*Standing Committee on Fabrication and Inspection of Metal Structures (AFH70)*
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# A Century of Fabrication and Inspection of Metal Structures

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## INTRODUCTION

Bridges have always played an integral role in the transportation network. To serve their critical purpose, bridges must not only be adequately designed, but must be fabricated and inspected properly. The Transportation Research Board (TRB) Standing Committee on Fabrication and Inspection of Metal Structures (AFH70) plays a critical role in facilitating research on the topics of fabrication and inspection of metal transportation structures. The majority of these structures are steel girder bridges, but the committee's involvement also includes areas such as orthotropic decks, ancillary traffic structures, expansion joints, etc. Since its inception in 1970, the Standing Committee on Fabrication and Inspection of Metal Structures, has shaped the evolution of fabrication to make metal structures safer, more reliable, and more cost-effective. Many advances within metal structures fabrication have originated from the committee through the development of research needs statements (RNSs). This paper provides a brief historical perspective of how the materials, fabrication, and inspection of metal structures has advanced over the last century as a result of this research. Where applicable, references will be made to research projects which originated from the Standing Committee on Fabrication and Inspection of Metal Structures as RNSs and were funded through the National Cooperative Highway Research Program (NCHRP).

## MATERIAL ADVANCES

Over the last century, significant advances have occurred in steel material specifications, many of which affected bridge fabrication practices. The first steel specifications were developed by the American Society for Testing and Materials (ASTM). In 1914, the American Association of State Highway Officials (AASHO), which later became the Association of State Highway and Transportation Officials (AASHTO), was formed and began to maintain its own material specifications, but were generally identical to those in ASTM. For this paper, steels will be referred to by their ASTM specifications due to their familiarity within the steel bridge community.

In the early 1900s, many steel bridges in the United States were constructed using ASTM A7, ASTM A8, and ASTM A94 steel (1, 2, 3). These steels had yield strengths ranging from approximately 32 to 55 ksi. By 1966, all three specifications had been withdrawn, and bridges were predominantly being constructed with ASTM A36, ASTM A572 Grade 50, and ASTM A588 with yield strengths of 36, 50, and 50 ksi, respectively (4, 5, 6). These specifications were introduced from 1960 to 1968 with the intent to produce steels with better weldability and higher strength. ASTM A588, typically known as “weathering steel,” was also the first corrosion-resistant alloyed steel commonly used by the bridge community. In 1964, ASTM A514 steel was introduced to offer a higher yield strength of 100 ksi (7). While A514 is weldable, there have been cases of delayed hydrogen cracking, often within 48 hours after welding, caused by diffusion of hydrogen into the heat affected zone (HAZ) (8).

The tragic collapse of the Silver Bridge in 1967 led to numerous specification changes. The eyebar chain suspension bridge carried traffic over the Ohio River between West Virginia and Ohio. A small stress corrosion crack in one of the eyebars led to a brittle fracture of the eyebar, resulting in complete collapse of the bridge and 46 fatalities (9). The investigation determined the eyebar was made from steel having extremely low fracture toughness. The collapse led to the creation of the fracture control plan, which had two aspects: improved control of fabrication quality and minimum toughness specifications. Numerous studies were conducted to determine the toughness values of bridge steels at the time and to develop specifications for minimum toughness values to safeguard against brittle fracture.

In 1974, AASHTO first set minimum Charpy V-notch (CVN) requirements for bridge steels (10). Originally, the specifications required a minimum CVN value of 15 ft-lbs, based on experience with the Liberty ship failures; the value applied to all load carrying members without distinction. The specifications were later revised to a minimum CVN value of 25 ft-lbs for fracture critical members and retained the 15 ft-lbs minimum value for non-fracture critical members. Minimum CVN requirements were also increased for higher strength and thicker steels. In addition, temperature zones were created in bridge design specifications to correlate fracture with minimum service temperatures at a bridge location. As a result of the Silver Bridge collapse and these new specifications, CVN testing of bridge steels became a simple alternative to traditional fracture toughness testing. Prior to these specifications, CVN testing had been conducted, but only on an ad-hoc basis with no required values.

ASTM A709: Standard Specification for Structural Steel for Bridges, was first published in 1974 (11). The specification unified all of the bridge steel plate ASTM specifications, making it easier for bridge engineers and owners to properly select and specify material. As part of ASTM A709, bridge steels are required to be weldable and are subjected to CVN testing.

