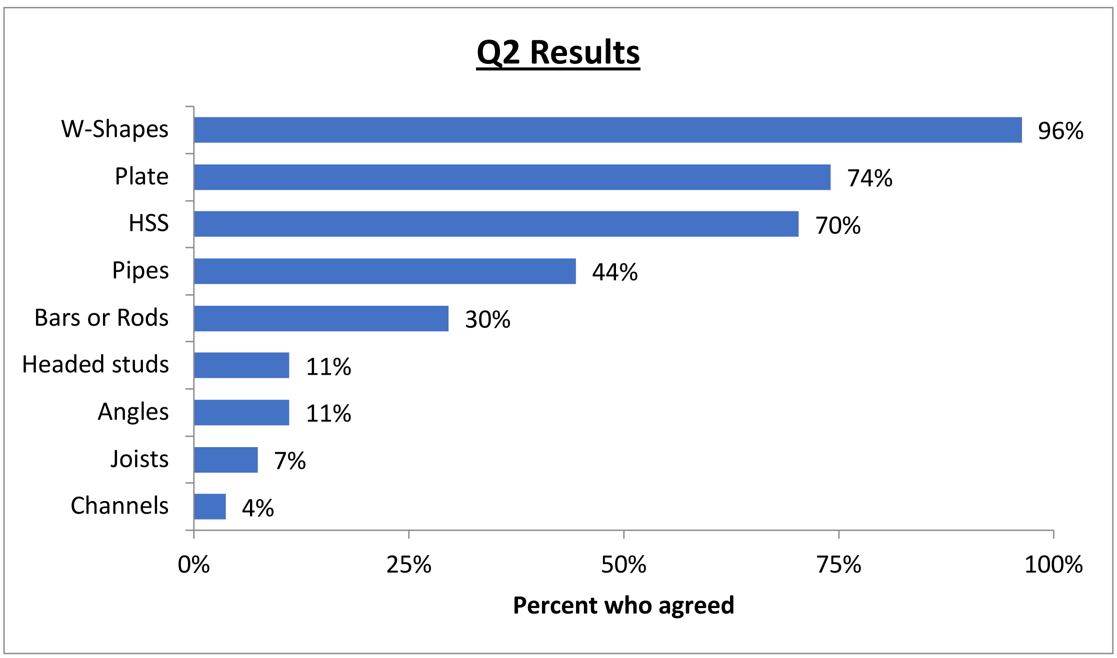
Do you think you could use AHSS in your practice and project design?

- Yes
- No



2. Which of the following shapes or materials do you think should be offered in advanced high strength steel? (check all that apply)

- [ ] a. W shapes
- [ ] b. Channels
- [ ] c. Angles
- [ ] d. Tubes
- [ ] e. Pipes
- [ ] f. Joists
- [ ] g. Plates
- [ ] h. Bars or Rods
- [ ] i. Headed Studs
- [ ] j. Cold form steel
- [ ] Other (please specify)

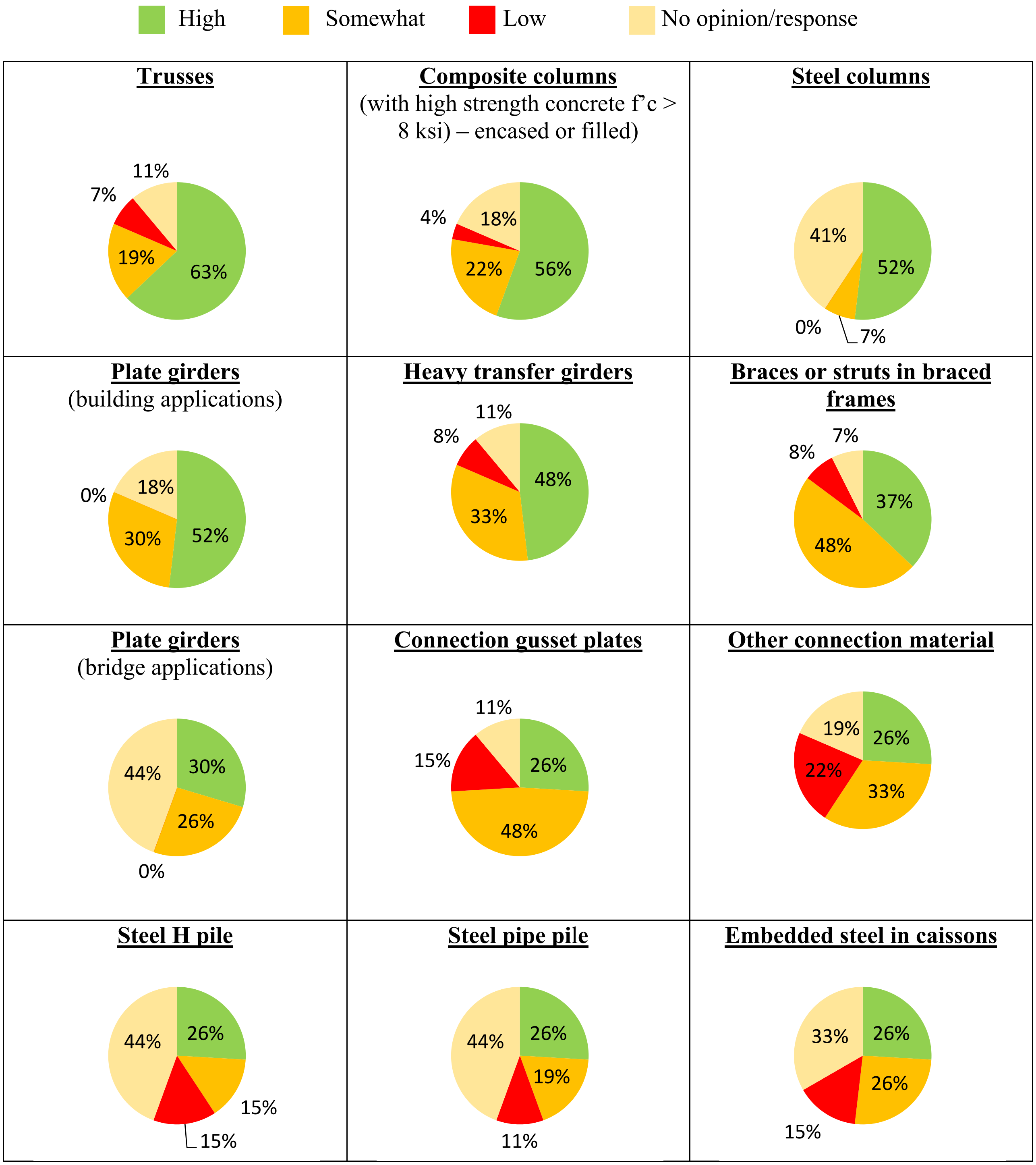


3. Indicate which of the following applications you believe advanced high strength steel (Fy > 65 ksi) would be desirable.

