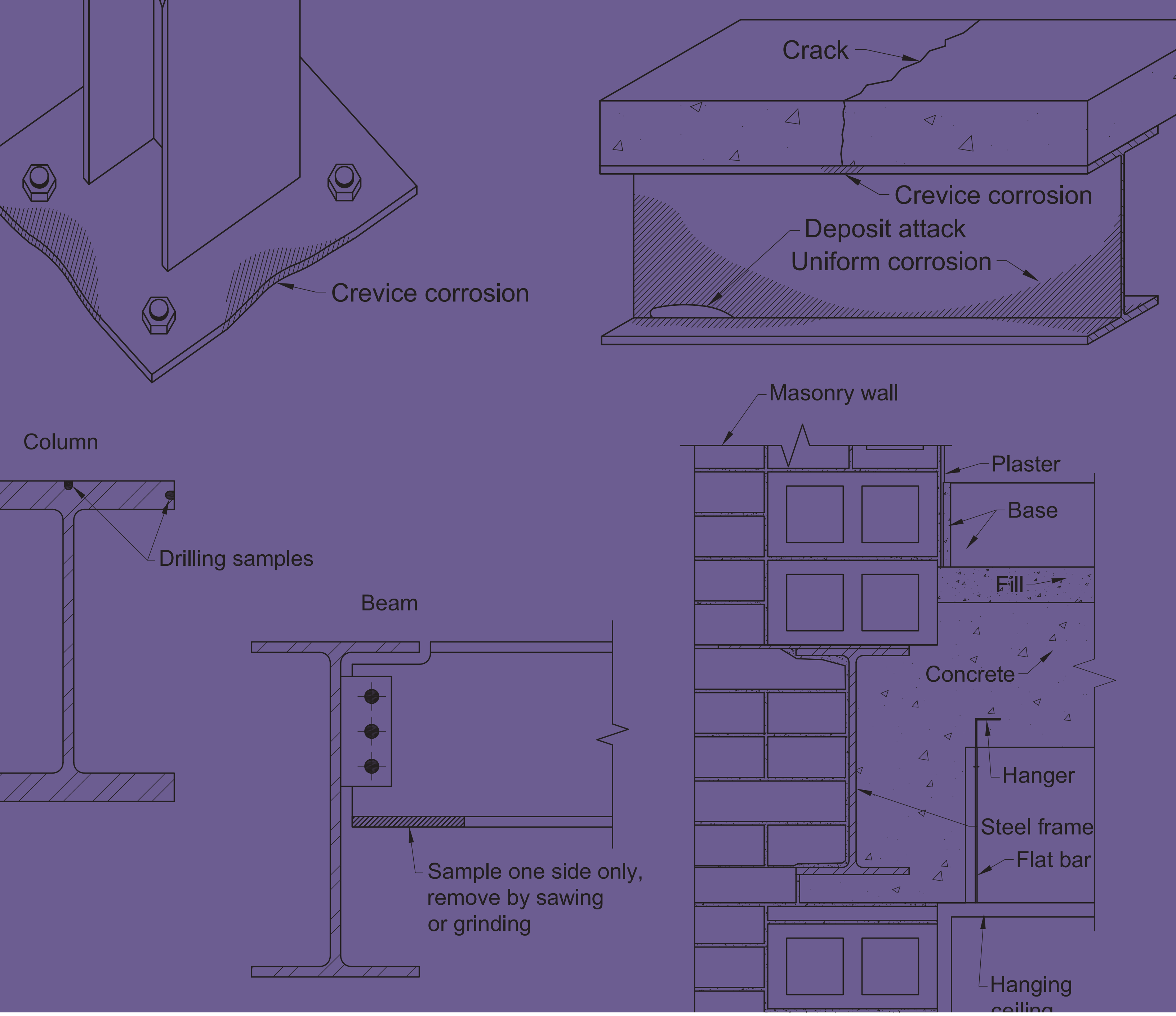

Design Guide 16

Assessment and Repair of Structural Steel in Existing Buildings

Smarter.
Stronger.
Steel.

Design Guide 16

Assessment and Repair of Structural Steel in Existing Buildings

Christopher M. Hewitt, SE, PE, PEng

American Institute of Steel Construction

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Preface

As buildings age, the need for effective structural evaluation and maintenance becomes increasingly critical. Originally conceived as a supplement to AISC Design Guide 15, *Rehabilitation and Retrofit*, this Design Guide aims to introduce engineers to the process and several tools for structural assessment and repair of existing steel buildings. The Design Guide discusses common considerations for initial assessments as well as methods for detailed inspection, evaluation, and nondestructive examination. It also explores several types of damage or deterioration commonly found in existing steel buildings and approaches for evaluating their structural effects. While no single document can address the full range of structural conditions that may be encountered in practice, reference is made to existing standards that provide structural acceptance criteria for several commonly encountered conditions. Finally, the Design Guide offers practical advice for planning and executing repairs.

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Chapter 1
Scope and Purpose

1.1 SCOPE

This Design Guide aims to assist structural engineers assessing the condition of existing steel building structures. The Design Guide is not mandatory and is intended to be used in conjunction with AISC Design Guide 15, *Rehabilitation and Retrofit*, 2nd Edition (Brockenbrough and Schuster, 2018); the *International Existing Building Code* (IEBC) (ICC, 2024b); ASCE/SEI 11, *Guideline for Structural Condition Assessment of Existing Buildings* (ASCE, 1999); and the AISC *Specification for Structural Steel Buildings*, ANSI/AISC 360-22 (AISC, 2022b), hereafter referred to as the AISC *Specification*.

AISC Design Guide 15 includes useful background and guidance on the rehabilitation and retrofit of structural steel members in buildings, strengthening of steel members, and determining the shape and composition of existing steel members. This Design Guide does not comprehensively address content already addressed by AISC Design Guide 15 or other AISC sources.

Since the publication of Design Guide 15, 2nd Edition, in 2018, the International Code Council, American Concrete Institute, and American Society of Civil Engineers have each issued updated standards that include recommendations for structural evaluation. In parallel, AISC developed the AISC *Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings*, ANSI/AISC 342-22 (AISC, 2022a). The committee writing AISC 342 accomplished considerable work that is useful for applications beyond high-seismic design. AISC commissioned this Design Guide to build upon the broadly applicable aspects of AISC 342, with more comprehensive guidance for evaluating existing structural steel members in all building types and conditions other than seismic. At AISC's request, portions of this text are adapted directly from AISC 342. This Design Guide also provides selected related details that are useful for repair planning.

1.2 LIMITATIONS

This Design Guide discusses structural condition assessment and evaluation approaches appropriate for common conditions encountered in steel buildings. Consistent with the scope of AISC Design Guide 15, this Design Guide does not address seismic evaluation and retrofit.

Steel and iron technology developed gradually but significantly since the late 1800s. Given the wide variety of existing materials, design approaches, loading, and exposure conditions in such buildings, a guidance document cannot address the full range of possible conditions. Encountered conditions may require significant application of engineering judgment.

1.3 DEFINITIONS

This Design Guide utilizes the following terms. Additional terms defined in the AISC *Specification* and the IEBC also apply.

Assessment. The process of gauging the quality, value, ability, or importance of something. *See also* structural condition assessment.

Evaluation. The process of making a judgment about values, numbers, or performance against acceptance criteria. An evaluation may be completed as part of an assessment.

Rehabilitation. Any work undertaken in an existing building, as described by the IEBC (according to IEBC).

Repair. The reconstruction, replacement, or renewal of any part of an existing building for the purpose of its maintenance or to correct damage (according to IEBC).

Structural condition assessment. A measure of the present structure's performance and capability for continued use by others under defined use conditions (according to ASCE/SEI 11).

Structural damage. An externally initiated impairment that may occur during construction or use. Examples include impact damage or fire damage.

Structural deterioration (or degradation). A reduction in section, material strength, or other impairment that occurs over time, often associated with corrosion or water damage.

Structural distress. An overload or other unanticipated structural impairment, often accompanied by unexpected deflections, deformation, racking, or settlement.

Work area. The portion or portions of a building consisting of all reconfigured spaces as indicated on the construction documents and as described by the IEBC.

Chapter 2
Assessment Planning

2.1 GENERAL

Structural condition assessment is a process used to determine an existing structure's state of health. It involves the investigation and analysis of various components of a structure to assess their composition, integrity, performance, and potential risks. Overall, structural condition assessment plays a crucial role in determining the structural integrity, durability, and safety of a building, enabling informed decision-making regarding repairs, renovations, or potential upgrades.

2.2 ASSESSMENT BASIS

An agreement between the owner and engineer should clearly describe the purpose of the condition assessment. Certain condition assessments are limited in scope and rigor to the requirements described by consensus standards. Others are tailored for a specific purpose, as defined by the owner or engineer. Common types of condition assessments include:

Property condition assessment (also called a due diligence evaluation). This type of assessment is intended to support property transfer in real estate transactions. The scope and approach are largely prescriptive and completed following ASTM E2018, *Standard Guide for Property Condition Assessments: Baseline Property Condition Assessment Process* (ASTM, 2024a). Typically performed by walkthrough surveys, document reviews, and interviews, this type of assessment is not intended to be technically exhaustive. It is used to screen for repairs or apparent physical deficiencies such as "conspicuous defects and material deferred maintenance." These assessments can assist owners in estimating the remaining useful life of components.

Post-event safety assessment. This type of assessment is intended to be a rapid and preliminary screening of buildings to identify apparent hazards and imminent danger to occupancy. The assessments are typically completed following a defined standard or guideline, such as ATC-20, *Procedures for Post-Earthquake Safety Evaluation of Buildings* (ATC, 2005), or ATC-45, *Field Manual: Safety Evaluation of Buildings after Windstorms and Floods*, (ATC, 2004). These standards are intended for rapid screening, and additional evaluation may be required after the initial assessment. Program training is typically required, which may result in a disaster worker classification, and many jurisdictions have "Good Samaritan" laws that limit the professional liability of qualified disaster workers.

Condition assessment. The broader term condition assessment applies to the general process of inspection, evaluation, and analysis used to investigate a structure's performance or existing state. Condition assessment can take various forms but is ideally used to evaluate a specific potential effect or concern for maintenance planning or to determine the structure's fitness for service. ASCE/SEI 11 (ASCE, 1999) and ASCE/SEI MOP 158 (ASCE, 2024) provide general guidance on structural condition assessment. The remainder of this Guide highlights strategies, tools, and tips to assist engineers in implementing this condition assessment process. Such condition assessments are often performed in two phases: preliminary and comprehensive. Different levels of detail and rigor are typical at each stage, and a preliminary assessment is often a necessary step in determining the comprehensive assessment approach.

2.3 BUILDING CODE REQUIREMENTS RELATED TO CONDITION ASSESSMENT AND REPAIR

2.3.1 Substantial Structural Damage per the *International Existing Building Code*

The IEBC defines a threshold for the extent of damage that must be considered *substantial structural damage*, which is used to classify the acceptable design basis of a repair program. It distinguishes damage that is permitted to be restored to pre-damaged conditions using the codes and standards in place at the time of construction from damage that requires reevaluation of the structure to current codes and standards. Special considerations in the IEBC apply to historic buildings, low-rise residential buildings, snow damage, or disproportionate earthquake damage.

2.3.2 Assessing Potentially Unsafe Conditions

The IEBC includes actions that must be undertaken when "dangerous" conditions are present. The IEBC defines dangerous conditions as those conditions where:

1. The building or structure has collapsed, has partially collapsed, has moved off its foundation, or lacks necessary support of the ground.
2. There exists a significant risk of collapse, detachment, or dislodgement of any portion, member, appurtenance, or ornamentation of the building under service loads.

Similarly, the IEBC defines the term "unsafe," which includes, among other things, conditions where:

> the structure or individual structural members meet the definition of "dangerous," or are otherwise dangerous to human life or the public welfare.

The IEBC does not define evaluation criteria for dangerous or unsafe conditions but identifies that these terms are included "to provide a basis for the code official to correct, either by repair or demolition, a condition that is hazardous to the health and welfare of individuals." The IEBC Commentary Section 302 clarifies that dangerous conditions are intended to relate to structural stability, whereas unsafe conditions are more general and may include nonstructural considerations.

While such determinations can only be made by the code official, in the author's experience, "dangerous" or "unsafe" structural conditions are usually ascribed to visible or apparent conditions that pose an immediate threat to the safety of the occupants or the public. These conditions may also include structures that are severely overstressed without apparent means of maintaining stability. Such conditions require immediate attention and action to mitigate the risk of injury or loss of life.

Other conditions encountered in an assessment may not pose an immediate safety risk but have the potential to do so over time. Such conditions may arise from structural deficiencies such as corrosion, cracking, or settlement, and may require ongoing monitoring and maintenance to ensure that they do not progress to a point where they become dangerous or unsafe.

If an engineer has reasonable doubt as to the stability or load-bearing capacity of a building, structure, or component for the expected loads, *International Building Code* (IBC) Section 1708 (ICC, 2024a) requires an engineering assessment of the affected structure or component by analysis or load testing. The assessment should evaluate the risk of instability, the potential for detachment of elevated components or pieces, and the risk of collapse under service loads. The assessment may be based on actual material properties, may consider the availability of alternative load paths, and may consider the load probability between the time of determination and the time of repair or retirement of the component.

Modern codes are reliability-based, such that there is a statistical measure of reliability between the code-prescribed loading (the required strength) and the limiting structural resistance (available strength) predicted by the AISC *Specification*. At an allowable strength demand, there is an implied safety factor of 1.5 in the predicted strengths in the AISC *Specification* under dead and live loading (at a live/dead ratio of 3). Additionally, the predicted strengths in the AISC *Specification* accommodate additional variability due to tolerances, residual stress, and variations in production using resistance factors. The resistance factors are also used to increase the target reliability of certain components that are considered more important or have a higher consequence of failure. For instance, the AISC *Specification* Commentary Section B3.1 details the targeted reliability of steel structures and their interaction with code-based forces and strengths, with steel connections having greater target reliability factors than steel members.

Project-specific knowledge of material strength, dimensions, and loading may allow for less variability in the predicted capacity of a steel structure than is contemplated in the AISC *Specification*. Comparing the expected loading against the nominal strength of a component will provide an initial understanding of the component's capacity to carry the expected load and may suggest a case for a more specific assessment of strength and reliability on a performance basis. For gravity, fluid, soil, and wind loads, capacity may be evaluated for members or structures considering the probability of failure on a performance basis following ASCE/SEI 7, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, Section 1.3.1.3 (ASCE, 2022). Using probabilistically determined load return intervals and considering the project's circumstances, the designer may establish an expected loading over a specific return interval for the affected component.

More comprehensive guidance on probability-based load criteria for structural design is presented in National Bureau of Standards (NBS) Special Publication 577 (Ellingwood et al., 1980).

2.4 BASIS DOCUMENTATION

A documented assessment, evaluation, or repair basis helps ensure that the engineer and building owner have a common understanding of the work and desired outcomes. The engineer should consider the intended remaining service life of the structure and repairs, the intended frequency of structural maintenance and inspection, and other factors critical to the use and reliability of the structure. Similarly, the basis of the original conditions and the effects of any prior modifications or repairs that have occurred during the life of the existing structure should be considered.

When repair or rehabilitation work is planned, the basis of compliance with the IEBC or other applicable building codes should be documented and coordinated with the entire project team. The extent of repair and size of the work area can affect the code compliance approach.

A basis document will often address the following:

- The work area and the scope.
- Description of the structure, including the date of construction, original design basis codes and standards, and prior occupancies or uses of the building.
- Description of any modifications that have occurred, including their extent and date.

- Structural assessment criteria (i.e., visual assessment, nondestructive assessment).
- Documentation of substantial structural damage in the work area, as defined by the IEBC (see Chapter 5).
- Design basis code criteria and basis of the repair or rehabilitation design, if required.
- Strategies to secure unsafe structural conditions, including shoring requirements.

Chapter 3
Condition Assessment

Condition assessments involve systematically reviewing existing structural conditions, which should be approached methodically and without bias, ideally following the scientific method and broadly including the following steps:

1. Make initial observations.
2. Pose the questions for testing.
3. Form a hypothesis or multiple hypotheses based on appropriate background facts, knowledge, and information.
4. Collect and analyze information needed to test the hypothesis. Confirm that the collected information is reliable and sufficient to test the hypothesis.
5. Synthesize and evaluate the information against the hypotheses. Iterate as needed.
6. Reach and convey a conclusion based on objective evidence.

Assessments are often performed in two primary phases—a preliminary assessment that involves information gathering and rapid screening of existing conditions and a comprehensive assessment that applies a more thoroughly planned and rigorous assessment of conditions to investigate specific concerns. Figure 3-1 describes a typical comprehensive investigation process, which serves as an outline for the remainder of this Design Guide.

3.1 PRELIMINARY ASSESSMENT

A preliminary assessment is an engineer's first opportunity to observe the in-place conditions of the building. As a preliminary assessment is typically limited in scope, the engineer should document the extent of field observations performed and pertinent findings in a report that is shared with the owner, such that the limitations of the investigation are made clear to all interested parties.

A preliminary assessment should identify the nature of the in-place structure and observed signs of structural deterioration, damage, or distress. If signs of damage, distress, or deterioration are identified during a preliminary assessment, the engineer should scrutinize the load path of the structure and available redundancies. Consider the effect that an observed impairment may have in reducing the structure's ability to resist the immediately probable environmental and

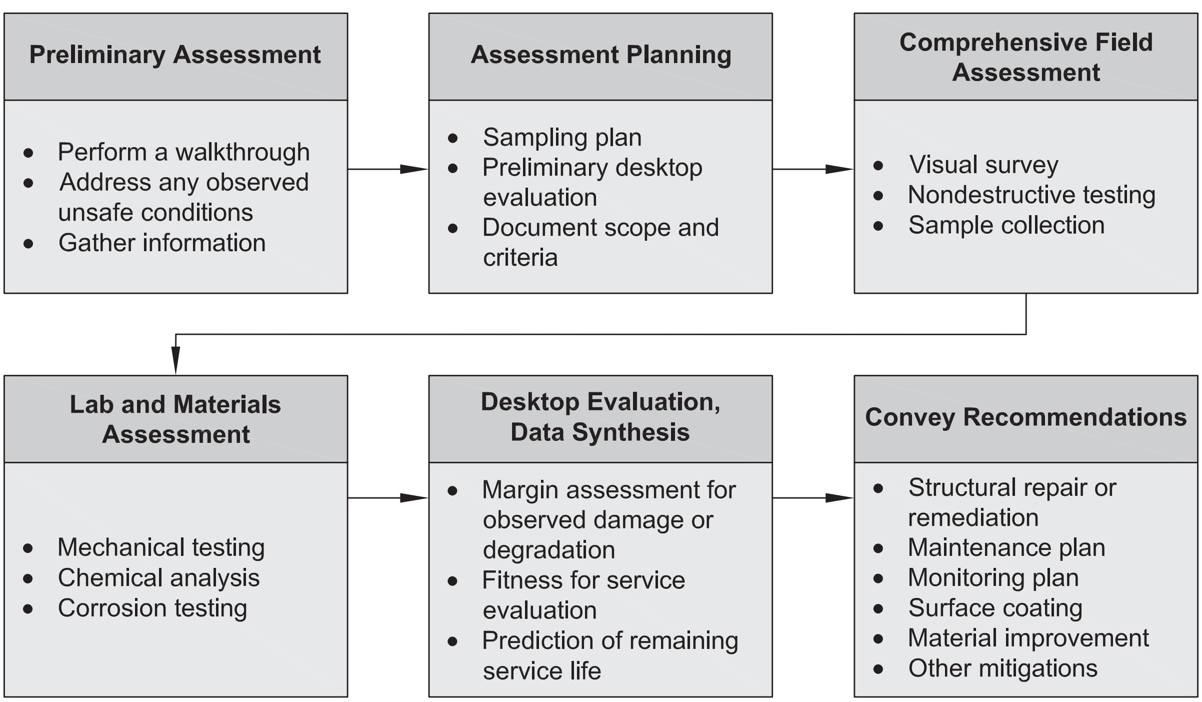


Fig. 3-1. Typical process for a comprehensive condition assessment (adapted from ASCE/SEI 11).

occupancy loads. Prompt interventions may be necessary to limit the use of spaces, monitor movements, or provide shoring to ensure the safety of the existing structure and its occupants.

Construction and design errors sometimes occur and may manifest as distress, damage, or deterioration. Engineers should be conscious of the possibility of errors contributing to adverse conditions and investigate the appropriateness of design assumptions and conformance of the as-built conditions to the design drawings as an indicator of possible design or construction errors.

The preliminary assessment should inform decisions regarding the scope of comprehensive assessment sampling needed to evaluate the in-place conditions.

3.1.1 Reviewing and Validating Existing Documents

Review existing construction documents, when available, to inform the scope of assessment. Documents of interest may include design documents (i.e., drawings), calculations, specifications, fabrication documents, material test records, maintenance records, and quality assurance reports covering original construction and subsequent structural modifications. Previous engineering reports should be explicitly requested. When structural documents are available, they can be a valuable tool for assessing existing buildings.

Unfortunately, existing documents are often unavailable or incomplete. Discussions with onsite staff, previous designers, code authorities, or historical groups may aid in discovering existing documents. Permit documents are sometimes available from the governing municipality through a Freedom of Information Act request. Drawings may contain member information, with connection design delegated to a third party. Where available construction documents fail to provide adequate information to identify required aspects of the design, such as connection strengths, a field assessment may be required to determine the details of the existing structure.