In the early 1990s, NCHRP Project 12-31: Notch Variability in Bridge Steel Plates examined the variability of CVN values of bridge steels (12). Research results showed bridge steel plates typically have two types of CVN variability: longitudinal scatter in the plate rolling direction and localized scatter. To address longitudinal variability, it was recommended that CVN samples be obtained from each end of a plate. To address localized CVN scatter, the individual minimum CVN value of a three sample dataset was increased from 67% to 80% of the required average CVN value. Both recommendations are still in place within ASTM A709 to ensure consistent toughness.

In the 1990s, research initiated between the Federal Highway Administration (FHWA), United States Navy, and the American Iron and Steel Institute (AISI) to develop a high performance steel (HPS) to combat the propensity for cracking in ASTM A514 steel, while meeting the relatively new CVN requirements (8, 13). HPS was developed to be easily weldable without requiring excessive or costly weld-process controls. HPS also demonstrated an improved fracture toughness compared to traditional bridge steels. Another added benefit of HPS was that it qualified as a weathering steel, giving it additional inherent corrosion resistance, which is denoted by the “W” in the name of the steel grade. The first high performance steel was HPS

70W having a yield strength of 70 ksi. In 2000, this steel replaced the conventional grade 70 steel within ASTM A709. In 2001 and 2004, HPS 50W and HPS 100W, respectively, were also incorporated into the ASTM A709 specifications.

In the early 2000s, FHWA began to consider using stainless steel for bridges in highly corrosive environments. Weathering steel had performed poorly in corrosive environments such as marine coastal, frequent rainfall or fog, industrial, and areas with heavy deicing salt application. As a result, FHWA issued a technical advisory in 1989 recommending against its use in these areas (14). This led to the use of ASTM A1010, a stainless steel with at least 10.5% chromium by weight (15). Research has shown ASTM A1010 has a corrosion resistance 4-10 times better than weathering steel (16, 17). Upon increased usage on bridges in the United States, ASTM A1010 was incorporated into the ASTM A709 specifications as Grade 50CR in 2017.

As of 2019, the current ASTM A709 specification contains seven steel grades for structural plates: 36, 50, 50W, HPS 50W, 50CR, HPS 70W, and HPS 100W. Over the course of many revisions, bridge steels have become much more weldable and possess increased fracture toughness. The presence of inherent corrosion resistance has also been a constant theme, with weathering steels being used for many years and the recent introduction of stainless steel for bridges.

## FABRICATION ADVANCES

### From Riveted to Welded Construction

In the early 1900s, riveting was the predominant method of steel plate girder fabrication, both in the shop and during field erection. Rivets were then used to mechanically fasten built-up I-shaped members, fabricated from plates for the web and flanges, and angles at their intersections. High-strength bolts became a popular replacement for rivets in the late 1950s, and the use of riveting declined due to higher labor costs and the lengthy time of installation. The widespread use of riveted construction did not extend past the early 1960s when steels with poor weldability were removed from the specification. In 2002, rivets were completely removed from the AASHTO Standard Specifications (18).

The use of high-strength bolts for bridges were introduced between 1947-1949, when the ASTM A307 and ASTM A325 specifications were issued; these specifications described bolts having an ultimate tensile strength of 90 ksi and 120 ksi, respectively (19, 20). The use of bolts expanded after 1957 when the AASHO Standard Specification stated that high-strength bolts were an acceptable substitute for rivets (21). The ASTM A490 specification for bolts with an ultimate tensile strength of 150 ksi was issued in 1964 (22). These bolts were permanently incorporated into the AASHTO specifications in 1974 (23).

The first bridge in the United States with welded construction was built in 1928 (24). Although a railroad bridge, it served as an example for future vehicular bridges to follow suit. During the same year, the American Welding Society (AWS) published its first structural welding code (25). In 1936, AWS published its first bridge welding code (26). These two documents eventually merged into a single specification intended for both buildings and bridges when AWS D1.1 was first published in 1972 (27). An AASHTO supplement specific to bridge welding was then published in 1974, containing requirements when welding ASTM A514 steel, along with enhanced inspection, qualification, and reporting requirements (28). In 1988, AWS and AASHTO published a cooperative specification on bridge welding entitled AASHTO/AWS D1.5 Bridge Welding Code, which is still in use today (29, 30).