	Not Desirable	Somewhat Desirable	Very Desirable	No Opinion
a. Steel columns	○	○	○	○
b. Composite columns with high strength concrete (f'c > 8 ksi) (encased or filled)	○	○	○	○
c. Braces or struts in braced frames	○	○	○	○
d. Trusses	○	○	○	○
e. Joists - normal span	○	○	○	○
f. Joists - long span	○	○	○	○
g. Joists - composite	○	○	○	○
h. Normal beams	○	○	○	○
i. Normal girders	○	○	○	○
j. Heavy transfer girders	○	○	○	○
k. Plate girders - bridge applications	○	○	○	○
l. Plate girders - building applcations	○	○	○	○
m. Connection gusset plates	○	○	○	○
n. Other connection material	○	○	○	○
o. Steel H piles	○	○	○	○
p. Steel pipe piles	○	○	○	○
q. Embedded steel in caissons	○	○	○	○

Are there any additional applications not listed above?

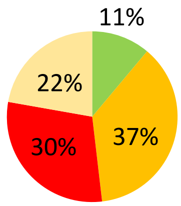
Q3 Results -- Desirability



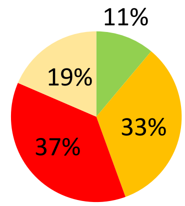
<u>Q3 Results (continued) -- Desirability</u>

High   Somewhat   Low   No opinion/response

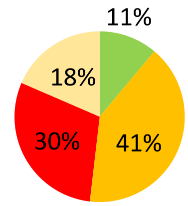
<u>Joists</u>
(long span)



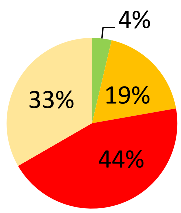
<u>Normal beam</u>



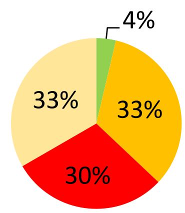
<u>Normal girders</u>



<u>Joists</u>
(normal span)



<u>Joists</u>
(composite)

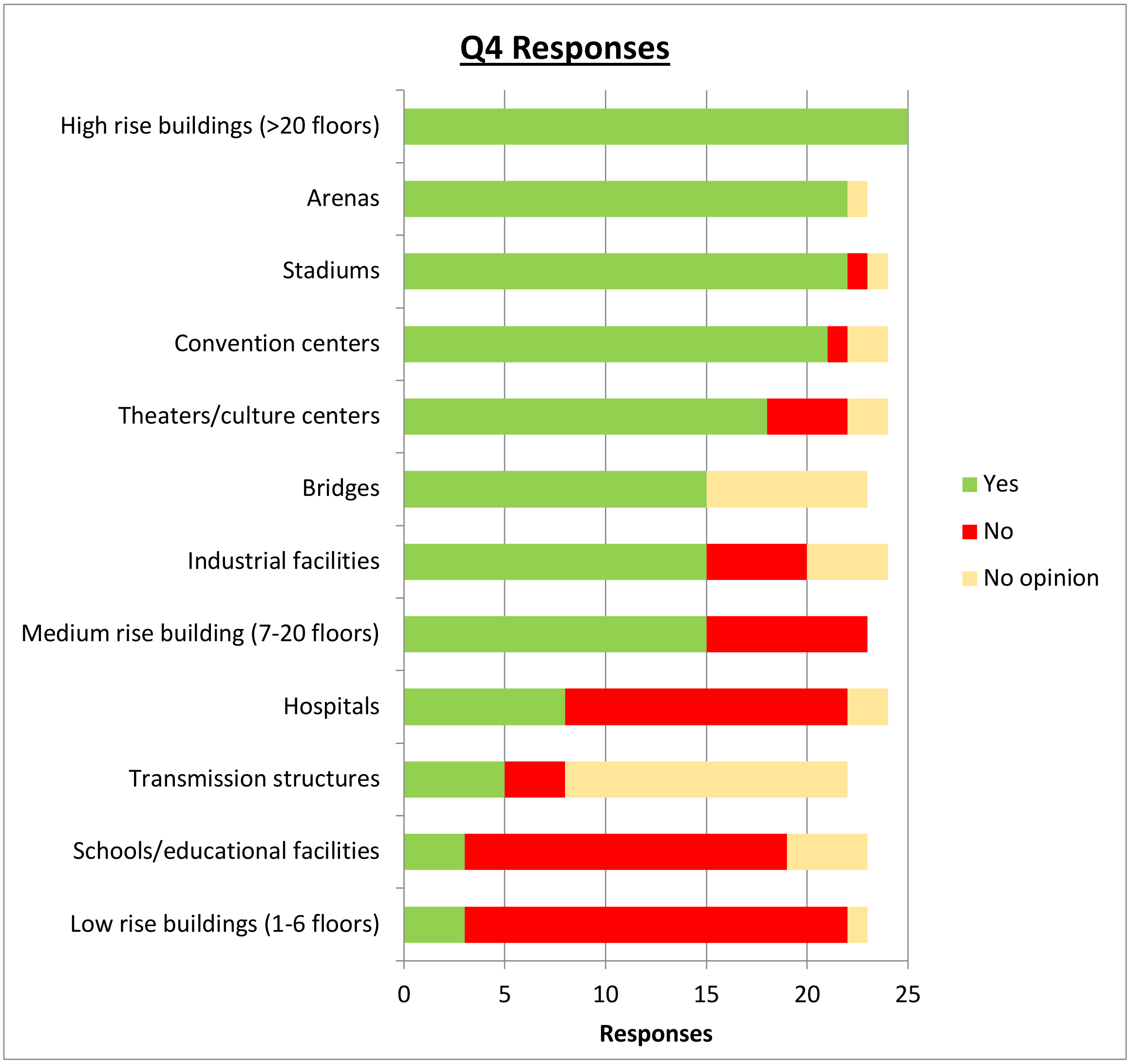


<u>Additional responses for Q3:</u>
1) Thick plates for fabrication of large cross-section, built-up column or brace sections.
2) Any cases where self-weight is a big part of design loads and deflections can be managed

4. In what type of structure(s) do you think could use advanced high strength steel (Fy > 65 ksi)? (check all that apply)

	Yes	No	No opinion
a. Low rise buildings (1-6 floors)	◯	◯	◯
b. Medium rise buildings (7-20 floors)	◯	◯	◯
c. High rise buildings (> 20 floors)	◯	◯	◯
d. Arenas	◯	◯	◯
e. Theaters or cultural centers	◯	◯	◯
f. Convention centers	◯	◯	◯
g. Stadiums	◯	◯	◯
h. Hospitals	◯	◯	◯
i. Bridges	◯	◯	◯
j. Industrial facilities	◯	◯	◯
k. Schools and other educational facilities	◯	◯	◯
l. Transmission structures	◯	◯	◯