Documents prepared by others should not be relied upon without proper validation. Drawings and other design documents are often revised throughout design and construction, and structures may be modified to accommodate changes in use or occupancy. Such revisions are often not incorporated into the original design or permit documents.

When the documents prepared by others are relied upon for assessments or modifications to the structure, the engineer should validate the accuracy of the documents and record modifications to the structure and its use since the original construction. Such validation efforts may include, among other things, field measurement of existing members and connections, measurement of spans and spacings, validation of slab thicknesses, and approximate determination of applied loads versus those listed on the documents.

3.1.2 Preliminary Assessment Guidelines

Preliminary assessments are usually performed based on readily visible conditions. When additional investigation is anticipated, a preliminary assessment may also be used to quantify the extent of required investigative openings or sampling and their preferred locations based on the field conditions.

A preliminary assessment often involves:

1. Examination of the physical condition of representative components and documentation of the presence of degradation.
2. Visual inspection of representative structural components to verify the information shown on available documents.
3. Verification of load paths among representative components of the systems.
4. Identification and documentation of other pertinent conditions, including neighboring party walls and buildings, nonstructural components influencing building performance, and prior structural modifications.
5. Collection of information needed to obtain representative component properties.
6. Depending on the assessment scope, a survey of present drift, deformation, and elevations.

Finishes or other obstructions may limit inspection access. When the inspection scope is incomplete, the engineer should record the locations of observations and communicate any limitations to the owner. When permitted, visual inspection behind finishes can sometimes be performed through drilled holes using a fiberscope.

When available access is insufficient, the engineer may need to plan for probe openings to allow for a more comprehensive assessment. Extensive openings may be needed to determine the existing conditions when signs of deterioration, distress, or damage are observed, when the scope of the assessment is extensive, or when reliable documents are unavailable. Chapter 4 of this Design Guide discusses sample size selection in more detail.

3.1.3 Common Observations of Interest during Preliminary Assessment

A preliminary assessment should include a full building walkthrough in addition to observations in the work area of focus. The engineer should pay close attention to conditions adjacent to observed water leakage or flow paths. When steel corrodes, the corrosion products are more voluminous than the uncorroded steel, and the associated expansion may cause pack rust or rust jacking. Concrete spalls or masonry cracking may be indications of steel corrosion within encased or enclosed members.

Finishes may mask distress, and a single distressed member in a redundant structure may not result in much deformation. When finishes obscure the structure, consider the condition of nonstructural elements surrounding structural components, which may give clues to appropriate locations for openings during a comprehensive assessment. Conditions of potential interest during a preliminary assessment include but are not limited to:

- Site hazards (i.e., damaged pipes or open excavations).
- Hazardous materials (i.e., asbestos fireproofing or lead paint).
- Failed coatings (potential underlying defects). See Figure 3-2.
- Missing fireproofing (potential underlying defects).
- Unexpected deflections.
- Cracks at reentrants (copes or penetrations).
- Weld cracks or incomplete fusion. See Figure 3-3.
- Buckled members. See Figure 3-4.
- Deformation or misalignment.
- Twisting of members (i.e., inadequate bracing).
- Missing or cut lateral bracing.
- Loose, broken, or missing bolts.
- Loose or unthreaded nuts.
- Exposed faying surfaces.
- Deformed clip angle connections.
- Impacted steel (i.e., forklift or vehicle damage).
- Evidence of previous alterations.
- Cracked or spalled concrete around embedded or encased steel.
- Water damage, rusting, or streaking.
- Archaic or specialty systems that cannot be easily replaced (i.e., clay tile floors, draped catenary floors, or post-tensioned decks).
- Unplanned modifications (i.e., MEP penetrations).

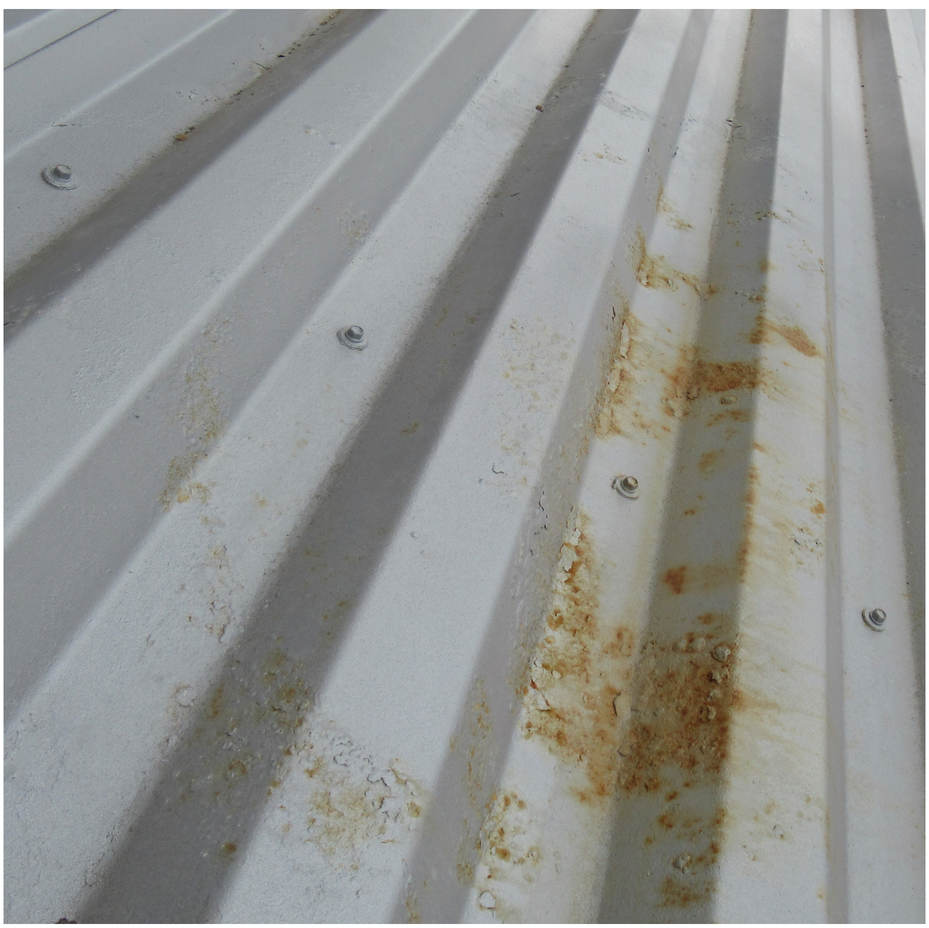


Fig. 3-2. Example of a failed coating system resulting in steel rusting (photo courtesy of Simpson Gumpertz & Heger).

3.1.4 Existing Loading and Exposure Conditions

The engineer should consider the structure's existing loading and exposure during a preliminary assessment to inform subsequent evaluation of the structure's load-carrying capacity. The loading history of the structure should be determined to the extent possible, noting any unusual conditions such as exposure to intense earthquake shaking or fire. The actual permanent and transient loads carried by the structure should be estimated and noted.

Knowledge of environmental exposures is essential for assessing the existence and rate of corrosion. Take note of the installed environment, with particular attention given to the range of temperature exposure, sources of water, and the presence of chemicals, chlorides, or organic materials to which the material may have been exposed. Additional details regarding loading and exposure conditions are sometimes available through discussions with site personnel and building occupants.

3.2 COMPREHENSIVE CONDITION ASSESSMENT

In some cases, where the investigation scope is small or when there are no visible signs of deterioration, distress, or damage, a preliminary assessment may sufficiently validate the accuracy of existing documents and apparent adequacy of structural conditions such that no further field assessment is required. A comprehensive condition assessment may be necessary when a preliminary assessment exposes conditions requiring additional investigation.

3.2.1 Assessment Planning—Extent of Field Investigation

As with preliminary assessments, comprehensive condition assessment should follow the scientific method. An engineer will typically define the problem to be tested and develop hypotheses of the condition's cause, building on the findings of the preliminary condition assessment. Inspection, sampling, or testing is planned and performed to test the hypotheses, recognizing that initial hypotheses are frequently incorrect. Samples from outside an area of suspected damage may aid in establishing baseline observations. The data is then analyzed to draw conclusions supporting or disproving the hypotheses. A disproven hypothesis is revised and retested. A supported hypothesis may be tested several times to achieve a statistically meaningful result.

Finishes and access limitations often inhibit a complete visual assessment of buildings. For example, fireproofing or coating materials may obscure conditions requiring assessment, and certain hazards, such as asbestos-containing fireproofing and lead-based paint—often inferred by age or the

Fig. 3-3. Example of incomplete fusion (photo courtesy of Simpson Gumpertz & Heger).

characteristic red color of a lead-based primer and confirmed by chemical testing—can make sampling very costly. As a result, engineers often need to apply judgment to determine the extent of openings and sampling required during an investigation. Investigations should be planned and performed methodically, with the extent of sampling determined either deterministically or probabilistically, as appropriate. When access is limited, openings or sampling should be selected based on objective measures shared with the owner.

3.2.2 Deterministic Approaches

When performing condition assessments for a defined purpose, condition, or concern, investigative openings and sampling requirements can often be established by engineering judgment or selected based on qualitative measures of failure modes and effects. The following sections discuss two deterministic approaches.

Fig. 3-4. Example of buckled roof framing (photo courtesy of Simpson Gumpertz & Heger).

3.2.2.1 Judgment-Based Deterministic Approaches

Deterministic investigation and sampling performed to assess deterioration, distress, or damage relies heavily on the experience of structural engineers.

For sampling performed primarily to validate in-place conditions, the extent of sampling may vary based on the availability of existing design documents. When existing documents provide member information, the guidelines for preliminary assessment in Section 3.1 of this Design Guide are ordinarily sufficient to validate member sizes and arrangements. However, connection information provided on design documents is often changed or further detailed on fabrication documents, and additional investigation may be warranted to validate as-built connection conditions.

For sampling performed primarily to confirm the consistency of connection details with existing documents, rules of thumb based on experience may guide sample size selection. When available construction documents specify the details of the connections, at least one connection type of interest in the work area should be exposed and visually inspected to confirm consistency with the documents. If the inspected connection deviates from the available documents, additional visual inspection should be performed until the extent of deviations is determined. No sampling is required to validate the material strength when available documents specify steel material grade.

In the absence of construction documents, or where available construction documents do not provide the required connection information, connection information needs to be determined by assessment. An engineer may elect to sample a set of connections and consider the inspected connections representative. When using such an approach, at least three connections of each connection type for primary structural components should be identified, and each identified connection and its connected components should be exposed and visually inspected. If no deviations within a connection type group are observed, an engineer may consider the inspected connections as representative of that connection type. If deviations within a connection type group are observed, additional connections of the same connection type and their connected components should be visually inspected until the extent of deviations is determined.

The experience-based rules of thumb described above may not apply to all conditions or conditions with signs of deterioration, distress, or damage. The investigator should apply judgment to ensure that the extent of inspection is sufficient to meet the assessment's overall goals.

Many connection elements can be inspected visually. Bolt heads should be examined for grade marks and the grade recorded along with the location of where grade marks are found. Figure 3-5 shows examples of common structural bolt grade marks. Additional grade marks of modern bolts and nuts can be found in the Research Council on Structural Connections' (RCSC), *Specification for Structural Joints Using High-Strength Bolts* (2020). Similarly, typical dimensions of finished and countersunk rivet heads should be measured, as these can be correlated to the shank diameter from historical references. Anchor rod ends painted in green, blue, yellow, or red often indicate the manufacturer's grade designation. Fillet weld leg sizes can typically be measured by visual methods and many types of weld flaws are visually inspectable. A report by the Electric Power Research Institute (EPRI) provides additional guidance for visual inspection and related acceptance criteria for welds (EPRI, 1987).

3.2.2.2 Risk-Ranked Deterministic Approaches

When potential deterioration, distress, or damage is suspected, or for other conditions when a complete assessment is impossible, a risk-based approach called Failure Modes and Effects Analysis (FMEA) may be helpful to guide sampling choices. Various standards for FMEA exist, including U.S. Armed Forces Military Procedures document MIL-STD-1629A, *Procedures for Performing a Failure Mode, Effects, and Criticality Analysis* (USDoD, 1980).

The FMEA process involves considering as many potential failure modes as possible and applying a screening process to rank the likelihood of initiating events and their potential safety consequences into a relative risk score. FMEA can be used to strategically align sampling to risks and establish graded mitigations for higher- and lower-risk areas. Figure 3-6 shows an example FMEA risk ranking. In

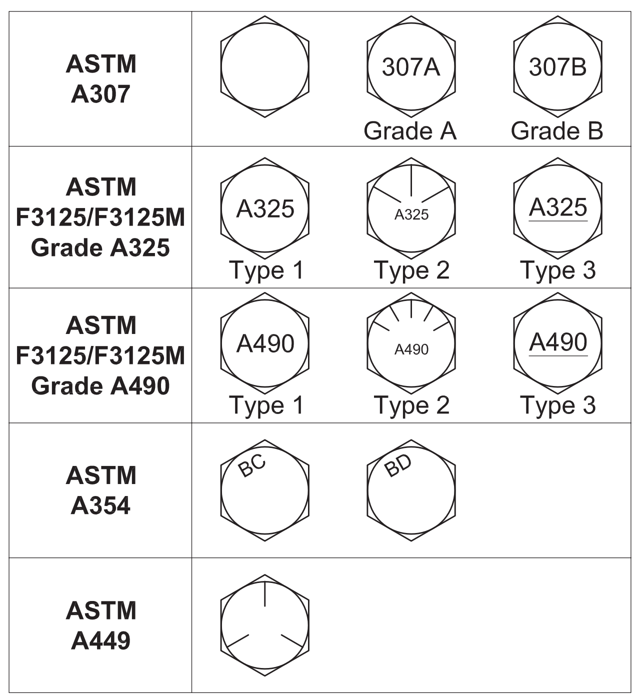


Fig. 3-5. Examples of common structural bolt grade marks.

structural applications, this approach is typically qualitative and judgment-based.

As FMEA is a relative ranking tool, a guided discussion with the owner on the items to be evaluated in the scope of an assessment can assist in decision making regarding a threshold of acceptable risk and the level of investigative effort that is appropriate for the project.

For example, many buildings constructed in the early 1900s contain steel frames built integrally with exterior masonry barrier walls, sometimes called transitional masonry structures, which tend to retain moisture through wetting and drying cycles. Typically, the spandrels in these early frames are more likely to be exposed to water than other members in an enclosed building. They also link the gravity frame of the building to the lateral system. These frames may be prioritized for sampling over other members when the deterioration of a steel frame in a transitional masonry structure is a concern, given their higher likelihood of water damage and the potential consequence of deterioration.

Similarly, nonredundant transfer trusses at the interior of a building may have a lower likelihood of deterioration from water effects but may be prioritized for sampling due to their significant consequences in the event of failure. Alternatively, interior infill beams in a fully enclosed building that are largely redundant and show no signs of distress may be a low priority for sampling. Adverse conditions that score high in their likelihood or consequence or have a combined high screening in both considerations are addressed as the highest priority in a risk assessment.

As with any deterministic approach, the discovery of poor performance in a chosen sample may lead to the need to increase the extent of sampling. However, discovering good performance where damage, deterioration, or distress are most likely to occur may provide greater confidence in satisfactory performance in other locations and may reduce the overall assessment of risk to an acceptable level.

3.2.3 Probabilistic Approaches

When the extent of assessment is not correlated to conditions that can be isolated by judgment or when the sample size is very large, statistical methods may be useful to determine an appropriate sample size. In building components, sampling is typically performed to establish conditions with a 95% probability of exceeding an expected performance characteristic on a normal distribution.

Approaches to statistically determining sample size and acceptance criteria are described in MIL-STD-105E, *Sampling Procedures and Tables for Inspection by Attributes* (USDoD, 1989), or ASTM E122, *Standard Practice for Calculating Sample Size to Estimate, with Specified Precision, the Average for a Characteristic of a Lot or Process* (ASTM, 2022a). Statistical methods are only effective for samples produced through stable environments—that is, the variability associated with production, exposure, and external effects should be sufficiently consistent such that the sampled condition is representative of the whole. Production defects in manufactured components may be candidates for this type of sampling.

3.3 NONDESTRUCTIVE EXAMINATION

Weld and nondestructive examination should be performed by inspectors qualified to applicable American Welding Society (AWS) or American Society for Nondestructive

Likelihood Score \ Consequence Score	< 50	50 – 75	76 – 100
76 – 100	Medium	High	High
50 – 75	Low	Medium	High
< 50	Low	Low	Medium

Fig. 3-6. FMEA risk ranking tool example.

Testing (ASNT) standards. The following types of nondestructive examination are common in field applications:

Visual testing. Visual testing is readily performed to determine the surface condition of steel materials and welds, to check the dimensional accuracy of conditions, and to detect the presence of corrosion or other irregularities. It may be performed by the naked eye or using visual devices such as magnifiers, borescopes, or fiberscopes. Only surface conditions can be examined by this method. Visual testing may be performed by AWS-certified welding inspectors or CWB-level 2 certified welding inspectors.

Liquid penetrant testing. Liquid penetrant testing (or dye penetrant testing) may be used to detect or highlight surface cracking or discontinuities. This method is sensitive to small surface discontinuities, is simple to apply, and produces indications directly on the affected surface. Liquid penetrant testing requires the tested material to be prepared by removing surface contaminants and applying a penetrating oil—a visible dye penetrant or fluorescent penetrant—to the tested surface. The penetrant is then removed from the test surface by wiping or rinsing with water before applying a drying developer. The penetrant remaining in the discontinuity bleeds out, forming a contrasting indication on the surface.

As the penetrating liquid is required to enter a discontinuity to be effective, it is only useful for detecting surface flaws. It is ineffective for detecting flaws in metals underlying coatings. A good penetrant will be effectively drawn into small openings, even against gravity. Fluorescent penetrants, which are typically made brilliant by a black light, provide better contrast than visible dye penetrants, making detection more accurate. However, surface roughness, rust, and contaminants can make liquid penetrant test results difficult to interpret and may require surface preparation for effective results. Liquid penetrant testing procedures should be performed by a certified Level II or Level III ASNT specialist.

Electromagnetic testing (eddy current). Electromagnetic testing induces small electrical currents into the material, and the changes in current flow are detected by a nearby coil. This testing method can detect surface discontinuities or variations. The material's surface must be electrically conductive; however, the procedures can be optimized so that coatings may remain on test surfaces and rough surfaces can be tested without grinding. The method can be very sensitive to small surface flaws and can detect changes in grain size.

Magnetic particle testing. Magnetic particle testing (Figure 3-7) uses magnetically charged metallic particles to detect cracks or discontinuities. This method relies on the principle that magnetic force lines are distorted by a change in material continuity, creating flux lines in the fine particles applied at the material surface. It is effective at locating tight surface flaws with little preparation and can detect cracking or flaws masked by common coating systems. Lack of fusion, base metal discontinuities, and "stringers" are also readily detectible by magnetic particle testing. The test method is most effective when performed directly perpendicular to the plane of the discontinuity. Small discontinuities deep within the material may not be detectible by this method. Magnetic particle testing procedures should be performed by a certified Level II or Level III ASNT specialist.

Ultrasonic examination. Ultrasonic testing (Figure 3-8) uses sound waves to evaluate the internal condition of the material. It can detect or confirm suspected internal flaws, such as cracks, inclusions, or laminations in steel materials and cracks, porosity, or lack of fusion in welds. Ultrasonic testing uses a transducer that converts electrical energy into mechanical vibrations to apply high-frequency sound waves to the material. The reflected high-frequency sound is then received and converted back into electrical energy. The signal height or amplitude relates to the amount of reflected sound energy. Larger return echo amplitudes suggest larger flaws. The transducer must be placed in direct contact with the metal surface. Calibration may be required to achieve accurate results by ultrasonic examination, which may be done by small drilled holes or cores.

Ultrasonic testing will typically be ineffective on rough surfaces, parts with complicated geometries, or where the discontinuity is expected to be smaller than half the wavelength of the device. The need for direct contact with the material can present challenges when poor surface conditions are encountered or the material is coated, and such surfaces may require preparation by grinding. Ultrasonic testing should be performed by a certified Level II ASNT specialist. A Level III ASNT specialist should develop testing procedures for uncommon applications.

Note: Ultrasonic testing, as defined in AWS, should not be confused with an ultrasonic thickness gauge. An ultrasonic thickness gauge is a useful tool for determining the thickness of solid metal sections.

Hardness testing. Hardness testing involves the indentation of a metal with a calibrated device. A hardness testing device will apply either a metallic ball or a spherical diamond-tipped cone to the material. The indentation depth produced by the device is measured, and the testing force, penetration depth, and area of impression are used to derive a material hardness value.

Many types of hardness testing apparatus exist, but portable hardness testers are common in field usage, and testing can be performed with limited surface preparation.

Fig. 3-7. Magnetic particle testing (photo courtesy of Simpson Gumpertz & Heger).

Fig. 3-8. Ultrasonic examination (photo courtesy of Simpson Gumpertz & Heger).

Hardness testing should be applied perpendicular to the tested surface, and some hardness testing methods require accounting for the direction of gravity. Brinell hardness testing is well suited for in situ testing that is performed to determine the tensile strength of the steel. As compared to other test methods, Brinell hardness testing measures the hardness over an area that results in an averaging of the localized hardness. Other laboratory-based hardness testing methods—such as Rockwell—are better for identifying localized conditions, such as weld heat-affected zone properties. Brinell is better suited for identifying the tensile strength of the base metal.