In addition to the aforementioned material specification changes, the 1968 collapse of the Silver Bridge also led to fabrication changes within bridge welding specifications. As part of the fracture control plan, the importance of crack control during fabrication was emphasized. D1.5 provides specifications to limit the potential for defects, and also includes inspection requirements during fabrication to ensure critical defects are identified and repaired before being placed in service. D1.5 also specifies greater CVN values in weld metal due to the presence of discontinuities and tensile residual stresses within welds.

In the 2010s, research initiated on the HAZ of bridge welds. The HAZ generally has variable fracture toughness, but there are no specifications on minimum CVN values of HAZ's. NCHRP 10-95 and NCHRP10-95A: Toughness Requirements for Heat-Affected Zones of Welded Structural Steels for Highway Bridges intends to determine the minimum toughness requirements. NCHRP 10-95 concluded cooling rate, weld speed, preheat/interpass temperature, and plate thickness have a substantial impact on the HAZ microstructure (31). NCHRP 10-95A is a current project tasked with determining additional factors that affect HAZ toughness and developing CVN requirements for HAZ within AASHTO and AWS specifications.

Currently, the large majority of steel bridges within the United States are built using welded construction; riveted construction is only used in rare cases for repair and rehabilitation of historic structures. This transition in bridge construction methods followed closely the evolution of using more weldable steels. High-strength bolts helped to bridge the gap from rivets to welding by providing a non-welding option which was stronger and more consistent than rivets. D1.5 has also provided specifications which have been successful in minimizing the potential for brittle fracture to occur.

**Advanced Fabrication Techniques**

Historically, steel bridge fabrication required significant manual effort for layout and marking. In the early 2000s a push was made to incorporate automated processes to reduce the time, error, and inefficiencies of manual layout. This resulted in three different marking processes that are currently used: milling, plasma marking, and laser marking (32). Each process uses computer numerical control (CNC) programming to define coordinates related to piece boundaries, welding requirements, etc. Milling uses a drill head to remove small amounts of material to create markings and can mark at approximately 160 inches per minute. Plasma marking uses a plasma arc to inscribe markings approaching 300 inches per minute. Finally, laser marking uses a dark oxidation marking which can be removed during the blast cleaning process, and can mark up to 410 inches per minute. Each system represents an improvement in efficiency and a decrease in likelihood of human errors.

After marking, steel bridge fabrication requires components to be cut from plate material prior to welding. Flame cutting with oxy-acetylene fuel (oxy-fuel) has been used since the early 1900s and can cut through steel up to 12 inches thick. One alternative method was introduced in the late 1950's and employs a high pressure, fine diameter water jet. This method gained traction in the 1980's when the cutting ability was improved by adding small amounts of abrasives, such as garnet or aluminum oxide. (33). Modern abrasive water jet machines are typically CNC controlled, with a water speed of approximately 2500 feet per second and can cut through steel up to 3 inches thick. Additional benefits include no distortion due to heat, no required slag removal, minimal burrs, and reduced costs relative to oxy-fuel cutting.

Plasma cutting, which uses a plasma arc to melt through steel, was developed in the 1970s and 1980s (34). Relative to oxy-fuel cutting, plasma cutting is faster and is advantageous for thinner material, tighter radii, or other geometrically challenging shapes. Cutting thicknesses are limited to a maximum of approximately 1 inch (35). Because plasma cutting seemed promising for use on bridges, NCHRP 10-40: Plasma Arc Cutting of Bridge Steels was initiated. The project was completed in 1995 and produced a user's guide for plasma arc cutting in bridge fabrication, evaluated current plasma cutting technologies, and evaluated the material, mechanical, weldability, and fatigue properties of cut edges using this process (36). Additional research by FHWA found that open bolt holes fabricated using plasma cutting had a significantly lower fatigue life compared to the standard practice of drilling or punching (37). The study recommended against using plasma cutting for bolt hole fabrication on fracture-critical members and potentially on primary load-carrying members.