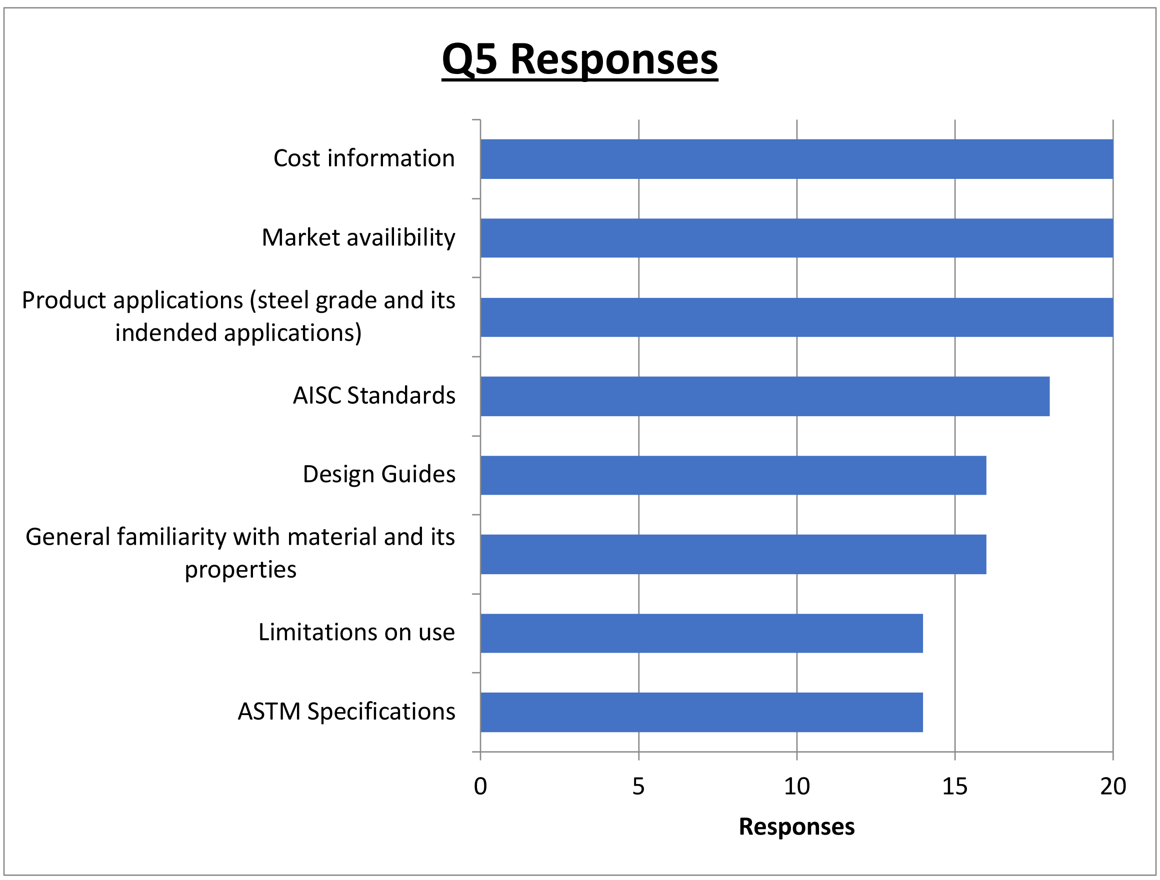
Other (please specify)



Additional responses for Q4:
1) Special structures
2) Any structures where self wt. is a big part of load and deflections can be managed

5. What is missing (i.e. impediments that exist) for you to use AHSS? (check all that apply)

- [ ] a. Product applications (steel grade and its intended application(s))
- [ ] b. General familiarity with AHSS and its properties
- [ ] c. ASTM specifications
- [ ] d. AISC standards
- [ ] e. Design guides
- [ ] f. Market availability
- [ ] g. Cost information
- [ ] h. Limitations on use
- [ ] Other (please specify)



Additional responses for Q5:

1) Testing showing adequate deformation capacity in sub assemblages (e.g., beam/column joints, brace to column connections, etc.)
2) Technical knowledge in mass distribution

6. What other problem(s) do you see that prevents AHSS to be used on your projects (other than those listed in the above questions)?

Responses for Q6:

1) Weldability, ductility, toughness concerns would all need to be addressed.
2) Material availability is the biggest question. Most projects are driven by schedule. Availability and cost information is necessity.
3) Reliable welding technologies; higher strength bolted connections
4) In projects where stiffness drives the design (high-rise columns, long span trusses, etc.), steel elements are chosen based on area, not strength
5) Wherever strength is needed and serviceability is manageable
6) No improvement in elastic modulus
7) Education to steel fabricators, erectors, and special inspectors. It's not enough for engineers to be able to specify them. If others involved with the product are resistant to its use, it will adversely impact its adoption in the built environment

7. Please provide any other comments or suggestions that you believe is pertinent to the Task Committee efforts.

Responses for Q7:

1) I think there is a lack of clear understanding of the availability of high strength, thick plate material in the market. There are producers who say they can produce plate products well beyond the current ASTMs, but the Structural Engineer needs reliable information on material properties, availability, cost, and other design considerations at an early stage, in order to be able to implement.
2) For seismic regions (say seismic categories C and D) it should only be used in elements intended to remain elastically or practically elastically.
3) As more and more high strength material is used, must further emphasize serviceability limit states in design.
4) The committee should take a look at the NIST and Pankow foundation reports on developing criteria for HS reinforcing bars in concrete
5) We are successfully implementing grade 65 steel for columns, braces and truss members in a high-rise residential project. Switching from grade 50 to 65 saved the owner 100 tons of material.
6) Need the research and design guides, cost info and availability
7) Most structural steel design is limited by deflection concerns. AHSS will not address this.

List of companies that participated in the survey:

- Brad Young & Associates, Inc.
- Computerized Structural Design
- Drucker Zajdel Structural Engineers, Inc.
- Gilsanz Murray Steficek LLP Engineers and Architects
- The Harman Group, Inc.
- Holmes Structures
- LeMessurier Consultants, Inc.
- Rutherford + Chekene
- Severud
- Simpson Gumpertz & Heger
- Stanford University
- Structural Affiliates International, Inc.
- Walter P. Moore and Associates, Inc.

Appendix 3: Q&A on Evolution of High Performance Steel in U.S. Bridges

High Performance Steel

Questions provided by

Ben Schafer, Professor, Civil Engineering, Johns Hopkins University

Answers provided by

American Iron and Steel Institute

1. **From your perspective what was the genesis of the desire for HPS steels in bridges?**

The problems we face today such as the aging infrastructure, stretches our resources thin and challenges our creativity. Deficient bridges are a critical part of the infrastructure, representing a major impediment to mobility on our highways. The resultant time lost to congestion and risks to public safety are a drag on our nation's productivity. Innovative materials, such as high performance steel, play an increasingly important role as we attempt to meet the transportation challenges of the future, including enhancing and expanding our highway bridges. We will be more dependent on high performance materials such as High Performance Steel to provide economical structures which have 100-year design life and which will help us with our goal to improve mobility by eliminating deficient bridges.