Hardness testing can be a valuable tool for determining the strength of welds and can provide context on the weldability of base metals. ASTM E10, *Standard Test Method for Brinell Hardness of Metallic Materials* (ASTM, 2023a), prescribes testing procedures. In situ hardness testing can be performed in accordance with ASTM E110, *Standard Test Method for Rockwell and Brinell Hardness of Metallic Materials by Portable Hardness Testers* (ASTM, 2023b). ASTM A370, *Standard Test Methods and Definitions for Mechanical Testing of Steel Products* (ASTM, 2024d), can be used to convert hardness values from one scale to another. ASTM A370 and SAE J417 (SAE, 2018) provide conversions of hardness values to estimated steel tensile strengths.

Hardness testing results are available immediately after the testing is performed. Because of the low cost and essentially nondestructive nature of in situ hardness testing, multiple steel members can be quickly and economically tested.

Less common nondestructive testing (NDT) methods include residual stress measurement, harmonic techniques, x-ray or neutron diffraction, acoustic emission testing, and fatigue measurement, which are left to other literature. AWS D1.1/D1.1M, *Structural Welding Code—Steel* (AWS, 2020b), and ASCE MOP 130, *Waterfront Facilities Inspection and Assessment* (ASCE, 2015), provide additional information on the field application of nondestructive examination.

Several other testing methods useful to steel assessment are more easily performed in lab applications and may require destructive sampling. For example, radiographic testing can provide greater fidelity to through-thickness conditions than other NDT methods but is challenging to accomplish in field applications. Similarly, tension testing, Charpy V-notch (CVN) testing, chemical analysis, and microscopy are almost always performed in a laboratory.

Chapter 4
Lab and Materials Assessment

Material properties should be based on available construction documents, test reports, manufacturers' data, and as-built conditions. Where such documentation fails to provide adequate information to quantify material properties or capacities of assemblies, such documentation may be supplemented by sampling and testing of in-place materials, mockup tests of assemblies, and assessments of existing conditions. Tensile properties, including yield strength, ultimate tensile strengths, and ductility, are typically needed for assessment. Notch toughness may be important for structures subject to dynamic or impact loads. If welding is anticipated, the chemistry of the steel should be tested and a carbon equivalent determined.

Basic properties of yield strength, tensile strength, and elastic modulus can typically be obtained by tension testing as shown in Figure 4-1. Materials testing is not required if material properties are specified on the available construction documents to appropriate standards or are based on the default values described in this Design Guide. Material properties of structural steel vary much less than those of other construction materials, and yield and tensile properties are typically considerably higher than the specified minimum values. As a result, testing for the material properties of structural steel is often not required.

4.1 DEFAULT MATERIAL PROPERTIES

When available documentation does not specify the material grade or strength, default values for basic material properties may be used without the need for testing. These values represent the lowest structural material strengths in commercial use in the era, as accumulated from historical standard specifications, beam mill catalogs, and AISC Design Guide 15 (Brockenbrough and Schuster, 2018). When default assumptions are adequate for the intended purpose, they are preferable to collecting samples, as sample retrieval can potentially cause unintended adverse effects on the existing structure's strength, and repairing the in-place structure after sample removal can be challenging. Default values are conservative by nature, and testing for the actual material strength may be appropriate when default values don't produce adequate results.

4.1.1 Structural Metals

The default minimum strength for structural metals may be determined as shown in Table 4-1. The default strengths shown for wrought iron and pre-standardized structural steel are from AISC 342 and ASCE/SEI 41, *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE, 2023), which interpreted unit stress values from *Iron and Steel Beams 1873 to 1952* (Ferris, 1953) and safety factors from historic producer catalogs to determine the minimum tensile stress and yield stress for design. The historic producer catalogs and other references of the era report a minimum working stress of 10,000 psi for wrought iron and 12,000 psi for steel, with a safety factor of 4, implying a minimum ultimate tensile strength of 40 ksi for wrought iron and 48 ksi for steel. However, because laminations are inherent to wrought iron manufacturing and existed to a lesser extent as inclusions in early steelmaking, strength reductions are appropriate for

Fig. 4-1. Tension test (photo courtesy of Simpson Gumpertz & Heger).

Table 4-1. Default Specified Minimum Material Strengths for Historical Structural Metals		
Year of Construction	**Material**	**Default F_y and F_u, ksi**
Before 1920	Wrought iron	$F_y = 18$, $F_u = 25$
Before 1901	Pre-standardized structural steel	$F_y = 24$, $F_u = 36$
After 1901	Prevailing minimum standard grade	See Table 4-1A
All eras	Cast iron	See Section 4.1.1.4

Table 4-1A. Specified Strength Properties from Withdrawn or Obsolete Editions of Selected ASTM Standard Specifications[1]

Issue Date[2]	Minimum Yield Stress, ksi	Tensile Strength	
		Minimum, ksi	Maximum, ksi
ASTM A9, August 10, 1901	30[3,5]	60[5]	70[5]
ASTM A9, August 16, 1909, 1913, 1914, 1916, 1921	27.5[3]	55	65
ASTM A9, 1924–1932	30	55	65
ASTM A9, 1933–1938	33	60	72
ASTM A7, 1939–1960	33	60	72
ASTM A36, 1960–1999[4]	36	58	80

[1] Values listed are for the grade “medium steel” or “structural steel” for rolled shapes.
[2] ASTM A9 was titled *Standard Specification for Structural Steel for Buildings*. In 1939, ASTM A9 for buildings was consolidated with ASTM A7 for bridges and issued under the single designation ASTM A7, *Standard Specification for Steel for Bridges and Buildings*. ASTM A36 is titled *Standard Specification for Carbon Structural Steel*.
[3] The specification requirement between 1901 and 1923 was that the actual yield point is to be at least one-half of the actual tensile strength from the mill test; the numeric value stated here for specified minimum yield strength is, therefore, one-half of the specified minimum tensile strength.
[4] Applicable through present day for connection parts and rolled shapes other than wide flanges.
[5] The first edition of the ASTM A9 standard was not universally adopted. Various trade standards and codes predated ASTM, including those from the Association of American Steel Manufacturers (AASM) and the American Railway Engineering Association (AREA), and were applicable to the production of steel sections other than rolled wide-flange sections, which at the time were only produced by Bethlehem Steel (Friedman, 2009). When the applicable standard of construction is not known to be ASTM A9, steel properties from 1901–1909 should apply the values from 1909–1921 as a default.

transverse loading. The author understands that the potential effect of laminations on strength was considered when establishing the reduced default values in AISC 342 and that increased default strengths may be acceptable when through-thickness loading is negligible.

Structural steel materials became highly standardized with the first published ASTM standards in 1901. However, structural steel materials from before 1901, wrought iron, and cast iron were not always manufactured to industry-based consensus manufacturing standards. Consequently, default values for these materials may be significantly conservative.

In addition to structural steel, in-place materials could include historical cast iron or wrought iron. Over time, cast iron was gradually replaced by wrought iron, and wrought iron was replaced by steel. Early widespread use of historical cast iron appeared on the facades of buildings because of the material’s relative impermeability and ability to be formed into ornate patterns, as shown in Figure 4-2. Cast iron was often used for columns in construction from the 1850s to the 1890s. The material was manufactured by casting the metal in sand forms, which produces a grainy textured surface. Holes and connection components are cast with the members because the material cannot be riveted and is not commonly drilled. Cast iron members exposed to inadvertent tension or bending have contributed to catastrophic failures, including the Ashtabula Bridge in 1876 and the Darlington Apartment Collapse in 1904. These events exposed the limitations of cast iron, which fell significantly out of favor with engineers after the late 1890s (for engineered commercial structures) and after 1905 (for residential high-rise applications, which were traditionally built without consulting engineering services) (Friedman, 2006). Limited use of cast iron continued through the 1920s.

Wrought iron, which is inherently more ductile than cast iron, was used in structural applications from the 1870s through the 1890s, with limited use as tension rods continuing into the 1930s. The material has a similar exterior appearance to steel and was commonly riveted, but slag inclusions and the manual production process used for the material often create laminations through the material’s thickness.

4.1.1.1 Structural Steel Materials from 1901 and After

ASTM A9 (for buildings) and ASTM A7 (initially for bridges) were first published in 1900, and steel materials after this date were quickly aligned to production under these standard specifications. The standards later merged into a common standard, ASTM A7. If the building age is known, the applicable date of the standard material specification may be reasonably assumed to be the date listed in the available construction documents.

Where the standard specification is listed in the available construction documents without any date, then the date of the standard specification should be taken as the date of the edition of the listed standard specification reasonably anticipated to have been used for the production of the in-place structural steel based on the documented date of construction of the building.

The material used in building construction from the late 1890s through approximately 2000 is likely to be carbon steel with a minimum yield stress between 28 ksi and 36 ksi. For wide-flange shapes, structural steel with a specified minimum yield stress of 50 ksi was commonly available in the 1990s but was not commercially dominant until after 2000. After 2000, structural steel with a minimum yield stress of 36 ksi was commonly used for plates, bars, and rolled shapes other than wide flange.

For historical perspectives on the development of steel standards, see AISC Design Guide 15.

4.1.1.2 Structural Steel Materials from Before 1901

Archaic steels were available from a limited number of producers who had invested in Bessemer or Kelley converters, with material strengths for design reported by the vendors of these materials. Steel used extensively in buildings may be assumed to have been produced by one of these early commercially available processes. Steel was first used in buildings in 1885. If the material is known to be steel, the minimum unit stress (design stress) of steel production in the era was 16 ksi for buildings, with a safety factor between 3 and 4 on tensile stress. A minimum strength assumption of 24 ksi yield stress and 36 ksi tensile stress is conservative. However, wrought iron and cast iron were also common in this era. If wrought or cast iron is suspected or possible, the strengths of those materials should be assumed in a structural evaluation.

4.1.1.3 Wrought Iron

The strength of wrought iron components can be determined by considering the applicable provisions of the AISC *Specification*, where wrought iron properties are substituted for structural steel properties. Minimum yield and tensile stress may be taken from Table 4-1 of this Design Guide. The modulus of elasticity may be taken as 25,000 ksi, consistent with ASCE/SEI 41 and AISC 342. Note that the transverse strength of wrought iron may be less than the longitudinal

Fig. 4-2. Historical ornamental cast iron facades in SoHo, New York, N.Y. (photo courtesy of WireStock).

strength due to the presence of longitudinal laminations that are typical within the through-thickness of the material. The designer should consider the potential effects of laminations when configuring joints.

4.1.1.4 Cast Iron

To avoid the potential for brittle fracture, historical cast iron should only be used in compression with appropriate consideration of the possible effects of eccentricity that may cause the material to be subjected to tension. Notably, eccentric cores are common in round historical cast iron sections, and the potential eccentricity should be considered if the core offset is potentially critical to the analysis. The following design formulae for cast iron were historically presented in ASCE/SEI 41 and have been adopted by AISC 342. AISC 342 incorporated a revised modulus of elasticity, increasing the recommended modulus from 15,000 ksi to 20,000 ksi, which is aligned with Paulson et al. (1996). Because the gray cast iron common in historical building construction lacks reliable tensile resistance, default tensile material properties are not provided. The minimum critical stress may be taken as 17 ksi to determine compressive strength and buckling.

The minimum compressive strength of a cast iron column may be determined as follows:

$$P_{cl} = A_g F_{cr} \tag{4-1}$$

The critical stress, F_{cr}, is determined as follows:
When $L_c/r \le 108$

$$F_{cr} = 17 \text{ ksi} \tag{4-2}$$

When $L_c/r > 108$

$$F_{cr} = F_e \tag{4-3}$$

where

A_g = gross area of the cross section, in.^{2}

E_{ci} = modulus of elasticity of cast iron
= 20,000 ksi

F_e = elastic buckling stress, ksi

$$= \frac{\pi^2 E_{ci}}{\left(\dfrac{L_c}{r}\right)^2} \tag{4-4}$$

L_c = unbraced length of compression member, in.

r = radius of gyration, in.

4.1.2 Weld Metal

The default minimum tensile strength for weld metal is as shown in Table 4-2. These values are determined from AISC Design Guide 15, Appendix A5, and Tables 4-8a and 4-8b.

4.1.2.1 Charpy V-Notch Toughness

The default minimum CVN toughness for weld metal may be assumed, as shown in Table 4-3. These values are provided in AISC 342.

4.1.3 Rivets

As noted in AISC *Specification* Appendix 5, rivets may be assumed to be ASTM A502 Grade 1 (ASTM, 2024c) by default unless a higher grade is established through documentation or testing.

4.1.4 Bolts

The default minimum tensile strength for bolt material may be determined in accordance with Table 4-4. These values are determined from AISC Design Guide 15, Tables 4-4a, 4-4b, 4-4c, and 4-5.

4.1.5 Concrete and Reinforcement

The material properties of concrete and steel reinforcement in composite members should be determined in accordance with the requirements of ACI 562, *Assessment, Repair, and Rehabilitation of Existing Concrete Structures—Code and Commentary* (ACI, 2021).

4.2 TESTING TO DETERMINE PROPERTIES OF IN-PLACE MATERIALS

Where testing is required, the properties of in-place material should be determined by removing samples of the in-place material and subsequent laboratory testing of the removed samples. Assess the potential for hazardous materials, such as lead-based paint or asbestos fireproofing, prior to removing finishes from in-place material. Samples should be removed by grinding or saw cutting because flame cutting can affect the hardness and strength of the sample. Laboratory testing of samples to determine properties of the in-place material should be performed in conformance with standards published by ASTM International (ASTM), the American Iron and Steel Institute (AISI), or AWS, as applicable, and in accordance with AISC *Specification* Appendix 5. Alternatively, it is permitted to use in situ testing of in-place materials where the testing and subsequent data analyses are in accordance with standard test methods.

The strength of cast iron components cannot be determined from small sample tests because component behavior is usually governed by inclusions in the cast iron and other manufacturing-related imperfections in the component. NDT for wrought iron and cast iron is complicated by their metallurgical structures; NDT techniques that are commonly used with structural steel may be unsuccessful when applied to cast iron or wrought iron.

Table 4-2. Default Minimum Tensile Strength for Existing Welds

Listing in Construction Documents	Construction Date or Date Listed on Construction Documents, whichever Is Older	Default Value
Filler metal listed	Any	The specified minimum tensile strength for the filler metal classification
Filler metal not listed	1980 or later	70 ksi
	1961 to 1979	60 ksi
	1928 to 1960	35 ksi

Table 4-3. Default Minimum CVN Toughness for Existing Welds

Listing in Construction Documents	Filler Metal Properties	Default Value
Filler metal listed	The filler metal classification has specified CVN toughness requirements.	The specified minimum CVN notch toughness for the filler metal classification
	The filler metal met the requirements of AWS D1.8/D1.8M *Structural Welding Code—Seismic Supplement* (AWS, 2021) for a demand-critical weld.	40 ft-lb at 70°F
	The filler metal classification has no specified minimum CVN toughness requirements.	7 ft-lb at 70°F
Filler metal not listed	Any	7 ft-lb at 70°F

Table 4-4. Default Minimum Tensile Strength for Existing Bolts

Listing in Construction Documents	Default F_u, ksi[1]
Standard specification designation and grade of bolt are listed	Use F_u as listed in the standard specification.
Minimum tensile strength of the bolt material is listed	Use the listed minimum tensile strength as F_u.
Bolt material not listed	Determined bolt grade in accordance with AISC *Specification* Appendix 5, Section 5.2.6; F_u is then based upon the determined grade.

[1] If a grade mark is observed on the head of the in-place bolt during the condition survey, F_u as determined from the grade of bolt indicated by the observed grade mark may be used.

Sampling of cast iron can cause irreparable damage and is not advised. Cast iron strength should be determined using lower-bound data discussed in Section 4.1.1.4.

4.2.1 Sampling and Repair of Sampled Locations

Sampling locations should be carefully selected, with due consideration given to the loss of capacity and the ease of repair. Samples should be taken from locations that have a tolerable effect on the remaining section's strength and deformation capacity. Sharp corners should be avoided to prevent adverse effects on the remaining section, and the cut surfaces should be ground smooth. Potential sampling locations include flange tips at the ends of simply supported beams and external edges of plates as illustrated in Figure 4-3. Sampling should avoid locations where significant inelastic behavior is expected. The component affected by material removal should be repaired such that it has at least the same strength as it did prior to the removal of the sample.

Where the decreased section strength caused by sampling becomes lower than the required strength, the affected component having the lost section should be temporarily supported and subsequently repaired to restore the required strength before temporary supports are removed.

Test samples extracted from an existing structure may not be consistent with the sampling location and orientation prescribed by ASTM A6/A6M (ASTM, 2024b) for the mill test. For example, a longitudinal sample extracted from the flange is most commonly prescribed for wide-flange shapes by ASTM A6/A6M. However, field samples might be more readily extracted from the web of the existing, in-place steel member. Additionally, the test sample may not have been extracted from the in-place steel in the orientation as specified by ASTM A6/A6M, which is along the rolling direction of the shape and may have been extracted with an orientation transverse to the rolling of the structural shape. Orientation of the test sample is important because the same piece of steel, when tested in the transverse direction (perpendicular to the rolling direction), will have slightly different tensile properties as compared to when tested in the longitudinal direction (along the rolling direction, as required by ASTM A6/A6M). AISC *Specification* Commentary Appendix 5, Section 5.2.2, defines an adjustment factor *R* that may be used to correct the tested yield strength for specimens taken from the web of a member. Alternatively, ASTM A370 provides testing procedures for both longitudinal and transverse specimens. When samples are obtained, the location and orientation of the sample should be recorded.

Where a weld or a portion of a weld is to be sampled for testing, the engineer should define details regarding weld sample removal. Weld samples should consist of both neighboring base and weld metal, which allows the composite strength of the welded connection to be evaluated (see Figure 4-4 as an example). This testing may include base and weld material chemical and metallurgical evaluation, expected strength determination, hardness testing, CVN testing of the heat-affected zone and neighboring base metal, and other tests depending on connection configuration.

Where a fastener such as a bolt or a rivet is removed for testing, a new bolt of the same nominal diameter and at least the same tensile strength as the removed connector should be installed and pretensioned at the time of sampling to replace the removed connector.

Details describing the repairs to the sampled component should be defined where repairs are necessary to compensate for the removed material. All welds associated with the repair should be ground smooth. The repair should be designed to provide equivalent or better strength and deformation capacity than the existing condition.

4.2.2 Alternatives to Destructive Testing

Where material removal is problematic, as is usually the case for weld metal removal, hardness testing may be a desirable alternative. Numerous studies have shown that hardness numbers strongly correlate with tensile strength but are not well correlated with yield strength or ductility (elongation). Hardness testing can be performed in situ on existing steel. If testing is performed in a laboratory, the required sample dimensions are smaller than those usually extracted for tensile testing.

Handheld tools for in-situ compositional testing by x-ray spectrometry have also recently become available and may allow for rapid confirmation of compositional uniformity throughout a structure. These tools have certain limitations and may not replace the need for sampling to determine a material's full composition.

4.2.3 Testing to Determine Material Properties

The engineer should establish the extent of sampling and testing of the installed materials. For example, where construction documents are available for a building, the design documents may identify the standard specification used for the production of the materials installed in the building or may indicate specified minimum properties for the materials. As a result of this knowledge, testing may not be required. If specific materials information is not listed or construction documents are not available, but the date of construction is known, some knowledge regarding materials likely used in the building can be obtained from published references that provide chronological listings of historical materials

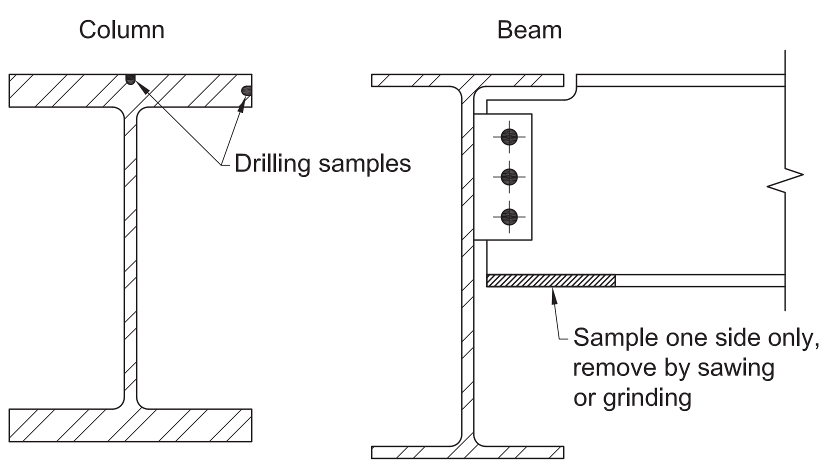


Fig. 4-3. Suggested sampling locations (based on Ricker, 1988).

specifications, such as AISC Design Guide 15. Absent any knowledge whatsoever, testing of the installed materials is required.

The IEBC defines three predominant compliance methods for building alterations: the prescriptive method, the work area method, and the performance method. The work area method further classifies three levels of alternations as Level 1, 2, or 3 based on the extent of the alteration, which must be coordinated across all design disciplines on a building project.

When the work area is limited to a specific section of a building, testing needs to focus only on the properties of the members and connections in the work area. Such a testing program is noted herein as "focused" testing and is often sufficient to meet the intent of AISC *Specification* Appendix 5 for a typical repair project, Level 1 alteration, or most Level 2 alterations as defined in the IEBC.