Laser cutting was first introduced to metals in the early 1970s, but has not yet been widely adopted for steel bridge fabrication. Benefits of laser cutting include small HAZ regions, faster cutting times, and small kerfs relative to oxy-fuel (8). Laser cutting excels on thinner materials, though the maximum thickness of 1 inch prevents widespread adoption since many bridge members require thicker plates. Additionally, capital cost is greater for laser cutting machines relative to plasma or water jet machines (38).

Automated fabrication, including welding processes, has received much attention over the years. The first application of automated welding occurred in the early 1960s in the automotive industry with the use of robotic spot welders (39). The first use of robotic arc welders was in the mid 1970s and required two passes of the robotic arm to 'learn' the geometry of the weld pass, and then subsequently perform the weld from the learned route (40). Modern bridge girders are fabricated using semi-automated submerged arc welding for higher-deposition longitudinal welds, typical of web-to-flange welds, and semi-automated gas metal arc welding for attaching components such as stiffeners and attachment plates. The automation process allows for more precise weld sizes as well as the use of double-beveled complete joint penetration (CJP) welds that requires significantly less weld material and time compared to a manually welded single-beveled CJP weld (41).

**Electroslag Welding**

The original electroslag welding (ESW) process was introduced in the United States in 1959 and was widely used on steel bridges due to its high efficiency (42). However, in the 1970's after years of use, welding defects and inadequate fracture toughness issues began to arise. This was especially apparent when a 10-foot long crack was found on the I-79 Neville Island Bridge in 1977 (43). The investigation showed the fracture initiated from an ESW repair, and that large grain structures within ESW microstructure had produced low CVN values. This prompted FHWA to issue a moratorium on ESW for tension members, effectively eliminating its use for bridges (44, 45).

NCHRP Project 10-10: Acceptance Criteria for Electroslag Weldments in Bridges examined the weld procedures and performance of ESW in the late 1970's (46). Because the research spanned from before the Neville Island fracture to after the FHWA moratorium, the results were not incorporated into specifications. Nevertheless, results showed that when made using controlled procedures under production conditions, ESW was suitable for strength and fatigue in tension members made with steels having a yield stress up to 50 ksi. The study recommended against using ESW in the lowest AASHTO service temperature zone because of potential low CVN values.

Beginning in the 1980s, FHWA initiated numerous research studies to show the ESW process could be improved by reducing the gap between the plates being joined, resulting in the narrow gap improved electroslag welding (NGI-ESW) process (45). This revised process allowed for increased vertical travel speed of the weld and incorporated additional alloys in the electrodes to improve the grain structure (47). The NGI-ESW process was able to achieve improved CVN values in both the weld metal and HAZ. As a result, FHWA rescinded the moratorium in 2000 and allowed NGI-ESW to be used for most non-fracture critical tension members (47).

In 2010, the NGI-ESW process was included in D1.5 and was called, "electroslag welding – narrow gap" (ESW-NG) (48). These specifications were consistent with the parameters developed through FHWA research to produce adequate toughness. The code does not allow ESW-NG on fracture critical members or on HPS, stating conservativism and lack of research. Around the same time, a study was completed examining butt-splice connections made from HPS 70W welded using ESW-NG (49). It demonstrated the fatigue performance of HPS 70W specimens welded with ESW-NG was at least as good as those welded with submerged arc welding and easily satisfied the AASHTO fatigue provisions.

Another research study through FHWA in 2017 evaluated the mechanical properties of HPS 70W qualification plates welded using ESW-NG (50). Although the majority of the tests performed well, some all-weld tension specimens failed prematurely, suggesting there may have been defects present in the specimens. Accordingly, the study indicated ESW-NG could not yet be pre-qualified for HPS 70W per the D1.5 specifications.

Currently, the status of ESW-NG remains unchanged since FHWA rescinded its moratorium and the process was integrated into D1.5. The AFH70 committee has an active RNS proposing to examine the suitability of ESW-NG for welding HPS and for use in fracture critical butt splices. The proposed research would provide design and fabrication recommendations for using ESW-NG in these applications.

**Orthotropic Decks**

The first orthotropic deck system was constructed in Germany in 1936 and has since been successfully used for many applications worldwide (51). Although its use in the United States has not been prevalent, it has been used when circumstances are advantageous: long span structures where dead load minimization is critical, box girder elements that require stiffening, and re-decking existing bridges for rapid construction.