High Performance Steel has reduced levels of carbon and carbon equivalents to provide improved weldability: HPS is weldable with reduced or no preheat making it more economical to fabricate. HPS has higher levels of fracture toughness to improve structure reliability. And it has improved corrosion resistance properties compared to conventional bridge steel. The higher strength versions of HPS allows the designer to use fewer lines of girders to reduce weight and cost, use shallower girders to solve vertical clearance problems, and increase span length to reduce number of piers on land or obstructions in waterways.

2. **From the perspective of the mills what has been the most challenging aspect in terms of material uptake for HPS steels in bridges?**

As common to technical innovations, convincing the decision-makers to change and adopt the new technology represents a consistent challenge. For HPS steels in bridges it was important to develop welding procedures to provide confidence in the new steels. The most common and effective method of eliminating hydrogen-induced weld cracking is specifying minimum preheat and interpass temperature for welding. In general, the higher the preheat the less chance for formation of brittle microstructures and more time for the hydrogen to diffuse from the weld. However, preheating is time consuming and costly. One of the goals in developing high performance steels was to reduce or eliminate preheat.

Initially, submerged arc welding (SAW) and shielded metal arc welding (SMAW) were the only processes recommended for welding high performance steel. Based on research, consumables for the flux cored arc welding (FCAW) and gas metal arc welding (GMAW) processes are now available. In addition, SMDI developed fabrication guides with recommendations for specific consumables that have demonstrated that they are capable of successfully producing acceptable quality welds.

3. **From the mills or steel industry perspective what sort of partnerships were required to make HPS steel happen in bridges?**

In short, partnerships were established among FHWA, US Navy, AISI, steel producers, bridge designers, welding experts, steel fabricators, and academia. The rest of the story…

In 1992, the Carderock Division of the Naval Surface Warfare Center (CDNSWC) partnered with the American Iron and Steel Institute (AISI) and FHWA to develop new and improved high-performance bridge steel alternatives. The team brought together a cadre of professionals in steel production, bridge design, bridge fabrication and welding, as well as specialists from the government, and academia.

Together the group developed three high-performance steels: HPS 50W, HPS 70W and HPS 100W. These are all weathering steel grades, meeting Zone 3 toughness requirements, and with significant improvement in weldability compared to conventional bridge steels. By 1997, their efforts had proven so successful that the Civil Engineering Research Foundation (CERF) awarded the team the Charles Pankow Award for Innovation.

The Nebraska Department of Transportation was the first to use HPS 70W in the design and construction of the Snyder Bridge - a welded plate girder steel structure. The 150-foot, simple-span bridge was originally designed for conventional grade 50W steel. When HPS became available, the Nebraska DOT replaced grade 50W steel with HPS 70W steel of equal size. The intent was to use this first HPS 70W bridge to gain experience in the HPS fabrication process. The fabricators found that no significant changes were needed.

The Tennessee Department of Transportation was another of the early users of HPS, which was chosen for their Route 53 Bridge in Jackson County. The project's immediate advantage was an approximate ten percent reduction from standard construction costs. While the cost per pound for the girders was higher, the weight of the steel required for the bridge was reduced by 24 percent, resulting in significant overall savings.

4. **How high, in terms of Fy, has been found to be practical today for HPS in bridges? Are there any active efforts continuing to push the use of higher Fy in bridge applications?**

Before the development of HPS, the steel grades for bridges had minimum yield strength of 36,000 and 50,000 lbs. per square inch (36 and 50 ksi). Currently available HPS steels have minimum yield strengths of 50, 70, and 100 ksi:

- HPS 50W Up to 3" As-Rolled
    - Yield Strength, Fy, ksi (MPa) min. 50 (345)

- HPS 70W Up to 4" (Q&T). 2" (TMCP)
    - Yield Strength, Fy, ksi (MPa) min. 70 (485)

- HPS 100W Up to 4”
    - Yield Strength, Fy, ksi (MPa) min 100 (690)

The industry has continued to improve the capability of HPS grades. For example, for HPS 70W plate thicknesses greater than 2.5 inches, the steel’s manganese content can be increased from 1.35% to 1.50% to maintain minimum yield strength requirements.

Other studies suggest that for special fracture-critical applications improved impact properties of HPS 70W can be achieved. For example, a minimum Charpy V-Notch level of 50 ft-lb is possible for test temperatures as low as -25°F. SMDI, through their various research committees, continues to discuss future upgrades of the specifications.

5. **Conceptual work on high strength steel often indicates the strength of using multiple grades of steel, whether as hybrid shapes or hybrid steel structures? Has this happened in the HPS bridge applications?**

Yes, the AASHTO HPS Guide encourages the use of hybrid girders, i.e. combining the use of HPS 70W and Grade 50W steels. A hybrid combination of HPS 70W in the negative moment regions and Grade 50W or HPS 50W in other areas results in the optimum use of HPS and attains the most economy.

HDR Engineering, Inc. in association with the University of Nebraska-Lincoln performed a study to compare the cost differences between bridge designs using HPS 70W, conventional grade 50W and a combination of the two grades of steels. A total of 42 different girder designs were made using the AASHTO LRFD Bridge Design Specifications - HL-93 Live Load. The girder designs had 2-span continuous layout, covering a span range of 150', 200' and 250', variable girder spacing of 9' and 12', and designs in grade 50W, HPS 70W and a variety of hybrid combinations.

The study concludes that:

(1) HPS 70W results in weight and depth savings for all span lengths and girder spacing.

(2) Hybrid designs are more economical for all of the spans and girder spacing. The most economical hybrid combination is grade 50W for all webs and positive moment top flanges, with HPS 70W for negative moment top flanges and all bottom flanges.

(3) LRFD treats deflection as an optional criterion with different live load configurations. If a deflection limit of L/800 is imposed, deflection may control HPS 70W designs for shallow web depth.

An example:

The Pennsylvania Department of Transportation (PennDOT) used HPS 70W in the Ford City Bridge, which was opened to traffic in July 2000. PennDOT performed full-scale tension and fatigue testing, extensive material testing and weld testing on this project. It is a three-span continuous welded steel plate girder bridge with spans of 320'-416'-320'. The first span is curved horizontally with a radius of 508'. The other two spans are on tangent. There are four lines of girders spaced at 13.5'. HPS 70W is used in the negative moment regions and grade 50W elsewhere. This hybrid combination of steels resulted in 20% reduction in steel weight, and enabled the girder sections to be constant depth instead of haunched. By eliminating the variable web depth, a costly longitudinal bolted web splice was avoided.

6. **Fracture is often cited as a concern for HPS or AHSS steel applications. From the mills perspective is this concern warranted, or is fracture performance related to material chemistry - and just a mother design option?**

SMDI [AISI] is not aware of a “concern” about fracture of HPS because the High Performance Steel grades have much higher fracture toughness than the conventional grades of steel used for bridge construction. HPS makes the transition from brittle to ductile at a much lower temperature than conventional grades. So HPS improves reliability by minimizing the chance of sudden brittle failure.