Although not required by AISC *Specification* Appendix 5, additional testing guidance is provided in ASCE/SEI 41 and AISC 342 for "usual" and "comprehensive" testing of structures subjected to seismic loading, which may guide testing decisions for various types of structures when a full building assessment is undertaken. A testing program of this extent would typically be reserved for Level 3 alterations as defined in the IEBC or seismic upgrades performed under AISC 342. The extent of recommended testing varies based on the extent of available knowledge and properties of interest, including the availability and accuracy of construction and as-built records, the quality of materials used, the quality of construction performed, and the physical condition of the structure.

4.2.3.1 Focused Testing

The recommended minimum number of tests to determine properties of in-place material for usual data collection is based on the following criteria:

1. In the absence of construction documents defining properties of the in-place material, default values for material strength are permitted to be used in lieu of testing. If the material composition is not known to be carbon steel, the lower bound strength of various candidate materials is permitted to be used in lieu of testing. Alternatively, at least one strength sample from each component type in the work area should be removed from in-place material and subsequently tested to determine material composition, yield stress, and tensile strength.
2. In the absence of construction documents defining filler metal classification and welding processes used for existing welds, default values for weld metal strength are permitted to be used as the existing weld metal strength. Alternatively, at least one sample of existing weld metal in the work area should be obtained for laboratory testing to establish weld metal strength, or weld metal strength is

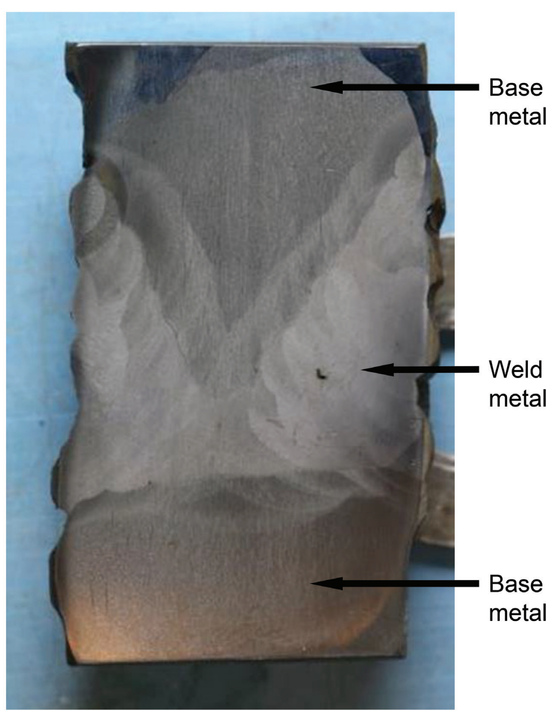


Fig. 4-4. Weld sample removed for testing (photo courtesy of Simpson Gumpertz & Heger).

permitted to be determined by hardness testing on existing welds in the structure without removal of weld metal samples.

4.2.3.2 Usual Testing (AISC 342)

The recommended minimum number of tests to determine properties of in-place material for usual data collection is based on the following criteria:

1. In the absence of construction documents defining properties of the in-place material, default values for material strength are permitted to be used in lieu of testing. If the material composition is not known to be carbon steel, at least one strength sample from each component type should be removed from the in-place material and subsequently tested to determine material composition, yield stress, and tensile strength.
2. In the absence of construction documents defining filler metal classification and welding processes used for existing welds, default values for weld metal strength are permitted to be used as the existing weld metal strength, provided that the standard specification used to produce the existing steel is defined in the construction documents and the existing steel is permitted for use with prequalified welding procedure specifications (WPS) in accordance with Chapter 8 of this Design Guide. Alternatively, at least one sample of existing weld metal for each component type having welded joints should be obtained for laboratory testing to establish weld metal strength, or weld metal strength is permitted to be determined by hardness testing on existing welds in the structure without removal of weld metal samples.

4.2.3.3 Comprehensive Testing (AISC 342)

Comprehensive testing involves a more thorough evaluation of the structure and may include NDT methods such as ultrasonic testing, magnetic particle testing, and radiography to detect defects and damage that may not be visible to the naked eye. It may also involve destructive testing, such as core sampling and load testing, to determine the structure's actual strength and load-carrying capacity. Comprehensive testing is typically conducted when the structure has been subjected to significant hazards or when there are concerns about its structural integrity.

The selection and scope of testing should be based on the specific conditions and hazards associated with the structure, as well as the level of criticality and importance of the structure. The recommended minimum number of tests to determine the properties of the in-place material for comprehensive data collection is based on the following criteria:

1. Where available construction documents defining properties of the in-place material are inconclusive or do not exist, but the date of construction is known and the material used is confirmed to be carbon steel, at least three tensile strength samples or three bolts and rivets, as applicable, should be randomly removed from each component type and subsequently tested to determine yield stress, where applicable, and tensile strength of the in-place material.
2. In the absence of construction documents defining properties of the in-place material, at least two tensile strength samples or two bolts and rivets, as applicable, should be removed from each component type for every four floors or every 200,000 ft^2 and subsequently tested to determine yield stress, where applicable, and tensile strength of the in-place material. If it is determined from testing that more than one material grade exists, additional sampling and testing should be performed until the extent of each grade in component fabrication has been established.
3. For historical structural wrought iron or pre-standardized structural steel, at least three tensile strength samples should be removed for each component type for every four floors or 200,000 ft^2 of construction and subsequently tested to determine the tensile properties of the in-place material. If initial tests provide material properties that are consistent with properties given in Section 4.1, further tests should be required only for every six floors or 300,000 ft^2 of construction. If these tests provide material properties with significant differences, additional tests should be performed until the extent of different materials is established.
4. In the absence of construction documents defining filler metal classification and welding processes used for existing welds, default values for weld metal strength may be used, provided that the standard specification used to produce the existing steel is defined in the available construction documents and the existing steel is permitted for use with prequalified WPS in accordance with Chapter 8 of this Design Guide. Alternatively, at least two samples of each component type having welded joints may be obtained for laboratory testing. The testing should determine the weld metal strength and CVN impact toughness. The CVN tests should be performed at a temperature not greater than the lowest ambient service temperature (LAST) plus 20°F but not higher than +70°F. In lieu of tensile testing, it is permitted to determine the hardness of the welds in the structure without the removal of weld metal samples.
5. For other properties of in-place materials, a minimum of three tests should be conducted. The results of any testing of in-place material of structural steel and wrought iron should be compared to the default minimum values described in Section 4.1 of this Design Guide for the particular era of building construction, where the standard specification used with Table 4-1A is permitted to be

taken as the standard specification representing the commercially dominant grade of structural steel for the applicable era of building construction. The amount of testing should be doubled if the lower bound yield stress and tensile strength determined from testing of the in-place material are lower than the default lower-bound values.

4.2.3.4 Statistical Sampling

In many buildings, steel section sizes are specified for similarity or deflection control such that a limited number of members are critical for strength. An initial screening using default material properties can often be used to limit the extent of required sampling and deterministic testing to members deemed most critical to the structure's performance.

When sampling requirements are established statistically, the engineer should exercise judgment to determine how much potential production variability exists in the sample. For steel that is known to be obtained directly from mill sources or for archaic steel where limited production sources were available, and supply chains for steel components were less diverse, there may be limited variation in source material properties between same-sized members. Alternatively, members from different size groups are more likely to have material property differences. Even when steel is supplied from consistent sources, various heats of steel will have varying structural properties, steel purchased from service centers may not be from a consistent source, and buildings constructed in phases will have additional material variability. A minimum number of tests on representative components may be required to establish statistically accurate properties. When a broad understanding of the strength of steel throughout a building is sought, choosing samples from a variety of members with different shape designations, plates of different thicknesses, etc., may avoid replicating samples from the same production heat and produce a reasonable estimate of the material variability.

Where yield stress and tensile strength of the in-place structural steel are determined by tensile testing of samples of steel extracted from the existing structural steel, equivalent specified minimum values may be determined from statistical analysis of test values, such that it is 90% confident that 95% of the test values fall above the equivalent specified minimum value. This statistical approach is called a lower tolerance-limit analysis, which, when using symbols that are typically applied to structural steel, may be stated in equation form as follows:

$$F_{min} = F_{avg} - k\sigma_{test} \tag{4-5}$$

where

F_{avg} = average of test values, ksi

F_{min} = equivalent specified minimum strength, ksi

k = lower tolerance limit factor from Table 4-5, a function of n, p, and γ

n = number of samples (statistical sample size)

p = proportion of test data falling above the lower limit

γ = confidence interval

σ_{test} = standard deviation of the sample of test values

This general approach, which assumes that the test results are normally distributed but can be readily adapted to a log-normal distribution, is used elsewhere for various kinds of structural materials and is consistent with the reliability-based approach for the design strength of structural steel as utilized by the AISC *Specification.* The combination of target confidence level (γ = 0.90) and target proportion of test data (p = 0.95) falling above the lower limit, as appropriate for structural steel, provides reasonable values for the statistical analysis of results from tensile tests of samples of historical structural steel for statistical sample size, n, of six and larger (Paulson, 2013).

Table 4-5 lists values for k at γ = 0.90 and p = 0.95 for a number of samples, n, ranging from 3 to 30. Values for the one-sided lower tolerance limit factor, k, may also be obtained from statistical handbooks (Odeh and Owen, 1980). Examination of the factors listed in Table 4-5 finds that the value of the factor increases significantly at relatively low numbers of samples. Consequently, to achieve practical values for equivalent specified minimum values, the number of samples in the data set to be analyzed should be at least six, with a number of samples of eight or more being preferable. The standard deviation should be determined using the formula for the standard deviation of a sample of a population, not the standard deviation of a population. This is because the individual tests are obtained from only a limited number of representative components, not from each component in the entire population of all components.

Table 4-5. One-Sided Tolerance Limit Factor, k, for Proportion of Data, $p = 95\%$ and Confidence Level, $\gamma = 90\%$			
Number of Samples, *n*	**Factor, *k***	**Number of Samples, *n***	**Factor, *k***
3	5.31	17	2.27
4	3.96	18	2.25
5	3.40	19	2.23
6	3.09	20	2.21
7	2.89	21	2.19
8	2.75	22	2.17
9	2.65	23	2.16
10	2.57	24	2.15
11	2.50	25	2.13
12	2.45	26	2.12
13	2.40	27	2.11
14	2.36	28	2.10
15	2.33	29	2.09
16	2.30	30	2.08

Chapter 5
Desktop Evaluation and Data Synthesis

The effects of damage, distress, or deterioration should be accounted for when predicting the strength of existing structures.

5.1 CONDITIONS NOT REQUIRING STRUCTURAL ANALYSIS

Certain conditions, such as water staining, evidence of prior leakage, limited corrosion, and local damage, may not require detailed structural analysis for assessment. The engineer establishes the acceptability of such conditions on a case-by-case basis, considering strength loss and serviceability constraints.

5.2 CONDITIONS REQUIRING STRUCTURAL ANALYSIS

If damage, faulty construction, material section loss, or deterioration has occurred, the impairment should be quantified, and section properties should be reduced accordingly using principles of structural mechanics. The structural analysis should account for deviations between available construction records and as-built conditions.

It may be necessary to modify the strength and serviceability criteria to account for the observed conditions of components. Degradation at connection points should be carefully examined; significant strength reductions may be involved, and corrosion may inhibit the required rotation of connections.

When reanalysis is appropriate, evaluations should conform with AISC *Specification* Appendix 5. Analyses performed on existing structures should consider the analysis methods available during the original design, which may need to be reconciled to quantify performance by modern standards. Two commonly encountered conditions are "type-2 with wind" connections and "transitional masonry" frames.

"Type-2 with wind" connections were popular in the 1960s through 1990s, though examples may be found in broader time ranges. Applying a judgment-based approach, the beams were evaluated as simply supported members under gravity load and as moment-connected members under lateral load. The semi-rigid nature of these connections was known to designers for many decades but accepted as reasonable, and subsequent studies have benchmarked their performance as reasonable in modern codes. Reevaluation of these wind frames will benefit by understanding the underlying assumptions and approaches and, if necessary, replicating the partially restrained nature of the connections for analysis, see Geschwindner and Disque (2005).

"Transitional masonry," shown in Figure 5-1, is a now archaic approach to steel construction—also called caged frames or skeleton construction—wherein the building frame is built integrally with or leans on masonry walls for lateral support. Reanalysis of the lateral load-resisting system for these frames using conventional modern methods is unlikely to produce successful results. ASCE/SEI 41 and other standards provide alternative methods for evaluating the lateral load resistance of these frames as a composite steel and masonry system.

In any assessment following the AISC *Specification*, reanalysis performed to modern codes may consider either the allowable strength design (ASD) method or the load and resistance factor design (LRFD) method; the designer may wish to choose the design method strategically for the most economical results. The LRFD method was introduced in 1984, but allowable stress or unit stress design methods have been dominant throughout much of the history of steel design. Reanalyzed structures may produce unacceptable results by one method and not the other depending on the ratio of design loads in the analysis, but either method provides an acceptable degree of reliability to ensure safety in accordance with the building code.

5.3 ACCOUNTING FOR COMPOSITE ACTION

Provisions for composite action are not explicitly addressed in this Design Guide. The design approaches for composite members have changed considerably throughout the evolution of AISC and ACI standards. Like early reinforced concrete designs, early composite design procedures were based on the allowable stress of a transformed equivalent elastic section. Some editions of the AISC *Specification* allowed the bond interface of embedded beam flanges to be considered effective for producing composite action without adding shear connectors.

Early vintage concrete encasement was historically credited for fire protection but ignored for strength and does not typically include confinement reinforcement sufficient to achieve fully effective composite strength. However, riveted construction and concrete encasement for fire protection were common approaches in the same era, and the shear transfer between rivets and encasement is known to provide some additional composite strength to the encased members and connections. Research to quantify the composite action developed between rivets and historic concrete encasement is available from Roeder et al. (1993, 1994, 1996), and Forcier et al. (2002).

Engineers should use judgment when considering the composite action of structures and should consider the stress level and evolution in composite design procedures since the time of original construction. AISC *Specification* Commentary Chapter I describes the limitations and basis of modern composite design procedures. Where reassessment is required, these should be reconciled with the condition of the in-place structure. For more guidance on the evolution of composite design procedures, see *Composite Construction Design for Buildings* (Viest et al., 1997).

5.4 EVALUATING CORROSION

The vast majority of assessments for structural steel deterioration relate to corrosion. Understanding the underlying corrosion mechanism is important for effectively mitigating the effects and preventing recurrence; when the causes of corrosion are not well understood, consultation with a metallurgist is recommended. Corrosion is a natural electrochemical process by which a refined metal gradually deteriorates to a more chemically stable oxide. An uncharged metal atom loses one or more of its electrons and becomes a charged metal ion that transfers from an anode to a cathode in an electro-chemical reaction. This results in a loss of metal at the anode. The difference in electrical potential can be caused by various external sources (impurities, differential metals, etc.), and the transfer of electrons is often facilitated or accelerated by the presence of an electrolyte (e.g., water). Some forms of corrosion that may be encountered in practice include the following.

Atmospheric corrosion. The most common form of corrosion on exposed and unprotected steel, atmospheric corrosion, shown in Figure 5-2, is typically caused by wetting of surfaces in humid conditions, creating an electrolyte that enables a corrosion cell. Atmospheric corrosion may be accelerated by chlorides in marine environments and in areas where deicing salts dissolve as aerosols and is more severe in frequently wetted areas or in areas with inadequate slope to drain.

Galvanic or dissimilar metal corrosion. Galvanic corrosion is caused by electrical current flow created when two different metals with different corrosive potentials are electrically connected in the presence of an electrolyte. Galvanic corrosion, shown in Figure 5-3, is more severe in marine environments where the conductivity of the chloride electrolyte is increased.

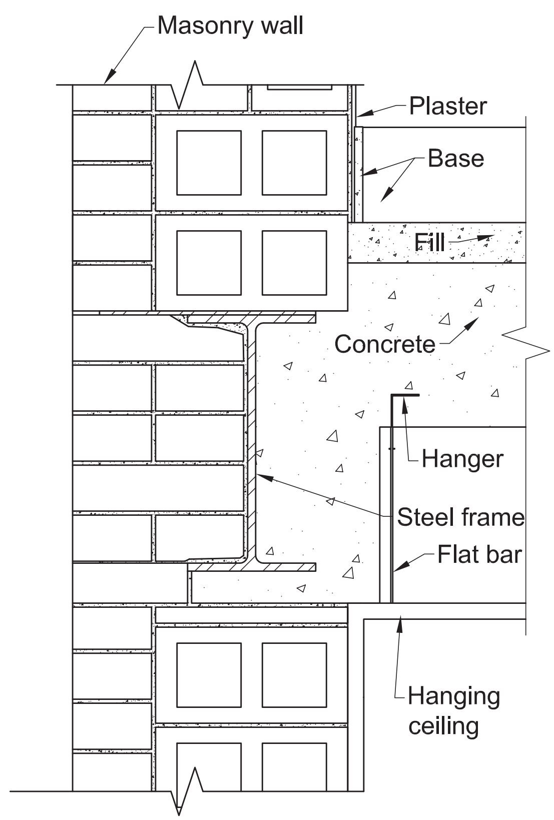


Fig. 5-1. Example of a transitional masonry spandrel beam (photo courtesy of Simpson Gumpertz & Heger).

Stress corrosion cracking (SCC). Stress corrosion is a form of corrosion wherein metal cracking is produced by the combined action of corrosion and tensile stress. SCC only occurs in susceptible microstructures; in structural applications, this includes common high-strength alloys such as austenitic (Type 304, 316) or martensitic (Type 410) stainless steel alloys.

Pitting. Pitting is a form of localized corrosion created by a self-perpetuating electrochemical cell that can cause deep localized section loss in metals. Pitting corrosion, shown in Figure 5-4, is most common in alloys with passive surface layers (e.g., stainless steels) that can be disrupted by chemical contaminants such as chlorides and sulfides.

Fig. 5-2. Example of atmospheric corrosion (photo courtesy of Simpson Gumpertz & Heger).

Fig. 5-3. Example of galvanic corrosion (photo courtesy of Simpson Gumpertz & Heger).

Fretting. Fretting is a form of corrosion caused by the relative motion of two nonlubricated contacting surfaces under load.

Erosion corrosion. Erosion is a form of corrosion caused by the impact of particles on a surface, such as from a water or gas flow. Erosion corrosion, shown in Figure 5-5, is a synergetic corrosion mechanism; that is, the effects of erosion and corrosion are greater when combined than they are in isolation.

Microbial corrosion. Microbial corrosion is a form of corrosion induced by microbes that secrete corrosive chemicals on a metal surface, often in a stagnant marine environment. A common example of microbial corrosion, shown in Figure 5-6, is sulfate-reducing bacteria, which reduce sulfates in water to sulfides, which then form corrosive sulfuric acid.

Less common forms of corrosion in structural applications include:

- Selective leaching (de-alloying materials, such as cast irons)
- High-temperature corrosion
- Stray current corrosion

The structural effects of corrosion can be local or widespread. Corrosion effects encountered in structural applications may include one of the following.

- *Uniform corrosion.* A corrosion effect characterized by the thinning of metal in a uniform fashion.
- *Localized corrosion or deposit attack.* A corrosion effect that is limited to a certain area of a member.
- *Crevice corrosion.* A form of localized corrosion occurring at locations where easy access to the external environment is prevented, such as at mating surfaces of metals or assemblies.
- *Corrosion fatigue.* Fatigue-type cracking of metal caused by repeated stresses in a corrosive environment, characterized by shortened life than would typically occur due to either the repeated stress alone or the corrosive environment alone.

The most common corrosion effects are represented in Figure 5-7. The following sections discuss methods for evaluating corroded conditions.

5.4.1 Limit State Approaches

Most corrosion effects can be evaluated on a limit state basis considering the residual capacity of degraded frame members, connections, and components. This approach was proposed by the National Cooperative Highway Research Program (NCHRP) Report 333, "Guidelines for Evaluating Corrosion Effects in Existing Steel Bridges" (Kulicki et al., 1990) for use in bridge structures but is similarly applicable to buildings, and the limit state nature of the approach allows for ready comparison to conventional AISC acceptance criteria. In this approach, the nominal capacity of the original undeteriorated member strength is multiplied by a residual capacity factor (RCF) to form a yield limit. This approach is useful for performing structural analysis of widespread corrosion effects in an analytical model and allows for simultaneous consideration of deterioration with corrosion-induced fixities or pressures.

Because local imperfections on surfaces caused by corrosion can affect load redistribution capacity, analyses using residual capacity factors with plastic acceptance criteria should be scrutinized carefully, considering 3D behavior, the section's inelastic strength, and the system's capacity for load distributions in the nonlinear response range.

An analysis of corrosion effects should consider corrosion on a local level, at the member level, and at the system level. When evaluating corrosion using a structural model, a member residual capacity factor, RCF_M, and a local residual capacity factor, RCF_L, may be defined, with the smaller of the two values applied to any affected member. Member properties for deteriorated conditions are defined with a subscript d.

The following equations present limit states for doubly or singly symmetric prismatic members—that is, wide-flange shapes, channels, etc. Similar approaches can be applied to other member types and limit states with appropriate judgment.

5.4.2 Uniform Corrosion or Loss of Section

Uniform corrosion may be evaluated by simplified methods accounting for the remaining section. The engineer should review conditions case-by-case to determine if the average or minimum remaining section thickness is appropriate for determining section properties. Remaining member thicknesses less than 3⁄16 in. are not addressed by the AISC *Specification* and may require additional consideration of buckling and effective section using other codes, such as AISI S100 (AISI, 2020).