When orthotropic decks were first introduced in the United States in the 1950s, many experienced poor performance (52). Some initial problems were due to deck plates having been designed too thin to reduce weight, and thus were not adequate for supporting heavy wheel loads. Fatigue cracking was also observed in either the rib-to-deck (RD) weld or the rib-to-floor beam (RF) welds. Historically, the RD weld was specified as a partial penetration weld with an 80 percent minimum penetration without melt-through (53). Unfortunately, this requirement was difficult for fabricators to achieve. In the early 2000s, research studies indicated 60 percent penetration was sufficient to prevent fatigue cracking (54, 55). Experience also indicated minimizing the root gap size increases fatigue resistance and reduces melt-through potential. To address RF weld cracking, a study in 1992 showed cut-outs in the floor beams should be made as deep as possible to achieve better fatigue performance (56). More recent studies have investigated the fatigue performance and potential for robotic welding of these connections (57).

Currently, AASHTO requires a minimum thickness of 0.625 inches for orthotropic decks, a value chosen based on experience to prevent cracking in overly flexible decks (58). The specification also requires a partial penetration weld with 60 percent penetration for RD welds, with no melt-through, and placed with a 0.02 inches or less root gap prior to welding. Guidance is also provided on detailing the RF welds. The specifications were developed based on research to achieve increased, successful use of orthotropic decks within the United States.

**Ancillary Highway Structures**

In addition to steel bridge members, the Standing Committee on Fabrication and Inspection of Metal Structures has been involved in issues related ancillary highway structures, such as highway signs, luminaries, and traffic signals. While TRB has a Traffic Structures Subcommittee, AFF10(1), housed under the General Structures Committee, AFF10, a collaborative alignment exists with Fabrication and Inspection of Metal Structures. The cooperation between groups has resulted in their research needs statements leading to funded NCHRP research and positive impacts on the AASHTO LRFD Specification for Structural Supports for Highway Signs, Luminaries, and Traffic Signals (59).

In 1993, in response to a number of failures, NCHRP 10-38: Fatigue-Resistant Design of Cantilevered Signal, Sign, and Light Supports initiated, aiming to improve ancillary highway structure design and performance (60). Fatigue testing revealed the AASHTO Category D design curve was a reasonable lower bound for snug and fully tightened anchor rods, while most other fatigue sensitive details should be Category E or E'. The research also identified loading phenomena and equivalent static load ranges for fatigue design. Additional advances were made through NCHRP 17-10 and NCHRP 17-10(2): Structural Supports for Highway Signs, Luminaries, and Traffic Signals commencing in 1994 and 1999, respectively (61). The projects recommended base plate flatness tolerances, design guidelines for bent tubes, and a synthesis on fatigue-damaged support structure inspection, repair, and rehabilitation. In 2006, NCHRP 10-70: Cost-Effective Details for Highway Sign, Luminaire, and Traffic Signal Structures aimed to develop fatigue resistant ancillary structure connections through large-scale experimental testing and an analytical study (62). The research proposed to maintain an infinite-life design philosophy for new structures, while a finite-life assessment was introduced for existing structures.

**Modular Bridge Expansion Joints**

The Standing Committee on Fabrication and Inspection of Metal Structures has also been involved in the fabrication and inspection of modular bridge expansion joints, which permit a bridge superstructure to undergo large longitudinal movement due to expansion and contraction. In response to premature failures, extensive research and testing was conducted to ensure modular bridge expansion joints resist applied loads, exhibit adequate fatigue performance, and prevent water and debris leakage. In 1994, NCHRP Project 12-40: Fatigue Criteria for Modular Bridge Expansion Joints developed a performance-based specification and associated commentary to ensure adequate fatigue performance (63). Results found fatigue cracking was largely due to underestimating dynamic response, poor detail design and fabrication, and inadequate installation. Results provided fatigue criteria to improve modular bridge expansion joint performance. In 1998, NCHRP Project 10-52: Performance Tests for Modular Bridge Joints further improved joint performance through large-scale laboratory experimental testing (64). The study resulted in additional performance tests not previously addressed by the fatigue testing and design specifications. The resulting design, fabrication, installation, and construction inspection guidelines, in conjunction with the prequalification test methods and equipment, ensure modular bridge expansion joints have adequate in-service performance.