Having greater fracture toughness, HPS better resists cracks in the bridge structure. This property provides more time for inspectors to detect and repair any fatigue cracks that might develop before the structure becomes unsafe.

The HPS 70W(485W) steels tested show ductile behavior at the extreme service temperature of -60°F for Zone 3. It is a major accomplishment of the HPS research and an important advantage of HPS in controlling brittle fracture. With higher fracture toughness, high performance steels have much higher crack tolerance than conventional grade steels. Full-scale fatigue and fracture tests of I-girders fabricated of HPS 70W (485W) in the laboratory showed that the girders were able to resist the full design overload with fracture even when the crack was large enough to cause 50% of loss in net section of the tension flange. Large crack tolerance increases the time for detecting and repairing fatigue cracks before the bridge becomes unsafe.

A US Department of Transportation Memo (https://www.fhwa.dot.gov/bridge/120620.pdf) states, "High Performance Steel (HPS) and use of internally redundant detailing both have the potential to further improve the fracture propagation resistance of FCMs and should be implemented where practical."

From the SMDI Guide - *The superior toughness of HPS 70W steel, combined with the requirements specified herein, suggest that fabrication in accordance with this HPS Fab Guide will produce structural members that meet Fracture Critical Member (FCM) specifications. At this time, it is necessary to fabricate fracture critical members, when identified as such in the contract documents, in accordance with AWS D1.5, Section 12, AASHTO/AWS Fracture Control Plan (FCP) for Nonredundant Members. Otherwise, fabrication of conventional, non-fracture critical HPS 70W components can be successfully completed when work is done in conformance with AWS D1.5 combined with the recommendations of this HPS Fab Guide. It is important to keep in mind that the HPS Fab Guide recommends consumable handling in accordance with AWS D1.5, Section 12.6.5 for the SMAW process, Section 12.6.6 for the SAW process, and Section 12.6.7 for the FCAW and GMAW Metal Cored process, to control the diffusible hydrogen levels to H8 maximum. Otherwise, no other provisions of the Fracture Control Plan are recommended, unless the component is specifically designated a FCM.*

7. **Are there other infrastructure applications (pipes, silos, towers, luminaries, etc.) that are adopting or actively looking at HPS or AHSS steel that you are aware of?**

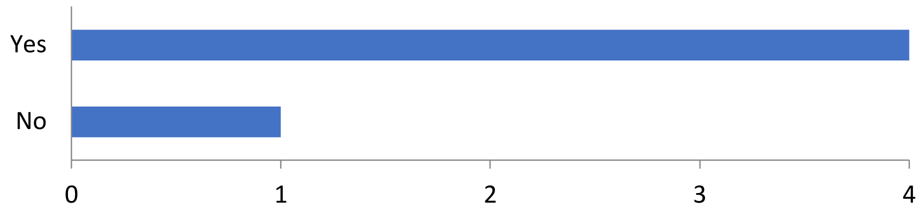
High-performance steel was originally developed for use by the military in submarine construction. As noted, in 1992, the Carderock Division, Naval Surface Warfare Center partnered with AISI and the Federal Highway Association (FHWA) to research ways HPS could be transferred from military technology to civilian applications and develop the new and improved steel alternative for use in bridge construction. HPS is now used primarily for highway bridge construction. I believe it has been considered for orthotropic bridge decks – I do not know of any other applications at this time.

Note: SMDI is also working with the industry to develop applications for A1010 steel (now known as A709-50CR – it was recently added to A709; the latest edition is A709/A709M − published in September 2017) – it is considered for bridge applications where severe chloride corrosive environments or excessive time of wetness exist. This includes exposure to excessive road salts or marine environments where maintenance is difficult, expensive or injurious to the environment. For more information, see http://www.usa.arcelormittal.com/~/media/Files/A/Arcelormittal-USA-V2/what-we-do/steel-products/plate-products/Duracorr-Bridge.pdf

Appendix 4: Survey of Manufacturers on High Strength Steel

AISC Adhoc Task Group on High-Strength Steel
Manufacturer Survey Results
5/21/19

1. Do you have a (high strength) structural steel product or considering a structural steel product for the construction market in which the AISC Specification is a potential impediment to its use in design?



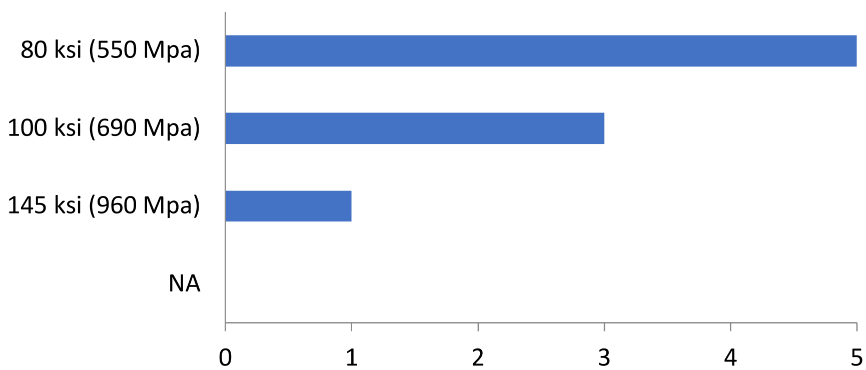
If appropriate, please describe some aspect of this product:

- **Grade 90 with a targeted yield strength of min 90 ksi having the purpose to generate lighter structural solutions.**

- **We have different high strength steels in programme, up to 100 ksi. Of special interest for the construction industry are the thermomechanically rolled steels acc. A1066, e.g Gr. 65, Gr. 70.**

- **www.bullmoosetube.com; Stratusteel 100 or 110; HSS with yield strength of 100KSI or 110KSI.**

2. Does your company have interest, current or future, in the use of structural steels in the construction market with yield stress greater than or equal to the following?

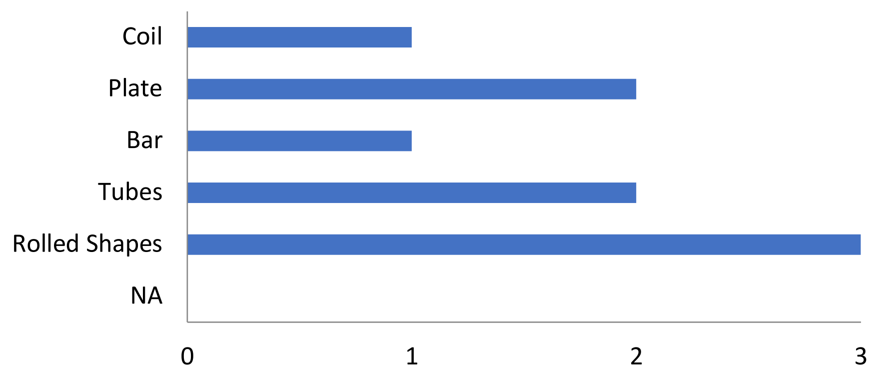


Any additional yield stress?