5.4.2.1 Uniformly Corroded Members Loaded in Tension

The tensile strength of uniformly corroded members can be determined directly from AISC *Specification* Chapter D based on the reduced section size or can be evaluated as a function of the original available strength using a residual tensile capacity factor, RCF_{Mt}, defined as:

$$RCF_{Mt} = \frac{A_d}{A} \tag{5-1}$$

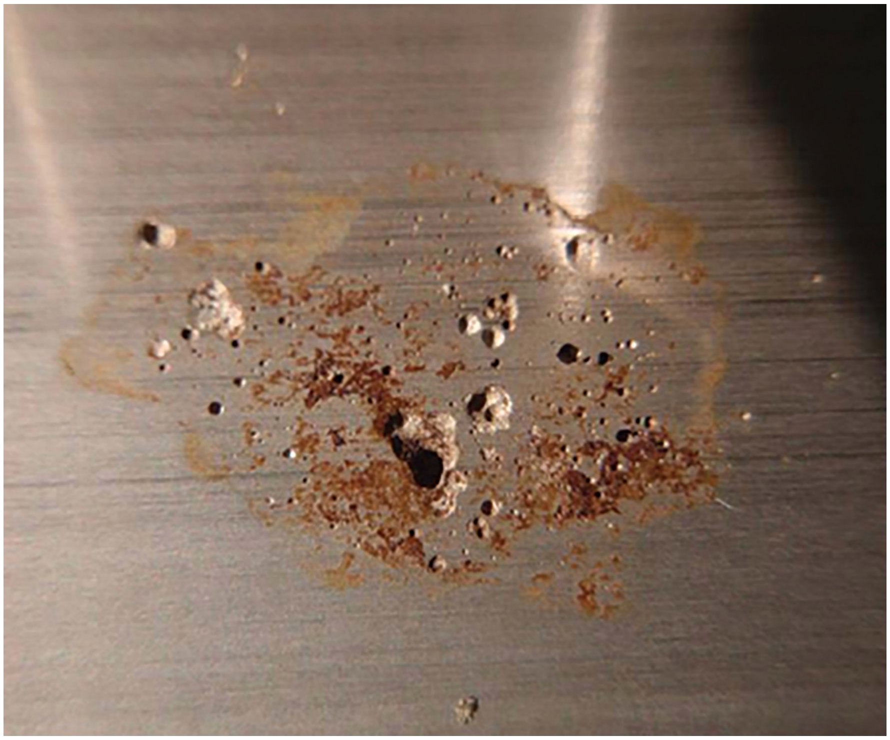


Fig. 5-4. Example of pitting corrosion (photo courtesy of Simpson Gumpertz & Heger).

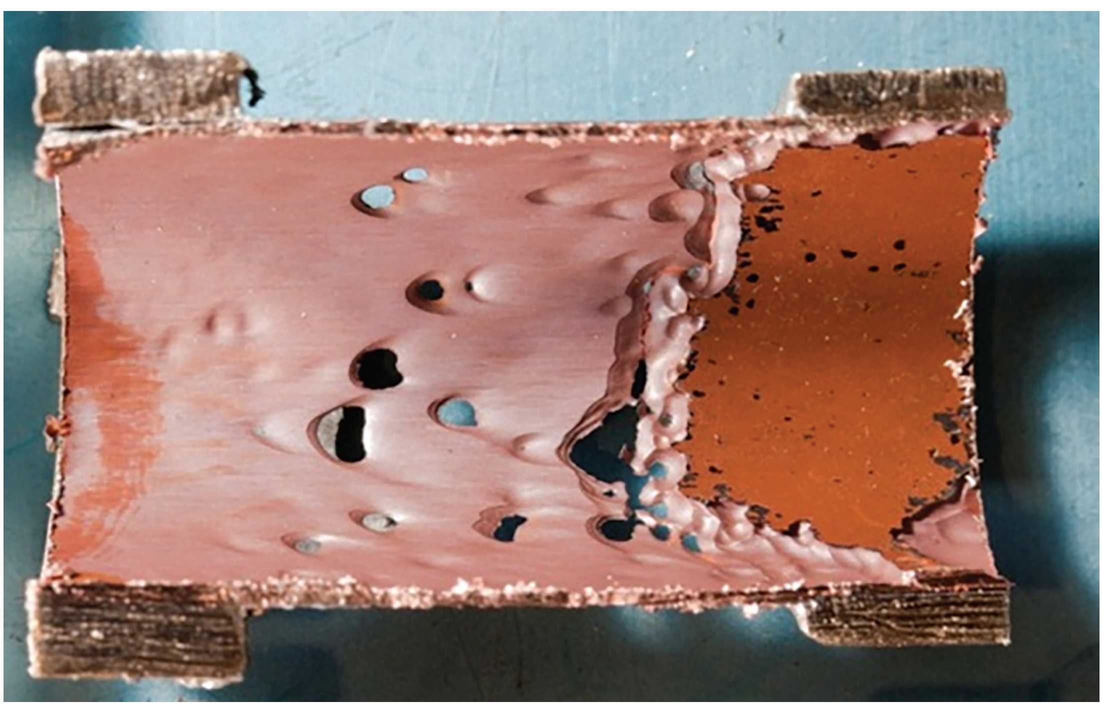


Fig. 5-5. Example of erosion corrosion (photo courtesy of Simpson Gumpertz & Heger).

Fig. 5-6. Example of microbial corrosion (photo from Krieg, 2010).

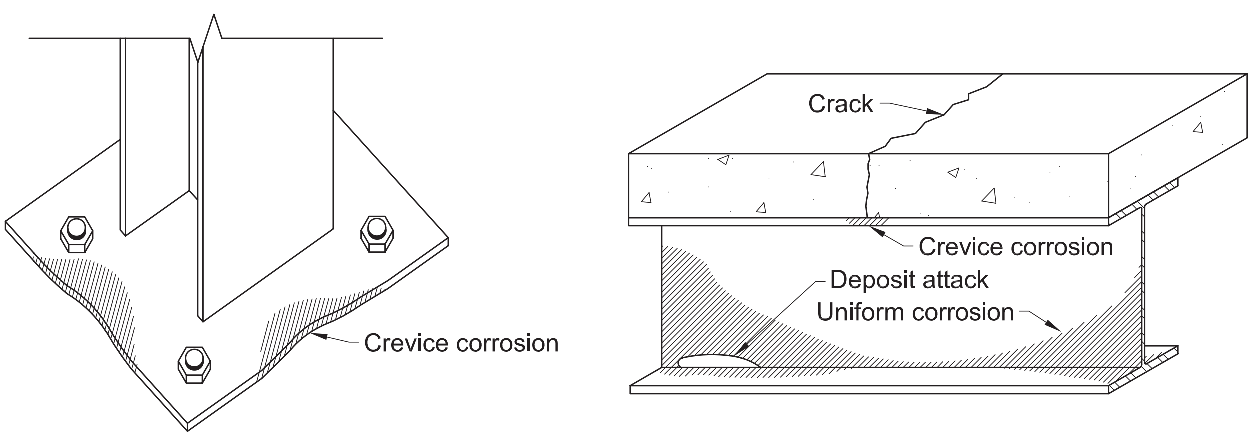


Fig. 5-7. Common corrosion effects.

where

A = original cross-sectional area of the uncorroded member (net or gross), in.^{2}

A_d = remaining cross-sectional area of the corroded member (net or gross), in.^{2}

A residual local tensile capacity factor, RCF_{Lt}, may control for tension members. See Section 5.4.3 for guidance.

5.4.2.2 Uniformly Corroded Members Loaded in Bending

The limiting b/t values in AISC *Specification* Chapter B should be considered when evaluating the local buckling of members with reduced thickness, $t = t_d$.

The bending strength of uniformly corroded members can be determined directly from AISC *Specification* Chapter F based on the reduced section size, or members not subjected to local buckling can be evaluated as a function of the original available strength using a residual bending capacity factor, RCF_{Mb}, defined as:

$$RCF_{Mb} = \frac{S_d}{Z} \tag{5-2}$$

where

S_d = remaining section modulus of the corroded member. Use plastic section modulus for members that, in the judgment of the engineer, do not contain local imperfections that are significant enough to prevent the remaining section from developing its full plastic strength. Otherwise, use elastic section modulus, in.^{3}

Z = original plastic section modulus of the uncorroded member, in.^{3}

A residual local bending capacity factor, RCF_{Lb}, may control for tension elements of bending members. See Section 5.4.3 for guidance.

5.4.2.3 Uniformly Corroded Members Loaded in Shear

The shear strength of uniformly corroded members can be determined directly from AISC *Specification* Chapter G based on the reduced section size, or for members not subjected to shear buckling can be evaluated as a function of the original available strength using a residual shear capacity factor, RCF_{Mv}, defined as:

$$RCF_{Mv} = \frac{t_{wd}}{t_w} \tag{5-3}$$

where

t_w = original web thickness of the uncorroded member, in.

t_{wd} = remaining web thickness of the corroded member, in.

The need for a residual local shear capacity factor, RCF_{Lv}, should be considered. Local shear strength will not usually control except at the ends of members, in locations of concentrated loads, and in areas of significant section loss. See 5.4.3 for guidance.

5.4.2.4 Uniformly Corroded Members Loaded in Compression

The limiting b/t values in AISC *Specification* Chapter B should be used as a guide for evaluating the local buckling of members with reduced thickness, $t = t_d$. AISC *Specification* Chapters C and E should be used for evaluating compressive strength and stability.

The compressive strength of uniformly corroded members can either be determined directly from AISC *Specification* Chapter E based on the reduced section size or, for prismatic symmetrical members without slender elements, can be evaluated as a function of the original available strength using a residual compressive capacity factor, RCF_{Mc}, defined as:

When $\frac{L_c}{r} \leq 25$

$$RCF_{Mc} = \frac{A_d}{A} \tag{5-4}$$

When $25 < \frac{L_c}{r} \leq 4.71\sqrt{\frac{E}{F_y}}$

$$RCF_{Mc} = \frac{A_d}{A}\left(\frac{0.658^{\frac{F_y L_c^2}{r_d^2}}}{0.658^{\frac{F_y L_c^2}{r^2}}}\right) \tag{5-5}$$

When $\frac{L_c}{r} > 4.71\sqrt{\frac{E}{F_y}}$

$$RCF_{Mc} = \frac{A_d}{A}\left(\frac{r_d}{r}\right)^2 \tag{5-6}$$

where

E = modulus of elasticity of the member, ksi

F_y = yield strength of the member, ksi

L_c = effective length of the member, KL, from AISC *Specification* Chapter E, in.

r = radius of gyration of the member, in.

r_d = reduced radius of gyration of the corroded member, in.

The need for a residual local compressive capacity factor, RCF_{Lc}, should be considered but will not usually control. See Section 5.4.3 for guidance.

5.4.2.5 Uniformly Corroded Member Deflections

The increased effects of deflections should be accounted for by multiplying the calculated deflection by $1/RCF_M$, where RCF_M is defined as:

$$RCF_M = \frac{I_d}{I} \tag{5-7}$$

where

I = original moment of inertia of the uncorroded member, in.^{4}

I_d = remaining moment of inertia of the corroded member, in.^{4}

5.4.3 Local Corrosion

Local corrosion may be evaluated by simplified methods accounting for the remaining section thickness when there is local thinning or by accounting for the reduced section where holes are made in the section. For most building structures, a limit states approach may be applied. However, considering that discontinuities and openings can affect stress redistribution capacity, sensitive structures may require a more detailed evaluation of fatigue and fracture.

Where required, holes and openings can be evaluated using theoretical stress concentration factors to estimate maximum concentrated elastic stresses. These theoretical predictions are typically uniaxial and can underpredict the effects of residual stress and multi-planar effects in prismatic members. Similarly, their elastic basis does not usually account for force redistribution and plastic stress acceptance criteria common to structural design. However, such an approach is often used to evaluate mechanical structures in fitness for service evaluations and can be applied in building structures with appropriate caution. See API-579/ASME FFS-1, *Fitness for Service* (API, 2007), for more details. Pitting corrosion, in particular, is a candidate for evaluation using API-579/ASME FFS-1. Absent other acceptance criteria, some engineers have used stress concentration factors consistent with round bolt holes to characterize an acceptable level of stress concentration for structures designed to AISC criteria.

Alternatively, an engineer may consider alternative load paths and truss behavior of the remaining structural section around locally corroded components. Webs of wide-flange sections are often candidates for this approach, where the remaining section may possess sufficient strut and tie strength or Vierendeel strength to bridge an opening caused by corrosion. Similarly, if repairs are required, an engineer may employ truss action to reduce the extent of required strengthening, as shown in Figure 5-8. When evaluating alternative load paths, the engineer should consider both the remaining section's strength and effective moment of inertia.

The effect of local corrosion should consider degradation at connections. Additional capacity reductions may be required when the capacity of connections is determined to be less than that of the attached members.

5.4.3.1 Locally Corroded Members Loaded in Tension

Stress concentrations and eccentricities resulting from local corrosion may be neglected when estimating the static capacity of a tension member with moderate deterioration. Yielding of the reduced area should be considered the governing limit state, with the original and remaining area of

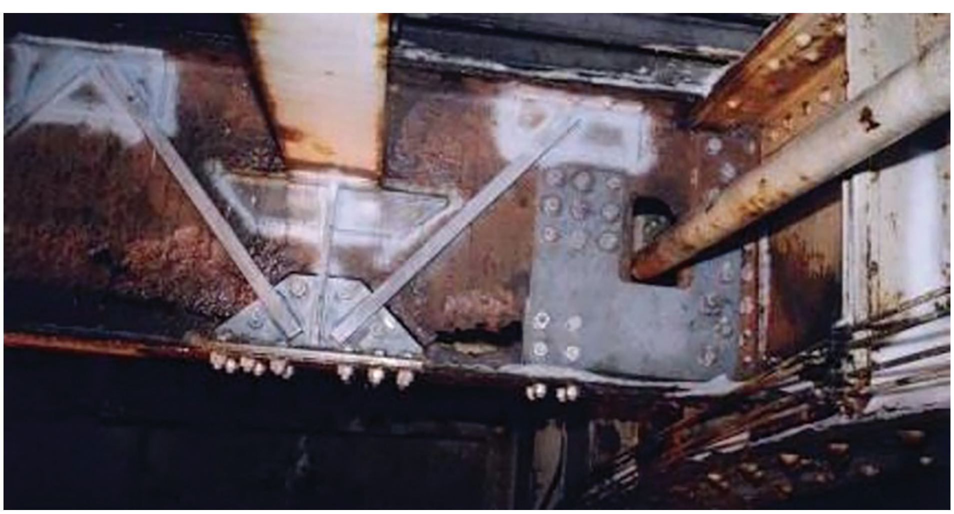


Fig. 5-8. Web strengthening by truss action (photo courtesy of Simpson Gumpertz & Heger).

the corroded members calculated based on the net section. In cases of significant section loss—typically greater than 20%—the possible increase in stress in elements adjacent to the corroded element should be investigated through analysis. The fracture potential for tension members with local holes should also be considered.

5.4.3.2 Locally Corroded Members Loaded in Bending and Shear

Local losses of web or flange thickness can be evaluated conservatively based on reduced overall section strength, similar to the approach previously discussed to address uniform corrosion. When holes are present or where considering a global reduction in thickness is overly conservative, the locally corroded portions of the member can be evaluated as openings or holes in the member. Local corrosion near connections or under concentrated loads should consider the effects on stability.

Holes in webs. Local corrosion or holes on webs can be evaluated using approaches outlined in AISC Design Guide 2, *Steel and Composite Beams with Web Openings* (Darwin, 1990). Typically, stress concentrations and surface roughness created at the boundaries of web holes may be neglected when evaluating static strength.

Holes in flanges. Local corrosion or holes in flanges may be evaluated based on the net section in the flange and reduced section modulus, similar to the net section strength checks in AISC *Specification* Section F13.1. Flanges of bending members are typically loaded in tension, and stress concentrations induced by sharp corners or surface roughness, which have the potential to increase the fracture susceptibility of tension components, should be considered.

5.4.3.3 Locally Corroded Members Loaded in Compression

Local corrosion of compression members can either be evaluated by considering a reduced section strength on the entire member, similar to the approach previously discussed to address uniform corrosion, or may be evaluated by considering a reduced section on only the locally affected portion of the compression member and treating the member as a "stepped" column with reduced section properties applied over the affected portion of the column. The stepped column approach is demonstrated in Example 6.2.3 of AISC Design Guide 15 (Brockenbrough and Schuster, 2018).

Asymmetrical deterioration can result in eccentricities and bending effects on compression members. The eccentric moment induced by compression should be accounted for in the evaluation as a combined stress on the member per AISC *Specification* Chapter H, using second-order analysis when appropriate.

5.4.3.4 Deflections

The effects of local corrosion on the deflection of a member under load can typically be neglected.

5.4.4 Corrosion Effects on Bolts and Welds

Crevice corrosion on faying surfaces may generate a rust pack that induces pressure on the faying surface and additional pretension on the bolts. The rust pack on faying surfaces can cause unintended fixity and preload within the connection. The engineer should assess conditions sensitive to this additional fixity by analysis and take measures to mitigate the further progression of crevice corrosion before it reaches a critical level.

Bolt pretension induced by rust pack on faying surfaces is not usually detrimental to the performance of the structure, provided that the bolts remain intact. However, in a corrosive environment, stress corrosion cracking may attack pretensioned bolts. Eventually, the bolts fracture and may leave open holes or missing bolts. Stress corrosion cracking and chloride stress corrosion of bolts and other preloaded components can be accelerated by the installed environment, such as the temperature and humidity typical to enclosed swimming pools. EPRI (1988) provides additional guidance for assessing the effects of bolt deterioration.

Weld deterioration is typically similar to that of the base steel. Although not common, it may be possible for crevice corrosion to form on the unfused areas of welded joints, inducing tension on the welds. Additional considerations of the weldability of the members may drive remedial decisions.

5.4.5 Corrosion-Induced Movements, Pressures, and Fixities

5.4.5.1 Axial and Rotational Fixities

Corrosion can induce axial restraint on joints that accommodate thermal changes and rotational restraint on members intended to perform as pins. Additional load effects from these axial and rotational restraints should be considered when evaluating members and connections.

5.4.5.2 Rust Jacking or Shifting

Metals expand as they oxidize; the pressure induced by the oxidized steel is often called rust pack or rust jacking. Rust jacking, shown in Figure 5-9, can contribute to unexpected preload on connection parts and compression in restrained members. The extent of the effect can be determined by measuring the strained condition and estimating the likelihood of increased exposure. Rust jacking is often most critical at interfaces with concrete bases or masonry walls, which can crack due to rust-induced movements.

5.5 ASSESSMENT OF PHYSICAL DAMAGE, DISPLACEMENT, AND DEFORMATION

AISC *Specification* Chapter C describes the requirements for determining the required strength and stability of steel structures. Stability is required to be provided for the structure as a whole and for each of its elements and requires considering:

- Flexural, shear, and axial member deformations, and other component and connection deformations that contribute to the displacements of the structure
- Second-order effects, including P-Δ and P-δ effects
- Geometric imperfections
- Stiffness reductions due to inelasticity, including the effect of partial yielding of the cross section, which may be accentuated by the presence of residual stresses
- Uncertainty in the system, member, and connection strengths and stiffnesses

The AISC *Specification* permits any rational method of design for stability that considers all the listed effects. Physical damage and degradation can amplify these destabilizing effects. Conditions encountered in practice may include:

- Second-order loading from imposed deformation, out of plumbness, or out of straightness
- Increased residual stress from cold working or local damage
- Geometric imperfections from external damage
- Stiffness reduction from section loss
- Partially yielded sections

5.5.1 Settlement and Imposed Deformations

The stability design provisions of the current (2022) AISC *Specification* require consideration of second-order effects, where equilibrium is satisfied on the deformed geometry of the structure. Initial imperfections in the structure, such as out-of-plumbness and material and fabrication tolerances, create additional destabilizing effects. Settlement effects can be similarly evaluated in a second-order analysis that includes imposed deformations in the analysis model to evaluate the risk of instability caused by settlement.

With a critical settlement defined, the engineer may establish an onsite monitoring procedure and an acceptable threshold or warning-level settlement measurement that allows time for interventions, should they become required.

Fig. 5-9. Examples of rust jacking and cracking induced by oxidized embedded steel (photos courtesy of Simpson Gumpertz & Heger).

The monitoring program should consider and accommodate the level of knowledge on the settlement rate, the frequency of monitoring measurement, the potential consequences of settlement, and the time required to implement mitigating measures if settlement monitoring exceeds the threshold or warning level.

Building settlements and deformations may induce preload on building elements. Structural shifting, load path changes, and load shifting may occur when preload is released, and the structure may require realignment or leveling to accommodate rebound effects.

5.5.2 Evaluation by AISC *Specification* Appendix 1

For more complex damage or degradation, AISC *Specification* Appendix 1 provides details for explicitly modeling system and member imperfections and inelasticity within the analysis. The appendix provides elastic and inelastic approaches for evaluating frame stability.

The elastic approach is an extension of the direct analysis method and permits direct modeling of initial imperfections and evaluation of nominal compressive strength based on the cross-sectional strength. This approach can be useful for assessing out-of-tolerance effects or member deformations that exceed the assumed tolerances in the prescriptive procedures in the AISC *Specification*. Candidates for this analysis may include twisting effects, out-of-plumbness, and out-of-straightness exceeding conventional fabrication and erection tolerances.