**Repair of Existing Structures**

Although much of the committee's fabrication involvement has focused on shop fabrication, the committee has been involved in initiating projects related to field repairs of steel bridges, including fatigue crack repairs, and heat straightening of damaged girders. One such project was NCHRP 12-27: Welded Repair of Cracks in Steel Bridge Members, initiated in 1984, to identify and evaluate welding methods for repairing cracked steel bridge members (65). Experimental testing and extensive surveys showed that welded repairs of fatigue cracks can provide satisfactory performance, which can lead to more cost effective strategies than bolted splice repairs. Some of these repair strategies were incorporated into the FHWA Manual for Repair and Retrofit of Fatigue Cracks in Steel Bridges originally published in 2005 (66). In 2003, NCHRP 10-63: Heat Straightening Repair of Damaged Steel Bridge Girders: Fatigue and Fracture Performance used experimental testing to conclude that a particular location on a girder could only undergo two damage and repair cycles without decreasing the girder's fatigue and fracture resistance (67). Recommendations from the research were incorporated into FHWA's guide on heat straightening damaged steel girders (68). In 2017, NCHRP 20-07: Maintenance Actions to Address Fatigue Cracking in Steel Bridge Structures was completed, which set forth guidelines and details on best practices for repair of fatigue cracks, as well as preventing further fatigue crack growth (69).

## INSPECTION ADVANCES

Nondestructive testing (NDT) methods have been used to perform weld inspection since welding replaced riveting for fabrication. The use of radiation and sonic energy to inspect welds was documented as early as the 1920s (70). While early bridge welding codes discussed the use of NDT for weld inspection, they provided little guidance on NDT procedures and inspection requirements. For instance, the only NDT requirement other than visual inspection in the 1956 edition of AWS D2.0 was that radiographic testing (RT) was required for butt welds in tension with thicknesses more than one inch. Engineers could also specify RT for other important highly stressed groove welds subject to tension, but there were no requirements on the RT procedure, technique, frequency of testing, or standards of acceptance. Other NDT methods such as magnetic particle testing (MT), dye penetration testing (PT), and ultrasonic testing (UT) could be specified by an engineer for weld inspection but no specific guidance was provided for these methods.

Conventional UT inspection procedure and requirements were added to AWS D2.0 in the 1969 edition (71). By 1974, the combined requirements of AWS D1.1 (72) and AASHTO Standard Specifications for Welding of Structural Steel Highway Bridge (28) resulted in RT, UT, and MT requirements, procedures, and acceptance criteria which were similar to those included in the current edition of AWS D1.5. According to NCHRP 10-13: Ultrasonic Measurement of Weld Flaw Size (73), the amplitude limits in the AWS conventional UT acceptance criteria were set to match the sensitivity of the AWS RT procedure which detects a two percent change in thickness. The author of the UT acceptance criteria stated that discontinuity size, shape, and position were also considered during its development (71).

Some of the recent advancements in nondestructive examination of steel bridge fabrication are phased array ultrasonic testing (PAUT) and digital RT. PAUT utilizes a multielement transducer and a time-delayed firing sequence in order to electronically sweep, scan, and focus ultrasonic waves. AWS D1.5 adopted PAUT in Annex K of the 2015 edition as an alternative method to replace conventional UT for inspection of CJP butt welds (30). PAUT advantages over conventional UT include a permanent recording of scan data, increase in scan speed, and reduced scan variability through automation; all of which aid in flaw detection and characterization if properly applied. Digital RT utilizes collector plates rather than film to directly acquire digital RT images. This advancement has improved the production speed and quality of RT images and allows for digital post-processing and digital record keeping of RT results.

In 2015, NCHRP 14-35: Acceptance Criteria of Complete Joint Penetration Steel Bridge Welds Evaluated Using Enhanced Ultrasonic Methods was initiated, and included extensive analytical and round robin testing programs to establish a critical flaw size considered rejectable for typical CJP welds in bridge fabrication (74). Proposed revisions to D1.5 included modifications to the scanning procedure and acceptance criteria, as well as additional requirements for technician qualification and calibration.

Future advances in ultrasonic imaging techniques, such as total focusing method (TFM) - full matrix capture (FMC) PAUT, will provide a means for more accurate flaw sizing and flaw characterization. This has the potential to provide an acceptance criteria based on fracture mechanics, which will improve steel bridge fabrication quality in the shop and fitness-for-service decision making for repair of discontinuities in existing bridges.