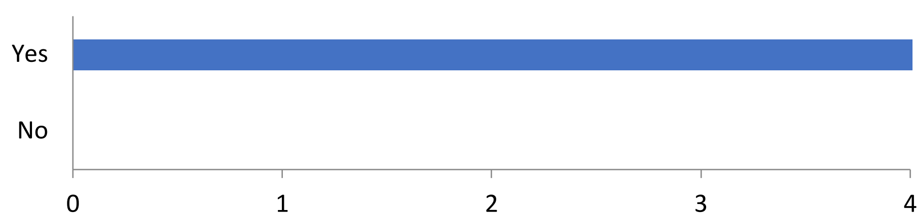
- **620 MPa (90 ksi)**

- **485MPa (70KSI), 620 MPa (90KSI) and 760MPa (110KSI); see ASTM A1112**

3. What types of high strength steels (Fy > 450 MPa, 65 ksi) are you producing, or likely to produce, for potential use in the construction market? (mark all that apply)



4. Structural steel design engineers have indicated in a separate poll (administered by this task group) a strong interest in employing high strength steels in their designs; however, they indicate that information on cost and availability are primary challenges. Do you have a mechanism for design engineers to ascertain preliminary cost/availability information?



If yes, what is it?

- **By requesting to our sales engineers, the price indication and the delivery time of high strength steel at the early stage of the design.**

- **e.g. product finder and easy Online feasibility check + additional online welding and processing help**

- **Website www.bullmoosetube.com**

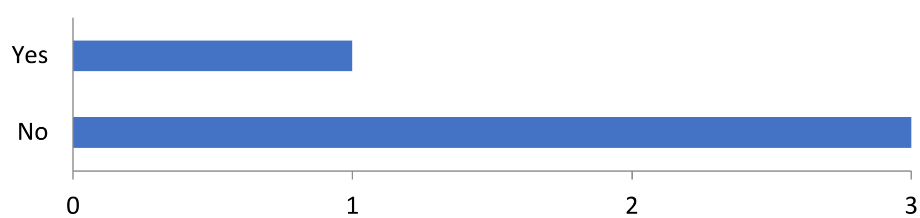
5. As a rough relationship between cost and Fy (assuming ~15% elongation) an IABSE (2005) article postulated that the relative price per ton of steel is which results in estimates of relative price as follows:

Fy (MPa)	Fy (ksi)	Relative Cost
235	34	1.0
345	50	1.2
450	80	1.4
690	100	1.7
960	145	2.0

Please comment, to the extent you can, the degree to which you find this estimate to be accurate and if you find this estimate to be one that would be useful for designers in the planning stage.

- **Design engineers should permanently be in contact with producers' sales engineers to obtain the latest price indications for different steel grades. To be taken into account that the relative prices differ from a product to another (i.e. plates vs structural shapes).**
- **Between 345 and 450 as well as 450 to 690 these are more or less reasonable estimates**
- **Estimate is accurate**
- **I believe the Relative Cost multiplier is for WF, not HSS. 50KSI = 1.0 80KSI = 1.4 100KSI = 2.0**
- **Don't know if this accurate or not.**

6. Steels employed in the U.S. construction market typically have ASTM standards, is this a barrier/impediment to the adoption of your products in the construction market?



Comments:

- **It is the position of this steel producer that an ASTM specification be opened for any new product that is introduced to the market, including high strength steel solutions. Official standards like ASTM are supporting engineers in the current design and provides confidence in their decisions.**
- **We have also ASTM grades in our delivery programme, e.g. A572, A1066, A514, A709**

7. Research on existing high strength steel rolled shapes has shown that the residual stresses may be lower than in conventional mild steels. Our task group is considering how this can/should impact design predictions in the future. If your company has a high strength steel product (> 450 MPa, 65 ksi) do you have information on residual stresses? Would you be willing to share this information in some form with the task group?
    - **Residual stresses are a consequence of the manufacturing process and for the same level of yield strength these values can be different from a producer to another.**
    - **Unfortunately not possible to share**
    - **Yes**
    - **We have some Universities researching this subject and their results will be public.**
    - **We do not have information on residual stresses to share with the task group.**

8. Please share anything else you would like with regard to high performance / high strength steel applications in the construction market.
    - **Composite steel concrete design should be extended in such way that includes the high strength steels up to 100 ksi.**
    - **High strength steels besides the usually already cost effective material reduction high strength steels often lead to reduced follow up costs, e.g. welding costs, foundations costs, transport cost, ...**
    - **high performance / high strength steel applications in the construction market.**
    - **Bull Moose Tube Company can produce all the Grades listed in ASTM A1112/A1112M-18 for HSS.**
    - **Once steel exceeds YS of 80 KSI the decrease in elongation/ductility becomes a concern as it impacts crack propagation and base metal toughness.**

Appendix 4 Addendum: Additional Response from CIMOLAI

Additional Email Q&A conducted with CIMOLAI

Questions: Thomas Poulos, TG Member

Answers: Ennio Picco, CIMOLAI

1. Does Cimolai see a benefit from the use of higher-strength steels?
   Yes, definitely.

2. Has Cimolai suggested/pursued/used high-strength steel in any of your projects?
   a. To what strength levels and for what shapes and building types?
      We have experience with following steel grades:
      - S690
      - S890

      With reference to shapes, we believe that the flat products could be enough to give to the designer/fabricators the possibility to customize sections, following the design requirements.
      It is important to extend the thickness range and to perform/improve material toughness and through-thickness proprieties (Z quality).
   b. What are some benefits that your project received from the use of high-strength steel?
      A. Weight reduction for movable structures
      - movable roofs
      - movable sheds
      - movable openings
      - gates

      Main benefit: savings on mechanical system
      Other benefits: MEP, foundations, maintenance

      B. Weight reduction for bridge decks
      - stay cable bridges
      - suspended bridges

      Main benefit: savings on cable section

C. **Weight reduction for OIL&GAS steel structures:**
   - GTG steel modulus

   Main benefits: transportation, foundations, equipment for lifting and movement

D. **Weight reduction for large structures to be lifted during erection phase:**
   - roof to be erected by strand jack system

E. **In general:**
   - Weight reduction due to foundational issues.
   - Shape reduction (mainly at structural nodes) for architectural and functional requirements.
   - Customized solution where commercial products as bars and cables are too expensive or not acceptable for architectural requirements
   - Special equipment for liftings (example: crane booms or launching noose)

c. What is the highest-strength material used by Cimolai?
   - S690 [fy = 690 Mpa] splice plates for bolted connections
   - S690 [fy = 690 Mpa] welded structures
   - S890 [fy = 890 Mpa] welded structures (main booms of cranes)

3. Which of the following shapes or materials do you think should be offered in high-strength steels?
   a. Wide flange shapes no, we can build built-up sections from plates
   b. Plate steel products already available
   c. HSS no
   d. Pipes no, we can build hollow section by cold forming process starting from plates
   e. Bars or Rods in this case we'd rather consider commercial product as MACALLOY® bars or steel grade ASTM A325/A490
   f. Headed Studs in this case we always use commercial products as NELSON® studs
   g. Angles no
   h. Channels no
   i. Others ---