The inelastic approach provides a means for explicitly evaluating the full range of stability parameters, including the effects of partial yielding. This approach requires a rigorous analysis but is sometimes the only practical means to assess overloads or conditions not contemplated in the prescriptive procedures of the AISC *Specification*.

AISC *Specification* Commentary Chapter C and Appendix 1, as well as AISC Design Guide 28 (Griffis and White, 2013), provide detailed guidance regarding this approach.

Chapter 6
Special Topics

6.1 ACCOUNTING FOR EFFECTS OF FIRE DAMAGE AND HIGH-TEMPERATURE EXPOSURE

Assessment of fire or thermal effects for steel beams and columns may depend on temperature exposure and the suitability of permanently deformed conditions. Steel is manufactured by a melting process that exceeds the heat produced by conventional fires, and carbon steels exposed to less than 1,300°F will not experience metallurgical changes. Effectively, once structural carbon steel cools, it regains full strength. However, loss of straightness may limit the serviceability of the member. Deformed members left in place will need to be evaluated for second-order effects using AISC *Specification* Chapter C or Appendix 1, and connections will need to be evaluated for catenary forces induced by the thermally deflected beams. Figure 6-1 shows the effects of severe fire exposure.

Certain steel materials rely on heat treatment in production, which may be affected by relatively low fire exposure. High-strength bolts and quenched and tempered steels both use heat treatment in production and may be adversely affected by fire. Bolt relaxation occurs rapidly at higher temperatures.

The ignition temperature of the nonstructural contents of the building can sometimes inform the exposure of these components to fire. For instance, glass in conventional incandescent light bulbs will distort and melt at 950°F, which is approximately the temperature at which quenched and tempered steels will temper to lower strength. National Fire Protection Association (NFPA) 921, *Guide for Fire and Explosion Investigations* (NFPA, 2024), provides comprehensive guidance for assessing fire and explosion effects. Additional guidance can be found in AISC Design Guide 19, *Fire Resistance of Structural Steel Framing* (Ruddy et al., 2003); API 579/ASME FFS-1; the ASME *Boiler and Pressure Vessel Code,* Section II Part D, Material Properties (ASME, 2023); and AISC *Specification* Appendix 4.

A method of post-fire steel assessment and recommendations for sampling and testing are provided in "Integrity of Structural Steel after Exposure to Fire" (Tide, 1998). The general guidance from that paper is incorporated into this Design Guide in the following paragraphs.

After fire exposure, it is convenient to categorize the members as follows:

- Category 1: Straight members that appear unaffected by the fire. This includes members that have slight deformations not easily detected by visual observations (within 4 or 5 times ASTM A6/A6M rolling tolerances).

Fig. 6-1. Examples of fire-exposed beams and columns from FEMA (2002).

- Category 2: Members noticeably deformed but could be heat straightened if economically justified.
- Category 3: Members severely deformed that only under extreme circumstances would repair be given any consideration.

Once the condition of each individual member is determined, the safety of the whole structure can be established. A member inventory should be performed before an assessment of repair or replacement can begin. The biggest challenge often encountered with this evaluation is convincing the interested parties that the basic steel properties of Category 1 and 2 members were unaffected by the fire. Camber and sweep of each fire-exposed structural steel member should be determined using appropriate measurement techniques (plumb bob, string line, laser). A Category 1, 2, or 3 designation should then be assigned to each member. Most often, the Category 2 or 3 designation can be assigned without measuring because of severe local buckles or excessive deflections…

…Earlier studies demonstrate that Category 1 straight members require only minimal consideration, Dill (1960), Smith et al. (1981), Kirby et al. (1986), Avent (1992), and others. Metallurgical or structural degradation does not occur with a Category 1 appearance. For any significant metallurgical degradation to occur, temperatures would have to exceed 1,330°F [for carbon steel]. Prior to reaching this elevated temperature level, buckling or large deflections would certainly occur. Slightly deformed Category 1 members, with deformations greater than rolling tolerances, should be analyzed to determine the repair level. Depending on individual circumstances, the analysis will determine if these members can be accepted unconditionally, heat straightened, stabilized with supplemental braces, or reinforced with plates and shapes.

Category 2 members require additional attention because the decision to repair or replace is often a function of the nearby members' condition. A beam is easy to replace when compared to a column supporting several floors. If a Category 2 member is heat straightened, the change in metallurgical and structural properties will be inconsequential. Rehabilitation or replacement of Category 2 members is usually dependent on expediency, economics, or overcoming the human psychological rejection of what appears to be damaged steel.

In most cases, Category 3 members are obvious and usually rejected without much consideration. Salvaging a Category 3 member would most likely occur in a very critical location where removal is inappropriate or impossible. Repair and reinforcement is then implemented as required. Inspecting connections is imperative for beams that will be retained.

Connection behavior is different than main member behavior when the temperature increases because of their relative compactness. The axial force developed by a restrained member will impose large forces on the end connections. Generally, the beam will buckle or deform to accommodate the axial force. Under these conditions, connection distress is easy to identify; when a Category 1 steel beam cools, if the connection has fractured, the steel beam will pull away from the adjacent member revealing the damage. It is common to see fractured connections at the ends of buckled beams. As the buckled beam cools and shortens, the connection material, bolts, or welds, will be torn apart…Bolts heated to the tempering temperature and held there for several hours will generally have a reduced pre-tension force once they return to ambient temperatures (Wakiyama and Tatsumi, 1979). For Category 1 beams, achieving this critical temperature is unlikely because the member geometry suggests this temperature was not reached. Because of the variability of bolt installation procedures and quality control, it is impossible for a visual inspection to determine temperature effects on high-strength bolts. Changes in pre-tensioning or metallurgical properties of high-strength bolts can only be determined by nondestructive or destructive testing. Destructive bolt tests by Kirby (1991) provide guidance on testing procedures that can be considered. Connections at the ends of Category 2 and 3 members to be salvaged are usually refurbished along with the beam; therefore, evaluation of the connections is not warranted. If the beams are restored in place, then the bolts, brackets, and welds should be given special attention.

6.2 ASSESSING FRACTURE POTENTIAL OF CRACK-LIKE FLAWS

Fracture potential depends on the size and shape of flaws, the extent of applied tension, material toughness, service temperature, joint restraint, residual stress from welding or production, and ductility demand.

In most cases, fractured members should be corrected by replacing or reinforcing the fractured component and providing alternative details with lower fracture potential.

Rough surfaces and crack-like flaws, such as those from gouges or flame-cut surfaces, can cause concentrated stresses, increasing a steel member's fracture and fatigue potential. It is usually preferred to remove crack-like flaws in tension members. However, small notches or gouges may be tolerated at the engineer's judgment. AISC's Engineering FAQs—**aisc.org/steel-solutions-center/engineering-faqs** (AISC, 2024)—provide guidance for evaluating thermally cut edges, which is useful guidance for evaluating notches or gouges of any type and is summarized as follows.

Inadvertent notches or gouges of varying magnitude may occur in thermally cut edges, depending upon the cleanliness of the material surface, the adjustment and manipulation of the cutting head, and various other factors. When thermally cut edges are prepared for the deposition of weld metal, AISC *Specification* Section M2.2 provides acceptance criteria that consider the effect of discontinuities that are generally parallel to the applied stress on the soundness of welded joints. Additionally, correction methods for defects of various magnitudes are stipulated therein. When thermally cut edges are to remain unwelded, the following surface condition guidelines are recommended:

1. If subjected to a calculated tensile stress parallel to the edge, edges should, in general, have a surface roughness value not greater than 1,000 μin. as defined in ASME B46.1 (ASME, 2019). The AWS *Surface Roughness Guide for Oxygen Cutting* (AWS C4.1) Sample 3 (AWS, 2010b) may be used as a guide for evaluating the surface roughness.
2. Mechanically guided thermally cut edges not subjected to a calculated tensile stress should have a surface roughness value not greater than 2,000 μin. as defined in ASME B46.1. The AWS *Surface Roughness Guide for Oxygen Cutting* (AWS C4.1) Sample 2 (AWS, 2010b) may be used as a guide for evaluating the surface roughness.
3. Hand-guided thermally cut edges not subjected to a calculated tensile stress should have a roughness not greater than 1⁄16 in.
4. All thermally cut edges should be free of notches (defined as a V-shaped indentation or hollow) and reasonably free of gouges (defined as a groove or cavity having a curved shape). Occasional gouges not more than 3⁄16 in. deep are permitted. Gouges greater than 3⁄16 in. deep and all notches should be repaired.

Roughness exceeding these criteria more than 3⁄16 in. deep should be removed by machining or grinding and fairing-in at a slope not to exceed 1:10. The repair of notches or gouges greater than 3⁄16 in. deep by welding should be permitted. The following criteria are recommended:

1. The discontinuity should be suitably prepared for good welding.
2. Low-hydrogen SMAW electrodes not exceeding 5⁄32 in. in diameter should be used.
3. Other applicable welding requirements of AWS D1.1/D1.1M should be observed.
4. The repair should be made flush with the adjacent surface with good workmanship.
5. The repair should be inspected to ensure soundness.

In some cases, predominantly in mechanical structures or structures whose failure would not present a life-safety concern, fitness-for-service evaluation may be performed in lieu of repair. Fitness for service evaluation looks at the increased stress intensity expected to be produced by a flaw of defined size, the available material toughness, and the demand-to-strength ratio of the material in order to evaluate the potential for brittle fracture. Assessment methods are provided in API 579/ASME FFS-1.

As most fitness-for-service methods are developed for mechanical applications with planar stresses, they should be used judiciously in structural applications. Limit state methods for steel design are usually premised on the development of plastic strength, which may be incompatible with elastically determined stress intensity factors. When stress intensity methods are applied, strength should be predicted on a purely elastic basis unless advanced fracture mechanics methods that account for elastic-plastic performance are applied to the evaluation.

6.3 ASSESSING FATIGUE DAMAGE

Fatigue, as described in AISC *Specification* Appendix 3 (i.e., high-cycle fatigue), is rare in building applications but can occur in structures exposed to repetitive load reversal, such as crane runways and machine bases, and may accelerate in structures in corrosive environments. Fatigue cracks typically initiate at notches, including natural notches that form at the roots of fillet welds. API 579/ASME FFS-1 provides methods for remaining fatigue life assessment.

A specialized form of fatigue cracking is corrosion fatigue. In this mechanism, material fatigue causes the deterioration of a metal's passive protective film, accelerating the corrosion rate. The progressing corrosion effectively causes the fatigued material to lose its endurance limit, and cracking may progress at relatively low stress levels. While this effect can occur on any metallic material, it is most common on passivated materials, such as stainless steel, under repetitive and alternating loads.

Fatigue cracks should be corrected or arrested to prevent further crack propagation. AWS D1.1/D1.1M, clause 8, and the associated Commentary provide methods for fatigue life enhancement at welds, which may include grinding or drilling of arrest holes to prevent crack progression. Arrest holes—illustrated in Figure 6-2—are a common technique for fatigue crack management in bridges and involve drilling a hole at the base of a fatigue crack to stop further growth. The arrest hole stops crack growth by relieving the concentrated stresses at the tip—the net section should be checked to ensure sufficient remaining material strength and the arrest hole radius should be selected with sufficient curvature to produce an acceptable reduction in concentrated stress. Optimally, the hole's edge should be placed such that the center of the hole is situated behind the crack, with the closest hole edge aligned to the crack tip (Fu et al., 2017). Crack arrest holes are most effective for arresting crack propagation from in-plane stresses and may not be effective at preventing propagation for all types of fatigue cracking, such as fatigue caused by twisting effects. Subsequent monitoring is appropriate.

Low-cycle fatigue—as encountered from earthquake damage—has been the subject of a significant number of related structural standards since unexpected fractures were observed in welded moment frame connections following the Northridge, California, earthquake in 1994. The reader is encouraged to review AISC 342 for guidance on low-cycle fatigue conditions and details.

6.4 EVALUATION BY LOAD TESTING

Load testing must comply with IBC Section 1708 and AISC *Specification* Appendix 5. In-situ load testing may be undertaken as part of an assessment when analytical evaluation is insufficient or impractical to ascertain the ability of an existing structure to carry the expected loads or to provide confirmation of an analytically evaluated condition. When an applicable standard exists, load tests should be performed in accordance with the standard. In the absence of a standard load test procedure, the IBC code requires test procedures to be determined by the engineer in a manner that simulates the applicable loading and deformation conditions.

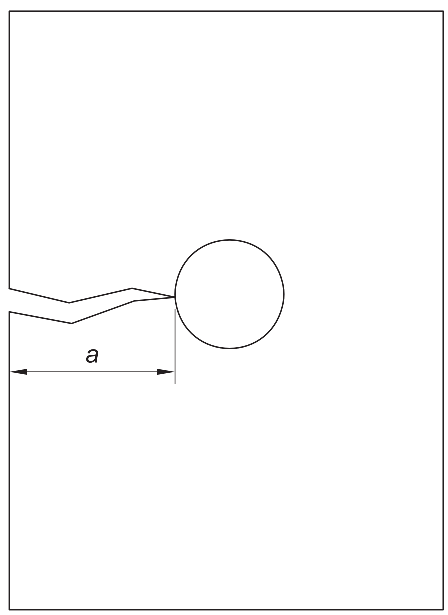


Fig. 6-2. Example of a crack arrest hole where "a" indicates the original crack length.

Chapter 7
Repair and Maintenance Planning

7.1 REPAIR AND MAINTENANCE PLANNING

Repairs to structural systems should be documented in a report or on construction documents. Repairs should aim to address the underlying cause of a deficiency when possible. The extent and basis of assessment related to a repair should be included in the repair documentation, including reference to the building code and AISC *Specification* versions used for repair designs. Required or recommended maintenance should be documented along with any limitations on the expected durability or service life of repairs.

7.2 IMPLEMENTATION PLANS

At times, there may be barriers to the full implementation of a repair program on an immediate basis, and the owner may require guidance on project priorities. In such conditions, a graded approach that distinguishes required mitigations to ensure structural safety from less critical conditions can help the owner with the productive implementation of repair recommendations.

Such a program will typically provide a graded priority for repairs, and the engineer should recommend implementation periods based on the project's risks and goals. The following outlines one possible example of a graded approach to repair implementation.

Priority 1—Immediate action is required. Implement immediately. Immediate intervention is required to correct a known safety hazard, stop accelerated deterioration, or maintain or restore occupancy.

Priority 2—High priority. Implement within 1 year. Conditions which, if not corrected expeditiously, could deteriorate to Priority 1 critical within a year. Situations within this category include rapid deterioration and potential life safety hazards.

Priority 3—Necessary. Implement in the next 2 to 5 years. Conditions that require appropriate attention to preclude deterioration or potential downtime and the associated damage or higher costs if deferred further.

Priority 4—Recommended. Plan for the next 6 to 9 years. Conditions that represent a prudent improvement to existing conditions. These are not required for the most basic function of the facility.

7.3 ACCESS LIMITATIONS

Repair designs need to respond to the physical limitations of the existing space. Pertinent considerations may include:

Access paths. Specify members that can be brought into the access of an existing space. Vertical lift paths—such as those from elevators, doorways, utilities, or external cranes—or horizontal lift paths will often limit the size and weight of pieces that can be brought in for repair through existing floors, conveyances, and access paths. Existing items beyond the immediate work scope may need to be removed or altered for work to be implemented.

Lifting limits. Consider the limitations of handling or conveying equipment in the space and the load applied by this equipment, such as forklifts.

Piece sizes. Multiple short members may sometimes require splicing in the field to allow for piece or worker access.

Welding or thermal cutting in existing spaces. Welding, flame cutting, or arc gouging can cause a fire hazard with the potential to ignite finishes and other combustibles. Additionally, fumes associated with welding or thermal cutting may exceed acceptable levels and limit the ventilation for the welder. Consider details that minimize field welding and provide fume exhaust equipment when appropriate.

7.4 MANAGING CORROSION AND EXPOSURE

Maintenance planning for exposed structures often involves assessing the extent and rate of corrosion. Mitigations for recurring corrosion often involve limiting future exposure and allowing for the free draining of trapped moisture. Consider the effects of environmental exposure, ice expansion (see Figure 7-1), and chemical exposure.

When corrosion is tolerable and aesthetically acceptable, the remaining service life of metals may be estimated considering the exposure conditions and the duration of expected service. The section properties used for analysis can be reduced by a corrosion allowance to account for the anticipated deterioration from continued service in the material's environment.

The corrosion rate is influenced by the extent of chlorides dispersed as aerosols in the environment, which is typically greater in marine environments. Local exposures, such as deicing salts and chemical exposure, can also form significant aerosols and should be considered. Smog sources, once a significant contributor to corrosive aerosols, have become less prevalent in recent years but may cause additional exposure in dense urban environments. Table 7-1 presents examples of annual corrosion rates that unprotected carbon steel subjected to atmospheric corrosion may experience in service.

Table 7-1. Examples of Atmospheric Corrosion Rates for Unprotected Carbon Steel (Cramer and Covino, 2006)				
Environment	**Location**	**Average Temp, °F**	**Rainfall, in./year**	**Corrosion Rate, mil/year**
Polar	Norman Wells, Northwest Territories	22.1	11.5	0.03
Arid	Phoenix, Ariz.	70.5	8.1	0.18
Industrial	Detroit, Mich.	49.7	32.9	0.57
Urban	Montreal, Quebec	45.2	41.8	0.9
Marine	Durham, N.H.	46.4	47.2	1.1
Industrial	Cleveland, Ohio	50.7	39.1	1.5
Industrial	Newark, N.J.	55.5	46.2	2.0
Marine	Kure Beach, N.C.	63.8	57.1	5.8
Marine	Cape Canaveral, Fla.	72.4	48.3	42.0

ASTM STP1137 *Corrosion Forms and Control for Infrastructure* (ASTM, 1992) provides comprehensive data that is useful for estimating the expected corrosion rate for various environmental exposures. Corrosion penetration data is provided as a function of time of exposure and regression analysis data of various steel materials in exposures found in North America. This information can be used to determine a corrosion allowance consistent with the owner's maintenance plans or planned remaining service life of the affected component. Alternative methods of evaluating corrosion progression are available in API 579/ASME FFS-1.

When corrosion has progressed beyond an acceptable level, strengthening or supplemental members will be required to restore the strength of the in-place steel. Given challenges in surface preparation, access, and management of the applied loads during remediation, this often involves investigating alternative load paths for the corroded members.

Fig. 7-1. Example of ice damage (photo courtesy of Simpson Gumpertz & Heger).

Chapter 8
Repair Considerations

An engineer may determine that repairs are necessary for any number of conditions. Repair designs may need to contend with supplementing or adding components to steel members that have existing coatings, corroded surfaces, eroded steel, or physical damage. Often, these components need to be repaired in service, leading to challenges with access and assessment of service load.

There are several existing standards and guides for repair, notably:

- AISC Design Guide 15 (Brockenbrough and Schuster, 2018), Chapter 2, discusses "enhancement of existing structural systems." The reader is encouraged to reference that Design Guide and its examples for repairs made by supplementing the structure.
- AISC Design Guide 21, *Welded Connections—A Primer for Engineers* (Miller, 2017), Chapters 12 and 13, provide detailed and helpful guidance for welding on existing structures, including strengthening and repair considerations.
- AWS D1.1/D1.1M, clause 8 and its Commentary, titled "Strengthening and Repairing of Existing Structures," include guidance on heat straightening, assessing weldability, and improving the fatigue life of structures.

Discussion of repair in this Design Guide is limited to topics not already addressed by the above sources.

Additionally, the paper "Field Welding to Existing Steel Structures" (Ricker, 1988) provides a helpful guide to engineering assessment of repair welds. The present discussion expands upon those considerations.

8.1 BOLTING CONSIDERATIONS

Wrought iron or riveted steel structures may require repair by bolting. Rivets are commonly removed by chipping or, where conditions allow, torching. Take care to avoid damaging base metals (Blodgett, 2006). Removed rivets are commonly replaced with bolts. According to AISC *Specification* Section J1.10, rivets may share load with slip-critical bolts. Paint in the faying surfaces is only permitted for qualified coating systems, and the coating slip resistance may not be known for existing conditions. When coatings are unknown, paint should be removed within 1 in. of the edge of any bolt hole. The RCSC *Specification* now permits galvanized surfaces to be considered as Class A surfaces for design without additional surface preparation, but hand wire brushing of the faying surface may be required to conform with previous code requirements.

Bolted reinforcement typically carries less fire risk than welding and may be preferred in occupied buildings where inhabitants are likely to be disturbed by fumes from welding. Bolting can also be a useful aid for drawing deformed parts of a steel member together. The engineer should ensure that there is sufficient clearance to drill bolt holes in place and that there is clearance on both sides of the bolted connection for installing and tightening bolts.

As discussed in AISC's "Engineering FAQs" (AISC, 2024), if required, mislocated holes can be structurally repaired in accordance with AWS D1.1/D1.1M, clause 7.25.5. Attention should be paid to the clause 7.25.5 Commentary, as it describes a rather involved process that can be used for such repairs. The process involves considerable gouging and welding and, therefore, considerable heat input. As with all repairs, the benefits of the repair should be carefully weighed against the potential problems that the repair itself could cause. Plug welding of mislocated holes is not an acceptable structural repair. If a bolt hole is mislocated by a small amount—say, less than a bolt diameter—it is often possible to adjust the connection material to accommodate the error.