## LOOKING AHEAD

As it has done for many years, the Standing Committee on Fabrication and Inspection of Metal Structures will continue to identify potential innovative solutions for the fabrication and inspection of metal structures that could benefit from additional research. Three potential topic areas that the committee has begun discussing are the cost effective use of stainless steels, computer aided robotic welding, and additive manufacturing.

Stainless steels, such as ASTM A709 Grade 50CR and duplex stainless steels, have begun being used in steel bridge fabrication (75). Although stainless steels can offer substantial corrosion resistance, they are relatively expensive. Currently, it would be more cost effective to use stainless steel in targeted locations of a bridge that experience heavy corrosion, such as close to water or at beam ends under a joint. In less corrosive areas, traditional materials, like weathering steel, could be used to minimize cost. This durable, cost effective solution requires dissimilar metal welded joints between stainless and weathering steel. Research is required to ensure dissimilar welded joints could be fabricated and inspected properly, as well as have sufficient fatigue strength.

Computer aided robotic welding has begun gaining traction in the shipbuilding community. Over the last few years, the Naval Shipbuilding Research Program has conducted research on computer aided robotics welding focused on automated path planning algorithm development, including robot reachability, collision avoidance, and path optimization (76). The algorithm and use of CNC was able to significantly reduce the robot programming time by an average of 90%. It is feasible that similar technology could be used for steel bridges to achieve faster fabrication, better quality control, and increased worker safety.

Additive manufacturing (AM), commonly referred to as 3D-printing, is an emerging technology used to create components by joining materials through a layer-by-layer process. Historically, metallic AM was reserved for small-scale components made from expensive alloys;

however, global advances are happening at a rapid pace. Recent work has demonstrated fabrication of large-scale components is possible, including a large excavator arm produced by Oakridge National Laboratory and a stainless steel pedestrian bridge manufactured in the Netherlands measuring 41 feet in length (77, 78). While metallic AM technology and development are still not at the point of implementation for transportation-related structures, future possibilities exist that may benefit the industry. The Standing Committee on Fabrication and Inspection of Metal Structures will serve a key role to ensure AM fabrication and inspection technologies are properly implemented into steel bridge practice.

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2. Metal structures the range of metal structures that can be altered to get different properties: crystal and glass structure, structures of solutions and compounds, grain and phase boundaries, equilibrium shapes of grains and phases; examples. 3. Equilibrium constitution and phase diagrams how mixing elements to make an alloy can change their structure; examples: the leadâ€“tin, copperâ€“nickel and copperâ€“zinc alloy systems; examples.Â  There are so many different metals â€“ literally hundreds of them â€“ that it is impossible to remember them all.Â  Figure 1.1 shows a model of a nineteenth-century steam traction engine built in a home workshop from plans published in a well-known modellersâ€™ magazine. Sheet metal fabrication is using sheet metal to build metal machines and structures. It can be a complex process involving many different professionals. Businesses that specialize in sheet metal fabrication are commonly referred to as fabrication shops, or fab shops for short.Â  Fabrication is a very broad term which can be applied to lots of manufacturing processes among other things in simple terms the word itself means the making or making up of eg we've all heard of fabricating evidence so sheet metal fabrication is the making up of a structure using sheet metal components which is anything up to 10mm thick which. The structure ofthe book should have a word of explanation here. The approach that has been taken is to describe briefly what metals are and to discuss phase diagrams and the kinds ofstructures to be found in different and relevant alloys, before proceeding to deal with the practical application of this knowledge: the sampling and preparation. of samples for metallographic study.Â  Metals are an aggregation of atoms that, apart from mercury, are solid at room temperature. These atoms are held together by "metallic bonds" that result from sharing available elecÂ trons. A negative electron bond pervades the structure, and heat and electricity can be conÂ ducted through the metal by the free movement of electrons. Many advances within metal structures fabrication have originated from the committee through the development of research needs statements (RNSs). This paper provides a brief historical perspective of how the materials, fabrication, and inspection of metal structures has advanced over the last century as a result of this research. Where applicable, references will be made to research projects which originated from the Standing Committee on Fabrication and Inspection of Metal Structures as RNSs and were funded through the National Cooperative Highway Research Program (NCHRP). MATERIAL ADVANCES O