4. Indicate in which of the following applications you believe high-strength steel Fy>65 KSI would be desirable
   a. Trusses yes, but for tension elements only
   b. Steel Columns no
   c. Plate Girders no
   d. Build-up Shapes yes, but for tension components only
   e. Steel Framing Members no
   f. Braces in Braced frames yes, but for tension elements only
   g. Plate girders in Bridge Applications yes, for stay cable bridges and suspended bridges
   h. Connection Gusset plates yes, for high-stressed structural nodes
   i. Other Connection material pins and splice plates for bolted connections
   j. Steel H Piles no
   k. Steel Pipe Piles no

5. Which type of Structures do you think could use high-strength Steel? FY>65 KSI
   a. High-Rise Buildings yes (*)
   b. Airports/Hangars yes (*)

   c. Arenas/stadiums yes (*)
   d. Convention Centers yes (*)
   e. Theaters yes (*)
   f. Bridges yes (stay cable bridge and suspended bridge)
   g. Heavy Industrial Facilities yes (GTG steel modules)
   h. Special equipment for lifting yes
   i. Movable structures yes

   (*) only for tension elements and high-stressed structural nodes

6. What current impediments exist towards the use of high-strength steel?
   a. Cost information
      Consider these differences in % starting from steel grade S355:
      - S460 +10%
      - S690 +60%
   b. Market Availability
      Only plates.
      Minimum thk 12 mm
      Maximum thk 200 mm but it is recommended to check with steel suppliers on case-by-case scenario
   c. Building Codes
      Item well covered by European Codes, therefore no prevention for the design.
      Especially to be consider code EN 1993-1-12 - Additional rules for the extension of EN 1993 up to steel grades S700.
   d. General familiarity with material and its properties
      This is generally true.
      For this reason, clients and designers have to promote partnerships with well-qualified steel fabricators also during design process.
      High-strength steel means high-qualified steel fabricator.
   e. Limitations on use
      No limitation in general.
      Clearly it depends on cost benefit analysis.
      Evaluate benefits where FIRE DESIGN is required.
      Evaluate benefits where SEISMIC DESIGN is required.
   f. ASTM Specifications
      No experience about this item
   g. Fabrication issues such as weldability, fracture, etc.
      See preliminary document developed by our QC department.

Appendix 5: Details on Rotational Capacity of Beams

			σ_y (ksi)				Flange	Web	Classification				
Author	**Specimen**	**Moment**	**Flange**	**Web**	$b_f/2t_f$	h/t_w	$0.38\sqrt{(E/F_y)}$	$3.76\sqrt{(E/F_y)}$	**Flange**	**Web**	R	L_b/r_y	**Limit State**
Adams et al (1965)	HT-28	Gradient	59	67	7.85	32.77	8.44	78.48	C	C	6.3	34.98	LB
Adams et al (1965)	HT-43	Gradient	59	67	7.85	32.77	8.44	78.48	C	C	6.0	22.91	LB
Adams et al (1965)	HT-52	Gradient	59	67	7.85	32.77	8.44	78.48	C	C	4.2	72.30	LTB
Adams et al (1965)	HT-29	Uniform	59	62	6.72	37.99	8.40	81.27	C	C	5.7	35.01	LB
Adams et al (1965)	HT-30	Uniform	59	62	6.72	37.99	8.40	81.27	C	C	2.9	40.00	LB
Adams et al (1965)	HT-31	Uniform	59	62	6.72	37.99	8.40	81.27	C	C	6.9	30.01	LB
Adams et al (1965)	HT-36	Uniform	59	62	6.72	37.99	8.40	81.27	C	C	1.5	45.00	LB
Adams et al (1965)	HT-37	Uniform	59	62	6.72	37.99	8.40	81.27	C	C	3.4	37.48	LB
Adams et al (1965)	HT-41	Uniform	59	62	6.72	37.99	8.40	81.27	C	C	10.4	25.02	LB
Adams et al (1965)	HT-38	Uniform	59	62	6.72	37.99	8.40	81.27	C	C	4.7	45.00	LB
Lukey & Adams (1969)	A-1	Gradient	41	45	9.42	30.78	10.07	95.66	C	C	11.8	35.0	LB
Lukey & Adams (1969)	A-2	Gradient	41	45	8.17	30.78	10.07	95.66	C	C	13.6	35.0	LB
Lukey & Adams (1969)	B-1	Gradient	54	57	9.73	43.07	8.80	84.50	NC	C	2.9	35.0	LB
Lukey & Adams (1969)	B-2	Gradient	54	57	6.99	43.07	8.80	84.50	C	C	10.4	35.0	LB
Lukey & Adams (1969)	B-3	Gradient	54	57	8.13	43.07	8.80	84.50	C	C	6.7	35.0	LB
Lukey & Adams (1969)	B-4	Gradient	54	57	8.91	43.07	8.80	84.50	NC	C	3.4	35.0	LB
Lukey & Adams (1969)	B-5	Gradient	54	57	9.16	43.07	8.80	84.50	NC	C	3.2	35.0	LB
Lukey & Adams (1969)	C-1	Gradient	54	51	9.68	52.50	8.80	89.63	NC	C	4.2	35.0	LB
Lukey & Adams (1969)	C-2	Gradient	54	51	6.99	52.50	8.80	89.63	C	C	13.7	35.0	LB
Lukey & Adams (1969)	C-3	Gradient	54	51	8.18	52.50	8.80	89.63	C	C	8	35.0	LB
Lukey & Adams (1969)	C-4	Gradient	54	51	8.91	52.50	8.80	89.63	NC	C	4.2	35.0	LB
Lukey & Adams (1969)	C-5	Gradient	54	51	8.54	52.50	8.80	89.63	C	C	6.5	35.0	LB
McDermott (1969)	Beam A	Gradient	120	116	3.23	33.98	5.91	59.44	C	C	2.0	-	
McDermott (1969)	Beam B	Gradient	119	116	4.82	13.89	5.93	59.44	C	C	2.1	-	
McDermott (1969)	1	Uniform	125	125	12.30	25.10	5.79	57.27	NC	C	-	-	
McDermott (1969)	2	Uniform	128	128	8.01	21.30	5.72	56.58	NC	C	-	-	