8.2 WELDING CONSIDERATIONS

The following guidance is intended to align with similar provisions that have been accepted for seismic applications in AISC 342.

Where welding to existing structural steel components is performed as part of a retrofit, the requirements summarized in Table 8-1 apply in addition to the requirements of AWS D1.1/D1.1M. For welding entirely new steel components without any welding to existing structural steel, the requirements of AWS D1.1/D1.1M apply.

8.3 ASSESSING WELDABILITY OF EXISTING STEEL AND IRON

AISC Design Guide 15, Section 2.4, and AWS D1.7/D1.7M, *Guide for Strengthening and Repairing Existing Structures* (AWS, 2010a), provide additional guidance on assessing the weldability of existing members.

When a repair or retrofit requires new welds to existing steel components, the engineer should assess the weldability of the existing steel. Depending upon the extent of available information, sampling and testing of the existing structural steel to be welded could be necessary. Weldability, as defined in AWS A3.0M/A3.0, *Standard Welding Terms and Definitions* (AWS, 2020a), is "The relative ease with which

Table 8-1. Recommendations for New Welds to Existing Structural Steel Components

Existing Steel Classification	Welding Requirements
The standard specification used to produce the existing steel is identified in the construction documents and is listed in AWS D1.1/D1.1M, Table 5.3.	AWS D1.1/D1.1M requirements apply. It is permitted to use prequalified WPS in accordance with AWS D1.1/D1.1M, clause 5.
The standard specification used to produce the existing steel is identified in the construction documents and is listed in AWS D1.1/D1.1M, Table 6.9.	AWS D1.1/D1.1M requirements apply. WPS should be qualified by testing in accordance with AWS D1.1/D1.1M, clause 6.
The standard specification used to produce the existing steel is identified in the construction documents as ASTM A7, the existing steel was manufactured after 1950, and the maximum thickness of any element of the existing steel component to be welded is equal to or less than 1½ in. (38 mm).	As accepted by AISC 342, AWS D1.1/D1.1M requirements apply. It is permitted to use prequalified WPS in accordance with AWS D1.1/D1.1M, clause 5. The existing steel can be considered as either a Group I or Group II base metal in accordance with AWS D1.1/D1.1M, clause 5, based on the tensile properties that are specified in the standard specification used to produce the existing steel. Preheat levels should be 50°F (28°C) higher than the values listed in AWS D1.1/D1.1M, Table 5.8, with a minimum preheat of 100°F (38°C). Sampling and testing of the existing steel are not required.[1]
The standard specification used to produce the existing steel is identified in the construction documents as ASTM A373 (ASTM, 1958–1966) or ASTM A441 (ASTM, 1963–1988).	
All other steels, including steels where the standard specification used to manufacture the steel is unknown or the specification used to manufacture the steel is not a standard specification.	Welding requirements should be established by the engineer.[2]

[1] ASTM A7 steel of thickness equal to or less than 1½ in. (38 mm), ASTM A373, and ASTM A441 steels are not listed in AWS D1.1/D1.1M Table 5.3 but may be welded with prequalified WPS when the specified additional requirements are met.
[2] Tables 8-2 through 8-5 provide guidance for establishing welding requirements.

a material may be welded to meet an applicable standard." Weldability is a qualitative term; steel with good weldability can be readily welded, whereas steel with poor weldability may require specialized techniques, such as higher levels of preheat, post-heat, and other measures. Additional information on weldability can be found in AISC Design Guide 21. An assessment of the existing steel for the soundness of the metal itself should also be considered in many cases.

For existing structures that were previously constructed by welding, the weldability of the existing steel can be established by observations that it was successfully welded in the past. Modern welding processes and filler metals are better than those of the past, particularly when compared to the bare electrodes that were used in the 1920s and 1930s. Modern buildings using modern steels that are prequalified in AWS D1.1/D1.1M (AWS, 2020b) need no investigation into weldability because these steels are permitted for use in AWS D1.1/D1.1M.

The most significant weldability challenges involve existing structures that were riveted, not welded. For these situations, good weldability and soundness of the steel to be welded cannot be assumed, but neither are poor weldability and the presence of unsound features a certainty. Under such circumstances, weldability and soundness need to be determined on a case-by-case basis.

Once the weldability and soundness of the existing steel are established, the retrofit construction documents should specify any unique welding requirements associated with the new welds to the existing steel, considering the guidance that follows in this chapter. The contractor will then develop a WPS that incorporates those unique requirements, as well as those of AWS D1.1/D1.1M. The EOR should review the WPS for completeness and compliance with the applicable requirements.

Table 8-1 provides several acceptable approaches for developing WPS for new welds to existing steel. For most structural steel buildings constructed since 1950, the approaches specified in Table 8-1 will typically result in using a prequalified WPS to make a new weld to an existing structural steel component. Nonetheless, there will be instances where the engineer will need to assess by testing the weldability of the existing steel to be welded as part of the requirements for developing a WPS.

For existing steels produced to standard specifications listed in AWS D1.1/D1.1M for use with prequalified WPS, no special requirements are specified by Table 8-1. Because AWS D1.1/D1.1M is focused primarily on welding new steels, structural steels manufactured according to now obsolete standard specifications are not listed in the current edition of AWS D1.1/D1.1M for use with prequalified WPS;

these steels are commonly referred to as “unlisted” steels. Consequently, Table 8-1 recommends extending latitude to certain selected unlisted steels to permit welding using a prequalified WPS, provided that preheat levels are increased. The selected steels are mostly limited to structural steels that were manufactured under some of the first standard specifications that were developed for structural steels with good weldability. Additionally, ASTM A7 structural steel in thicknesses not exceeding 1½ in. (38 mm) that was produced after 1950 is also permitted to be welded under a prequalified WPS because ASTM A7 steel of this era is commonly held to have good weldability, as evidenced in part by the listing of ASTM A7 steel for use with prequalified welds in AWS D1.0-63 (AWS, 1963). AWS D1.0-63 is one of the predecessor standards to AWS D1.1/D1.1M. The thickness limit of 1½ in. is based upon the limitations on welding to ASTM A7 steels as specified in AWS D1.0-63.

For unlisted steels that are produced to standard specifications that are not permitted for use with prequalified WPS by Table 8-1, or for steels of unknown classification, the engineer should determine the welding requirements. This category necessarily includes steels with known good weldability that are prequalified by AWS D1.1/D1.1M but cannot be so designated by the engineer because the classification is not known due to the lack of documentation. For example, the existing steel may be ASTM A36/A36M, but the actual identity of the steel has been lost over time because original construction documents are not available to the engineer. Also included in the category of unlisted steels are those that have poor weldability due to limited control of the compositional characteristics; some steel in this category may be uneconomical to weld when executed under appropriate procedures.

Because of the wide range of possibilities with unlisted and unknown steels, the engineer is obligated to determine welding requirements. However, because of the large number of variables involved and the considerable engineering judgment to be exercised while completing this task, Tables 8-2, 8-3, 8-4, and 8-5 provide general guidance to assist the engineer in fulfilling the obligation to determine welding requirements for steels that are not permitted to use a prequalified WPS. The engineer may need to retain the services of a welding engineer or metallurgical specialist to determine appropriate welding requirements.

Fusion welding began to be used for building construction in the 1920s. Before that, riveting was the primary joining method used for building construction. Weldability is governed by the chemical composition of the steel being joined, and the steel specifications of the 1920s and 1930s were not sufficiently restrictive in the allowable compositional ranges to ensure good weldability. Still, some steels produced during this era have compositions that may be easily welded, whether by coincidence or by deliberate practices of the producing mills.

The shipbuilding efforts of World War II brought about an increase in the popularity of welding as a joining method. While the governing steel specifications did not ensure good weldability, steel producers adjusted the compositional limits to meet the demands of shipbuilding that had transitioned from riveted construction to welded construction. Structural steels were not commercially produced under standard specifications explicitly written for the manufacture of easily weldable structural steels until after circa 1950. A general pattern resulted from these trends: Structural steels produced after World War II generally have good weldability. For steels manufactured before this time, weldability is variable; in some cases, the steel has good weldability, but in other cases, weldability is poor.

Steels manufactured before 1950 may exhibit physical features that adversely affect the soundness of the steel, such as nonmetallic inclusions, stringers, voids of various shapes, tears, and segregation. Steels exhibiting these soundness concerns are commonly referred to as “dirty” steels, particularly when compared to the soundness provided by modern steels manufactured with the intent to be welded. Certain manufacturing processes that were conducive to the production of steel with soundness concerns, such as the acid-Bessemer process that was commercially dominant from 1871 to 1908, were permitted by standard specifications well into the 1960s; these types of processes would not ordinarily be used for manufacturing of steel intended for welding. The last completely new facility for producing acid-Bessemer steel ingots in the United States was built in 1949. However, by 1955, over 90% of the steel produced in the United States was made using the basic open-hearth (BOH) process pioneered by Siemens, which had better capacity to remove phosphorus from the metal than the acid-Bessemer process, making it more suitable for welding. The BOH process remained commercially dominant through the early 1970s when it was replaced by more modern basic oxygen furnace (BOF) and electric arc furnace (EAF) processes. Still, the acid-Bessemer process persisted in use for some time, principally in the production of steel for buttwelded pipe, seamless pipe, free machining bars, flat rolled products, wire, steel castings, and blown metal for the duplex process (AISE, 1998).

Rimmed and capped steel may also have soundness concerns. In particular, structural steels manufactured prior to circa 1930 have an even greater likelihood of exhibiting soundness concerns because control of either chemical composition or manufacturing process with strict regard for weldability was not a commercial concern during that era; instead, riveted structural connections were used. Metallographic examination, as described in the following

Table 8-2. Guidance for Steel Examination and Soundness Concerns		
Existing Steel Classification	**Considerations/Investigations**	**Welding Suggestions**
The existing steel was produced to a known specification that is dated after 1950.	By this era, as-manufactured structural steels were generally sound, even though the steel was not necessarily produced according to a standard specification for structural steels that are intended to be weldable. One exception is structural steel manufactured using the acid-Bessemer process.	There is no compelling reason for metallographic examination for unsound features, although such an examination should be considered if compositional testing of the steel to be welded yields unusual results or if the steel may have been manufactured using the acid-Bessemer process, either of which is potentially indicative of soundness concerns.
The existing steel was produced between 1930 and 1950 inclusive; steel manufacturing specification is known or unknown.	While many structural steels produced during this era were sound, there is nonetheless a chance that some structural steels from this era might contain significant inclusions, such as stringers, and other features that may lead to soundness concerns.	If the existing structural steel in the completed connection could become stressed in the through-thickness direction—whether due to weld shrinkage, structural loading, or a new multipass weld is to be made to the existing steel—nondestructive examination by ultrasonic testing (UT) or magnetic particle testing (MT) of the welded area, or metallographic examination of the existing steel for the presence of inclusions, such as stringers, and other soundness concerns is recommended. If inclusions or other soundness concerns are present, mechanical testing of samples extracted from the structure of the existing steel to be welded may demonstrate acceptable performance of the welded joint that is proposed to be used; otherwise, mechanical connections should be considered.
The existing steel was produced prior to 1930.	There is a significant probability that structural steel from this era may contain significant inclusions, such as stringers, that could lead to lamellar tearing at welded joints. In structural steels produced during this era, such inclusions may occur frequently and may be relatively large. Other soundness concerns may also be present.	Regardless of the welded joint configuration, nondestructive examination by UT or MT of the welded area or metallographic examination of the existing steel should be undertaken to examine for the presence of inclusions, such as stringers, and other soundness concerns. If inclusions or other soundness concerns are present to an extent and severity such that susceptibility to lamellar tearing is heightened, mechanical connections should be considered.

paragraphs and Table 8-2, may be used to assess the soundness of existing structural steel to be welded.

A representative photomicrograph showing unsound features identified as elongated inclusions, or "stringers," in a sample of structural steel obtained from a building constructed circa 1905 is given in Figure 8-1. Inclusions of this nature need to be considered when the retrofit measures involve welding because their presence can readily lead to lamellar tearing where the steel is stressed in the through-thickness direction, whether due to weld shrinkage restraint or applied loads. Weld shrinkage may cause the inclusions to join together and locally tear; this concern is most probable when the weld axis is parallel to the rolling direction. Welded joints perpendicular to the rolling direction may fail in tearing under applied loads.

Inclusions of the nature shown in Figure 8-1 and other soundness concerns that may lead to poor weld performance are detected visually through metallographic examination, not by compositional analysis. Guidance for use of metallographic examination is provided in Table 8-2.

If significant inclusions or other significant soundness concerns are present in an existing structural steel component, connections may be accomplished by mechanical means, such as bolting instead of welding. Alternatively, the suitability for welding could be established by testing the actual steel to be welded. In this case, attention should be given to weld joint design and control of weld shrinkage strains.

The slag that is inherently present in historical structural wrought iron creates weakness in the through-thickness and

Table 8-3. Guidance for Determining Welding Requirements for New Welds to Existing Steel		
Existing Steel Classification	**Considerations/Investigations**	**Welding Suggestions**
The existing steel was produced in accordance with a known standard specification, and the steel is permitted to be used with prequalified WPS by Table 8-1.	None.	Use prequalified WPS in accordance with AWS D1.1/D1.1M, clause 5.
The existing steel was produced in accordance with a known standard specification, but the steel is not permitted to be used with prequalified WPS by Table 8-1. Evidence is available showing that the existing steel was previously welded.	Weldability has been partially established by previous structural welding. It is recommended that at least 10% of the connections, or three connections minimum, be inspected for lamellar tearing of the existing steel and for weld metal cracking. Existing complete-joint-penetration (CJP) groove welds should be inspected with UT. Partial-joint-penetration (PJP) groove welds and fillet welds should be inspected with MT or penetrant testing (PT). The strength level of the existing steel to be welded can be based on the tensile property requirements listed in the standard specification.	If in the recommended inspection at least 80% of the inspected connections meet AWS D1.1/D1.1M acceptance criteria for statically loaded structures, then prequalified WPS in accordance with AWS D1.1/D1.1M, clause 5, may be used, but with preheat levels increased by 50°F (28°C) above the prequalified preheat level for steels with equivalent strength levels. If an 80% passage rate is not achieved in the recommended inspection, preheat should be in accordance with AWS D1.1/D1.1M Annex B, based on the maximum compositional limits specified in the manufacturing specification. WPS should be qualified by test, using a sample of existing steel to be welded that is extracted from the existing structure.
The standard specification used to produce the existing steel is unknown. Evidence is available showing that the existing steel was previously welded.	Weldability has been partially established by previous welding. It is recommended that at least 10% of the connections, or three connections minimum, be inspected for lamellar tearing and weld metal cracking. Existing CJP groove welds should be inspected with UT. PJP groove welds and fillet welds should be inspected with MT or PT. Compositional analysis of the existing steel to be welded is recommended. Additionally, given the unknown manufacturing specification, it is recommended that the strength level of the existing steel be estimated from hardness testing of the steel. Alternatively, samples of the existing steel to be welded could be extracted from the existing structure and subsequently tested for tensile properties.	If in the recommended inspection at least 80% of the inspected connections meet AWS D1.1/D1.1M acceptance criteria for statically loaded structures, then prequalified WPS in accordance with AWS D1.1/D1.1M, clause 5, may be used, but with preheat levels increased by 50°F (28°C) above the prequalified preheat level for steels with equivalent strength levels. If an 80% passage rate is not achieved in the recommended inspection, preheat should be in accordance with AWS D1.1/D1.1M Annex B, based on the composition determined by testing of the steel to be welded. WPS should be qualified by testing, using a sample of existing steel to be welded that is extracted from the existing structure.

Table 8-4. Guidance for Determining Welding Requirements for New Welds to Existing Steel: For Existing Steel That Was Not Previously Welded (Produced under a Known Standard Specification)

Existing Steel Classification	Considerations/Investigations	Welding Suggestions
The existing steel was produced in accordance with a known standard specification, and the steel is permitted to be used with prequalified WPS by Table 8-1.	None.	Use prequalified WPS in accordance with AWS D1.1/D1.1M, clause 5.
The existing steel was produced in accordance with a known standard specification, but the steel is not permitted to be used with prequalified WPS by Table 8-1. The existing steel was not previously welded.	The existing steel standard specification requirements meet all the mechanical and compositional limits of a steel that is permitted to be used with prequalified WPS in AWS D1.1/D1.1M, clause 5.	The existing steel should be welded in accordance with the requirements of a steel with equivalent mechanical and compositional requirements that is permitted to be used with prequalified WPS in AWS D1.1/D1.1M, clause 5.
	The existing steel standard specification requirements do not meet all the mechanical and compositional limits of a steel that is permitted to be used with prequalified WPS in AWS D1.1/D1.1M, clause 5. The specified limits on phosphorus (P) and sulfur (S) are under 0.050%.	Preheat should be determined in accordance with AWS D1.1/D1.1M Annex B, based on the maximum compositional limits stated in the standard specification for the existing steel. WPS should be qualified by testing, using a sample of existing steel to be welded that is extracted from the existing structure.
	The existing steel standard specification requirements do not meet all the mechanical and compositional limits of a steel permitted to be used with prequalified WPS in AWS D1.1/D1.1M, clause 5. The standard specification permits levels of P or S or both to exceed 0.050%. This includes instances where the standard specification for the existing steel does not provide limits on P or S or both. In the case of unregulated levels of P or S or both, the composition of the existing steel to be welded should be determined by testing samples of the existing steel to be welded that are extracted from the structure.	Preheat should be in accordance with AWS D1.1/D1.1M Annex B, based on the maximum compositional limits stated in the standard specification for the existing steel, or as determined from compositional testing. WPS should be qualified by testing, using a sample of existing steel to be welded that is extracted from the existing structure.

lateral directions of the iron (perpendicular to the direction of rolling). As such, mechanical means of making connections should be considered first. If mechanical connections are impractical, welding should be done with caution. The connection and joint should be configured so the wrought iron will not be subjected to through-thickness stress from applied loads (AWS, 2010a). Laminations caused by inclusions in the manufacturing process are usually oriented in the longitudinal direction of members, and welding may be more successful for welded joints transverse to the member axis. AISC Design Guide 21 should be consulted for guidance on detailing joints to avoid lamellar tearing, and the engineer should consider the potential for reduced strength and ductility for loads applied perpendicular to the direction of rolling.

The coarse-grained metallurgy and compositional characteristics of historical gray cast iron are not conducive to fusion welding, although brazing is possible. A welded joint in cast iron is not as strong nor as ductile as the original cast iron itself and should be avoided where possible, although welding might be used for nonstructural architectural and aesthetic purposes where stresses in the weld due to applied forces and weld shrinkage are very low.

The guidance provided in Tables 8-2, 8-3, 8-4, and 8-5 pertains to structural steel and is not intended to be directly applicable to historical structural wrought iron or historical gray cast iron.

Table 8-5. Guidance for Determining Welding Requirements for New Welds to Existing Steel: Existing Steel That Was Not Previously Welded (Produced under an Unknown Specification)		
Existing Steel Classification	**Considerations/ Investigations**	**Welding Suggestions**
The standard specification used to produce the existing steel is unknown. The existing steel was not previously welded.	The composition of the existing steel to be welded should be determined by testing. The mechanical properties of the existing steel should be determined by testing.	If the composition and mechanical properties are consistent with a steel permitted to be used with prequalified WPS in AWS D1.1/D1.1M, clause 5, the steel may be welded in accordance with the prequalified WPS of a steel having equivalent mechanical and compositional requirements that is also permitted to be used with prequalified welds in AWS D1.1/D1.1M, clause 5.
		If the composition and mechanical properties are not consistent with a steel that is permitted to be used with prequalified WPS in AWS D1.1/D1.1M, clause 5, preheat should be in accordance with AWS D1.1/D1.1M Annex B, based on the results of compositional testing. WPS should be qualified by testing, using a sample of existing steel to be welded that is extracted from the existing structure

8.3.1 Chemical Composition Analysis and Determining Preheat Requirements

ASTM A751, *Standard Test Methods and Practices for Chemical Analysis of Steel Products* (ASTM, 2021), outlines the chemical analysis methods used for steel and iron products. The specification provides guidance on the selection of appropriate chemical analysis methods and the requirements for sample preparation, testing, and reporting. Chemical analysis can be completed with a small sample size—approximately the size of a nickel.

When repairing historical steels, the extent of impurities, such as sulfur and phosphorus, will strongly influence the through-thickness properties, and certain uncontrolled chemicals in early steel making—such as manganese, carbon, or silicon—can affect hardness and inhibit weldability. Chemical analysis can quantify these properties.

AWS D1.7/D1.7M outlines carbon-equivalent formulas for assessing the likelihood of cracking during welding or weld cooling based on hardness and hydrogen control. In addition to chemistry, local factors such as cooling rate, joint geometry, and restraint will affect the potential for cracking, and the nature of the joint needs to be considered in conjunction with chemical analysis to evaluate weldability.

AWS D1.1/D1.1M Annex B provides guidance on preheat, interpass temperature controls, and electrode selection that can be effective crack mitigations for welding to uncontrolled steels. The preheat required to avoid cracking

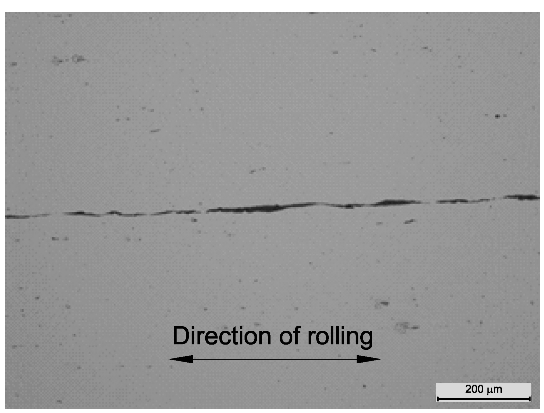


Fig. 8-1. Representative photomicrograph showing elongated inclusions, or "stringers," in structural steel manufactured circa 1905 from AISC 342.

is unlikely to be achievable in the field for highly susceptible materials.