McDermott (1969)	3	Uniform	115	115	6.04	19.60	6.03	59.70	NC	C	3.8	-	
McDermott (1969)	4	Uniform	118	118	5.01	19.84	5.96	58.93	C	C	6.8	-	
McDermott (1969)	5	Uniform	119	119	4.01	19.58	5.93	58.68	C	C	5.1	-	
McDermott (1969)	6	Uniform	120	116	3.20	34.47	5.91	59.44	C	C	1.0	-	
McDermott (1969)	7	Uniform	119	116	4.78	33.13	5.93	59.44	C	C	1.8	-	
Iyengar et al (1976)	W12X19-1	Gradient	65	65	5.73	68.81	8.03	79.42	C	C	3.1	37.5	LB
Iyengar et al (1976)	W12X19-2	Uniform	65	65	5.73	68.81	8.03	79.42	C	C	4.8	37.5	LB
Kemp (1985)	1C1	Gradient	49.2	51.9	9.26	32.90	9.23	88.88	C	C	1.4	54.3	LB
Kemp (1985)	2F4	Gradient	41.3	47.7	6.84	31.60	10.07	92.71	C	C	3.3	54	LTB
Kemp (1985)	3F12	Gradient	48.1	56.2	7.48	46.60	9.33	85.41	C	C	1.1	87.8	LTB
Kemp (1985)	4S5	Gradient	49.3	51.9	8.71	32.40	9.22	88.88	C	C	7.6	27	LB
Kemp (1985)	5S7	Gradient	42.6	43.5	8.81	32.40	9.91	97.08	C	C	7.4	27.2	LB
Kemp (1985)	6S6	Gradient	41.7	47.7	6.50	31.90	10.02	92.71	C	C	7.0	27.9	LB
Kemp (1985)	7W3	Gradient	54.3	58.4	8.70	60.70	8.78	83.79	C	C	0.7	60.9	LB
Kemp (1985)	8W9	Gradient	45.4	43.5	7.83	16.80	9.60	97.08	C	C	4.2	47.7	LB
Kuhlman (1989)	1	Gradient	34.2	31.5	8.81	55.60	11.06	114.15	C	C	8	67.2	LB
Kuhlman (1989)	2	Gradient	34.2	31.5	9.38	55.60	11.06	114.15	C	C	7	72.9	LB
Kuhlman (1989)	3	Gradient	65.1	31.5	9.41	50.36	8.02	114.15	NC	C	1	78.7	LB
Kuhlman (1989)	4	Gradient	41.6	37.7	10.00	43.50	10.03	104.28	C	C	12.7	50.0	LB
Kuhlman (1989)	5	Gradient	41.6	36.5	10.00	51.60	10.03	105.93	C	C	8.6	51.9	LB
Kuhlman (1989)	6	Gradient	41.6	37.6	10.00	64.75	10.03	104.48	C	C	4.6	53.5	LB
Kuhlman (1989)	7	Gradient	41.6	36.5	10.00	56.00	10.03	105.93	C	C	13.5	35.4	LB
Kuhlman (1989)	8	Gradient	41.6	36.5	10.00	56.00	10.03	105.93	C	C	11.5	43.2	LB
Kuhlman (1989)	9	Gradient	41.6	36.5	10.00	55.80	10.03	105.93	C	C	7.8	51.1	LB
Kuhlman (1989)	10	Gradient	34.2	31.5	10.63	55.80	11.06	114.15	C	C	5.5	55.2	LB
Kuhlman (1989)	11	Gradient	34.2	31.5	11.38	50.55	11.06	114.15	NC	C	8.9	59.1	LB
Kuhlman (1989)	12	Gradient	34.2	31.5	11.88	50.55	11.06	114.15	NC	C	7.6	66.9	LB
Kuhlman (1989)	13	Gradient	48.3	102.8	6.91	43.56	9.31	63.15	C	C	5.1	59.1	LB
Kuhlman (1989)	14	Gradient	48.3	102.8	7.50	43.45	9.31	63.15	C	C	3.8	63.0	LB
Kuhlman (1989)	15	Gradient	48.3	102.8	7.69	43.13	9.31	63.15	C	C	3.6	69.1	LB
Kuhlman (1989)	16	Gradient	48.3	102.8	7.84	27.02	9.31	63.15	C	C	10.5	45.4	LB

Kuhlman (1989)	17	Gradient	48.3	102.8	8.00	36.36	9.31	63.15	C	C	9.5	43.4	LB
Kuhlman (1989)	18	Gradient	48.3	102.8	8.05	48.91	9.31	63.15	C	C	6.6	41.3	LB
Kuhlman (1989)	19	Gradient	48.3	50.6	8.00	46.33	9.31	90.01	C	C	12	39.4	LB
Kuhlman (1989)	20	Gradient	48.3	50.6	8.00	46.50	9.31	90.01	C	C	8.7	47.3	LB
Kuhlman (1989)	21	Gradient	48.3	50.6	8.00	46.50	9.31	90.01	C	C	7.2	55.2	LB
Kuhlman (1989)	22	Gradient	48.3	50.6	8.50	46.50	9.31	90.01	C	C	10	47.4	LB
Kuhlman (1989)	23	Gradient	48.3	50.6	8.88	46.40	9.31	90.01	C	C	6.7	49.2	LB
Kuhlman (1989)	24	Gradient	48.3	50.6	9.31	46.43	9.31	90.01	C	C	5.2	53.1	LB
Green (2002)	1	Gradient	80	80	2.98	28.27	7.23	71.59	C	C	4.6	36.22	Tens. Fract.
Green (2002)	2	Gradient	80	80	5.88	45.43	7.23	71.59	C	C	1.4	36.22	LB
Green (2002)	3	Gradient	80	80	5.83	29.42	7.23	71.59	C	C	7.1	36.22	LB
Green (2002)	4	Gradient	80	80	6.05	53.90	7.23	71.59	C	C	1.2	36.22	LB
Green (2002)	4A	Gradient	80	80	6.09	54.29	7.23	71.59	C	C	2.9	36.22	LTB
Green (2002)	5	Gradient	36	36	4.51	39.85	10.79	106.72	C	C	9.7	36.22	LB
Green (2002)	6	Gradient	80	80	9.24	28.27	7.23	71.59	NC	C	3.4	36.22	LB
Green (2002)	7	Uniform	80	80	6.17	29.85	7.23	71.59	C	C	3.5	17.95	LB
Green (2002)	8	Uniform	80	80	6.09	52.67	7.23	71.59	C	C	1.3	12.01	LB
Green (2002)	9	Uniform	36	36	4.60	39.09	10.79	106.72	C	C	5.2	12.01	LB
Green (2002)	10	Cyclic	80	80	6.22	29.00	5.71	46.65	C	SC	2.2	20	LB
Green (2002)	11	Cyclic	36	36	5.83	19.37	8.51	69.54	SC	SC	7.2	20	LB
Green (2002)	12	Cyclic	80	80	9.24	28.95	5.71	46.65	NC	SC	1.2	20	LB
Dexter et al (2002)	1	Gradient	70	70	5.33	34.67	7.73	76.53	C	C	3.9	60	LTB
Hartnagel (2003)	1	Gradient	83.35	69.83	6.67	64.00	7.09	76.62	C	C	1.44	58.5	LB
Hartnagel (2003)	2	Gradient	83.35	69.83	6.00	76.00	7.09	76.62	C	C	1.34	58.5	LB
Hartnagel (2003)	3	Gradient	83.35	69.83	7.33	84.00	7.09	76.62	NC	NC	-		
Hartnagel (2003)	4	Gradient	83.35	69.83	7.33	100.00	7.09	76.62	NC	NC	-		