Further guidance on this topic is provided in AISC Design Guide 21.

8.3.2 Trial Tests

AWS D1.7/D1.7M provides several weldability testing options, including both simulated and actual weld test options. However, the results of these tests should be considered qualitative. These tests may be useful to evaluate the potential for hot cracking, delayed cracking, or the effects of welding on the heat-affected zone.

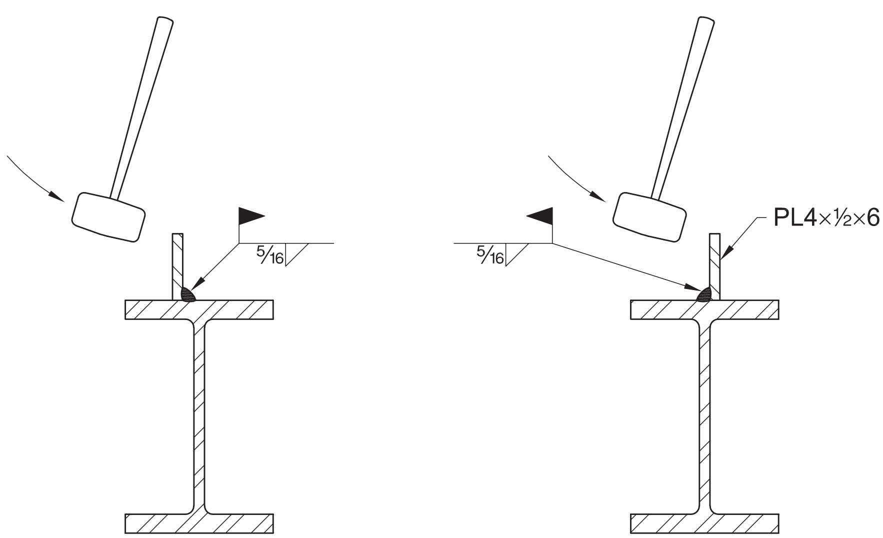
Tab plate tests or fillet bend tests—illustrated in Figure 8-2—can be useful for assessing the weldability of steel and can serve as an initial quality assurance mechanism for a welding procedure determined as discussed in Section 8.2. The tab plate test involves welding a small steel plate, or tab, to a larger steel plate and then striking the sample with a hammer to put the fillet weld into tension. The test aims to evaluate the strength and quality of the welded connection between the tab and the larger plate. If the weld deforms without fracturing, the steel can be considered weldable. If the weld separates from the base metal at the junction of the weld and base metal, it indicates the base metal is subject to hardening, often the result of high carbon content (Ricker, 1988).

Tab plate tests are of limited accuracy and should be treated as such but can be used to guide prompt decisions on weldability and supplemented by chemical analysis, as appropriate. AWS D1.1/D1.1M historically outlined the procedures and acceptance criteria for tab plate tests but has since withdrawn the approach in favor of more determinate methods of assessing weldability.

8.4 REPAIR UNDER LOAD

AWS D1.1/D1.1M, clause 8.3.5, requires that "The engineer shall determine the extent to which a member will be permitted to carry loads while heating, welding, or thermal cutting is performed." The heat from welding or thermal cutting will reduce the yield strength, tensile strength, and modulus of elasticity while the weld and surrounding steel are hot. The reduced modulus may increase the deflection of members under load. The reduced strength and stiffness properties of steel at elevated temperatures vary slightly by grade, but AISC *Specification* Table A-4.2.1 provides reduction factors for yield, elastic modulus, and proportional limit of common structural carbon steel material grades. The elastic modulus begins to reduce above 300°F (150°C); however, yield stress is not appreciably affected until the steel temperature is above 750°F (400°C). For stainless steels or steels that have been quenched and tempered, consult with the more detailed presentation of material strength reductions in the ASME *Boiler and Pressure Vessel Code* (ASME, 2023).

Options to avoid welding members under load should be investigated. When welding to members under load is unavoidable, the engineer should consider strength and stiffness reductions in the heat-affected zone of welds, the residual stresses induced by weld cooling, and joint details that minimize these effects. The application of heat may cause thermal displacement or local weakening of sections, and



(a) Test for weld ductility *(b) Test for hardening of base metal*

Fig. 8-2. Tab plate test (from Ricker, 1988).

the engineer should consider the potential for distortion or displacement of members and the potential for load redistribution during welding. Shoring or load reductions during welding may be required.

In the seminar "Design of Reinforcement for Steel Members Part 2," Dowswell (2016) proposed some approaches that engineers might specify to limit the heat input during welding:

- Specify intermittent welding in short lengths.
- Specify time for welds to cool between passes.
- Specify the use of stringer beads only (rather than weave beads).

Additionally, Dowswell proposes several WPS-related recommendations that could be considered in dialogue with the contractor:

- Consider small diameter electrodes to limit heat input.
- Specify a maximum interpass temperature appropriate for the conditions.
- Monitor the base metal temperature near the weld with temperature crayons or other suitable means.
- Consider shielded metal arc welding (SMAW) rather than flux-cored arc welding (FCAW) electrodes.

Fracture potential is reduced by configuring joint details to minimize restraint induced by weld cooling. Conditions sensitive to residual stress, such as multiaxial welded joints, warrant greater attention. Similarly, plates over 1½ in. thick will likely require high heat input for welding and the plate material may have uncontrolled centerline toughness, warranting closer attention to fracture potential.

Vild et al. (2019) performed experiments on column stability during welding by measuring the effect of plates welded parallel to the member axis of a column under load. They considered that welding renders a part of the cross section ineffective during welding due to the high applied temperature, which causes the center of gravity to be shifted, inducing additional moment on the column. They accurately predicted the performance of columns under load when discounting the portion of the member heated higher than 500°C (932°F) and evaluating the resulting section of increased slenderness as one would evaluate a stepped member with second-order effects. The paper builds on work from Rosenthal (1946) and Huenersen et al. (1990).

Rosenthal (1946) provided a generalized procedure for evaluating heat gradients in metals. As noted in the procedure:

1. The rise in temperature in front of the heat source is much steeper than the fall in temperature behind the heat source. That is, cooling does not occur instantaneously and can take tens of seconds to several minutes, depending on the heat input and size of the welded parts.
2. Because heat does not propagate instantly in metals, the temperature does not pass through the maximum at the same time at various points of the same cross section. There is a lag at points that are farther away from the weld, which is a function of the welding speed and the metal's diffusivity.
3. An increase in welding speed produces a greater lag in the temperature distribution at a given cross section; hence, heat is more concentrated around the heat source. However, the temperature distribution along the line of welding behind the electrode remains unchanged.
4. An increase in the current intensity extends the range of heat in the solid but does not affect the shape of the isotherms.
5. An increase in temperature (preheat) causes a corresponding increase in the numerical value of each isotherm but does not change its shape or size.

Huenersen et al. (1990) propose a detailed procedure for assessing the temperature field of 36 ksi and 50 ksi welded carbon steel components. They propose a generalized temperature field and correction factors for weld process type, 2D vs. 3D heat flow, material thickness, welding velocity, and weld length. Figure 8-3 presents their general procedure (note that the equations in Figure 8-3 are nondimensional and must be used with variables in the units provided here and on the subsequent page). The procedure can be used to estimate the extent of an isotherm for a particular weld process heat input and welding speed. Additional guidance in the form of nomographs is available in the referenced paper and may be more convenient for quickly evaluating thermal effects.

C_x = normalizing coefficient to account for welding and joint variables in x-axis

C_y = normalizing coefficient to account for welding and joint variables in y-axis

F_L = weld length factor, from Figure 8-3

F_M = base metal grade factor, from Figure 8-3

F_N = weld factor from Table 8-7

I = amperage, from weld procedure specification, A

T = critical temperature isotherm under consideration, °F

$T_{8/5}$ = cooling time from 800°C to 500°C (1,470°F to 932°F), s

U = voltage, from weld procedure specification, V

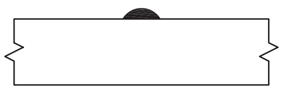
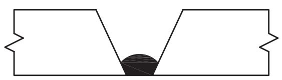
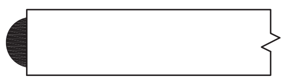
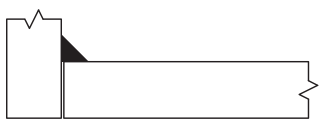
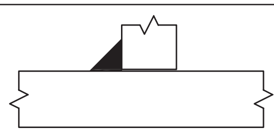
l_w = length of weld, in.

q_s = heat input in relation to arc power, J/in.

$$= \eta\left(\frac{IU}{v_s}\right)$$

t = thickness of material, in.

Table 8-6. Relative Heat Efficiency Factor, η (adapted from Huenersen et al., 1990)	
Welding Process	**η**
Shielded metal arc welding (SMAW) and flux cored arc welding (FCAW)	0.9
Shielded metal arc welding (SMAW), low-hydrogen electrode	0.8
Gas metal arc welding (GMAW), CO_2 shielding gas	0.8–0.9
Gas metal arc welding (GMAW), argon shielding gas	0.6–0.7

Table 8-7. Weld Factor, F_N (Huenersen, et al., 1990)	
Type of Weld	**Weld Factor, F_N**
	1.0
	0.90
	2.0
	0.90–0.67
	0.45–0.67

t_{eff} = effective thickness, in.
$t_{2/3}$ = limiting thickness accounting for 2D versus 3D heat flow, in.
v_s = welding velocity, from weld procedure specification, in./s
x_T = temperature parameter in the longitudinal axis of the weld from Figure 8-4, in.
x_i = affected width in the longitudinal axis of the weld, in.
y_T = temperature parameter transverse to the axis of the weld from Figure 8-4, in.
y_i = affected width transverse to the axis of weld, in.
η = relative heat efficiency factor from Table 8-6

With knowledge of the estimated size of the critical heat-affected temperature field around the weld, an engineer can determine an effective net section for strength and stiffness evaluations. The effective net section is the remaining section after discounting the portions of the section with temperature exposures above a critical level. The selection of a critical temperature for the material that should be neglected in a strength or stiffness evaluation requires engineering judgment; however, Huenseren et al. imply that 500°C (932°F) may be considered critical for such an assessment, and experimental research by Vild et al. supports this value. At 500°C, AISC *Specification* Table A-4.2.1 suggests that steel will retain about 75% of its yield strength and 55% of its elastic modulus, which, when coupled with practical accommodations to limit heat input, may typically be acceptable for evaluating effects from the transient heat of welding.

Inspection of results from Huenersen et al.'s procedure demonstrates, among other things, that the reduction in carrying capacity depends on the location of the heated areas in the cross section. Longitudinal single-pass welds running parallel to the member axis and parallel to the governing stress may experience critical temperatures in a limited portion of the cross section. Temperature changes near transverse welds of flanges may result in quickly heating the entire flange cross section to a critical level. Inspection of welding effects by a rigorous approach supports conventional industry guidance to avoid cross-flange welding of members under load when possible.

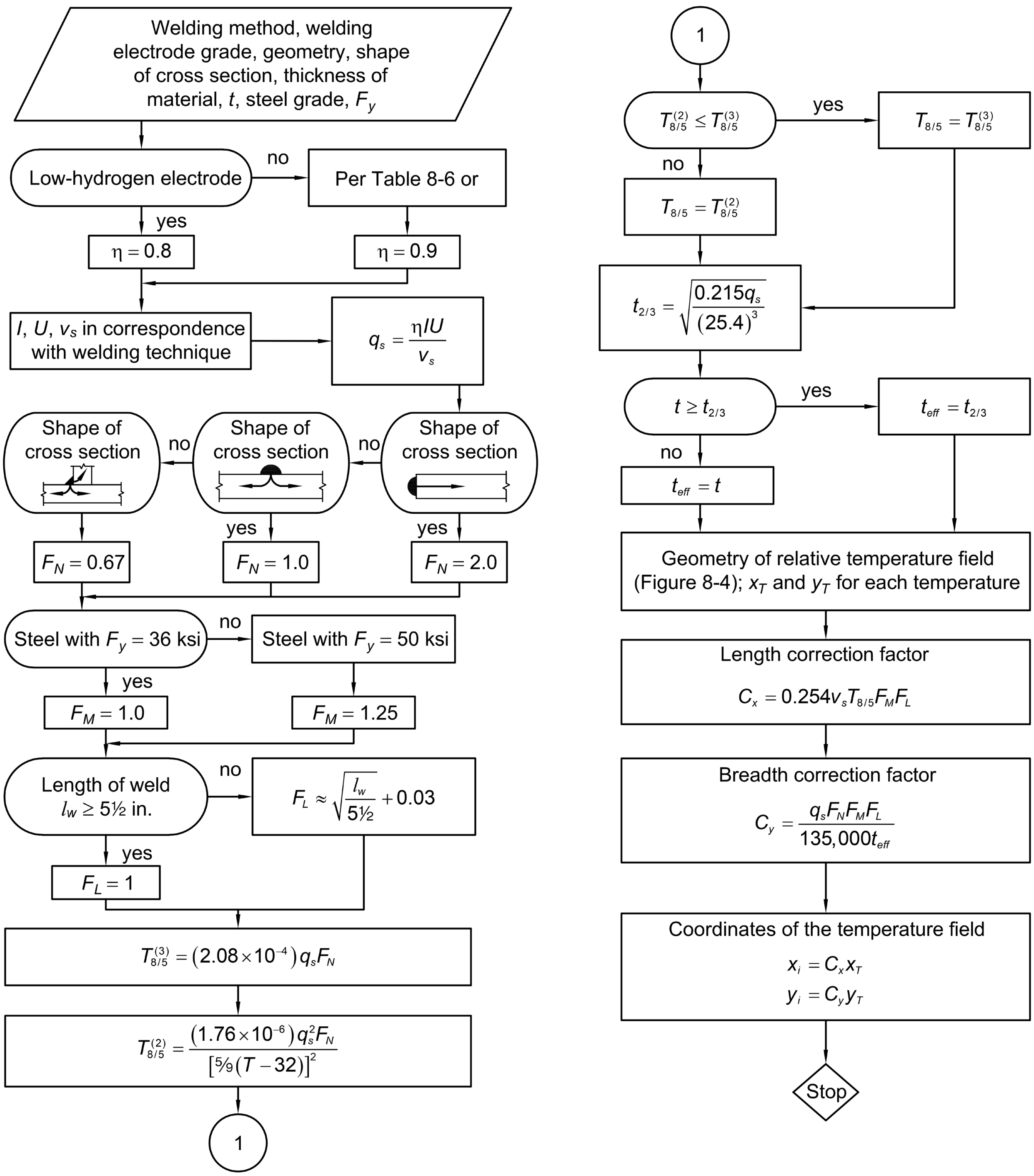


Fig. 8-3. Flow chart for determining the coordinate transformations necessary to modify the generalized temperature field in Fig. 8-4 (adapted to U.S. customary units from Huenersen et al., 1990).

EXAMPLE 8.1—APPLICATION OF HUENERSEN ET AL.

Given:

A 3-in.-thick ASTM A992/A992M (ASTM, 2022b) column flange is welded at its flange tip with a 6-in.-long longitudinal weld to add a cover plate. Estimate the extent of the flange that is affected by temperatures exceeding 500°C (932°F) in the cross-flange direction, x_i, and in the through-thickness direction, y_i, during welding. This connection is shown in Figure 8-5.

Solution:

The following welding variables are used:

Electrode classification	= E7018 (low hydrogen)
Current, I	= 120 A
Voltage, U	= 25 V
Welding velocity, v_s	= 5 in./min
	= 0.0833 in./s
Weld length, l_w	= 6 in.

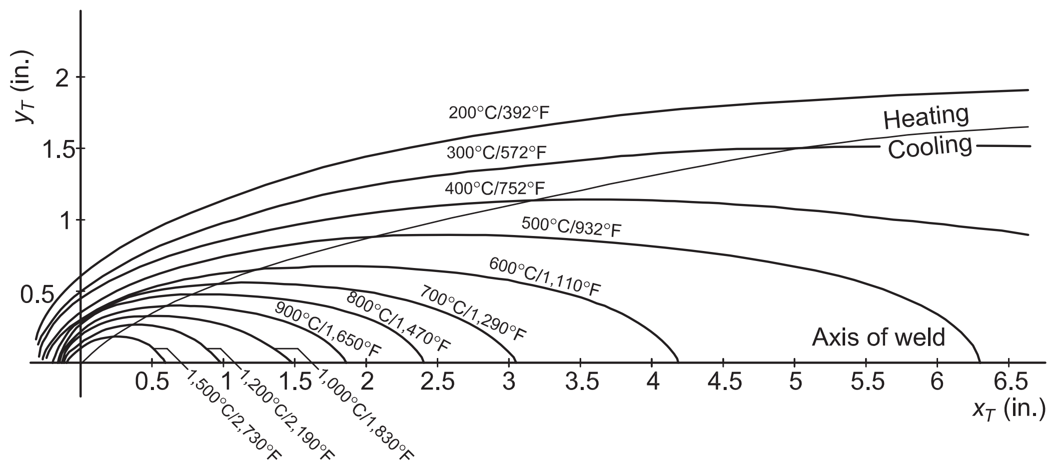


Fig. 8-4. Generalized temperature field (adapted to U.S. customary units from Huenersen et al., 1990).

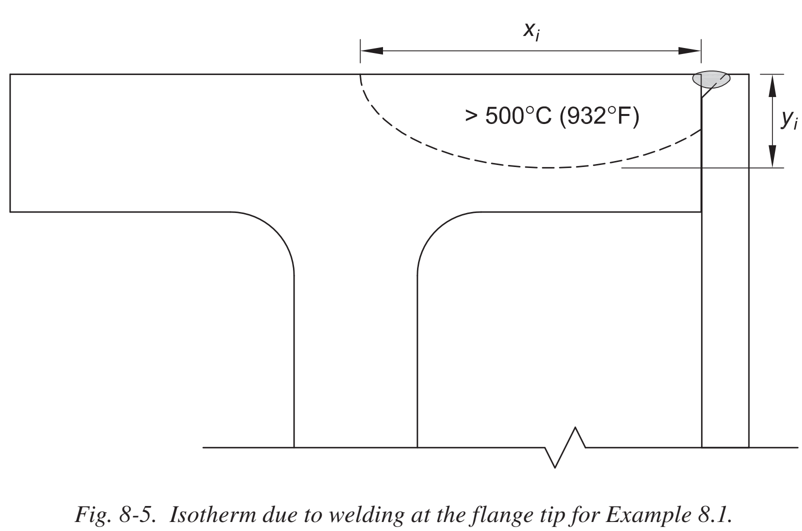


Fig. 8-5. Isotherm due to welding at the flange tip for Example 8.1.

From the flowchart in Figure 8-3:

$F_N = 2$

$F_M = 1.25$

$F_L = 1$

$\eta \;= 0.8$

In keeping with the approach presented in Huenersen et al. (1990), nondimensional scalar variables are applied in computations consistent with the definitions in the method.

The heat input in relation to arc power is:

$$\begin{aligned} q_s &= \frac{\eta IU}{v_s} \\ &= \frac{0.8(120\text{ A})(25\text{ V})}{0.0833\text{ in./s}} \\ &= 28{,}800\text{ J/in.} \end{aligned}$$

The calculated cooling time from 800°C to 500°C is the larger of:

$$\begin{aligned} T_{8/5}^{(3)} &= \left(2.08\times10^{-4}\right)q_s F_N \\ &= \left(2.08\times10^{-4}\right)(28{,}800\text{ J/in.})(2) \\ &= 12.0\text{ s} \end{aligned}$$

$$\begin{aligned} T_{8/5}^{(2)} &= \frac{\left(1.76\times10^{-6}\right)q_s^2 F_N}{\left[\tfrac{5}{9}(T-32)\right]^2} \\ &= \frac{\left(1.76\times10^{-6}\right)(28{,}800\text{ J/in.})^2(2)}{\left[\tfrac{5}{9}(932\text{°F}-32)\right]^2} \\ &= 0.0117\text{ s} \end{aligned}$$

Thus,

$$T_{8/5} = 12.0\text{ s}$$

The limiting thickness accounting for 2D vs. 3D heat flow is:

$$\begin{aligned} t_{2/3} &= \sqrt{\frac{0.215 q_s}{(25.4)^3}} \\ &= \sqrt{\frac{0.215(28{,}800\text{ J/in.})}{(25.4)^3}} \\ &= 0.615\text{ in.} \end{aligned}$$

$t_f > t_{2/3}$, therefore, the effective thickness is:

$$\begin{aligned} t_{eff} &= t_{2/3} \\ &= 0.615\text{ in.} \end{aligned}$$

From Figure 8-4, the extent of the generalized 500°C (932°F) isotherm before accounting for normalizing variables is approximately:

$$x_T = 6.3 \text{ in.}$$

$$y_T = 0.85 \text{ in.}$$

The normalizing coefficients to account for welding and joint variables in the *x*- and *y*-axes, respectively, are:

$$\begin{aligned} C_x &= 0.254 v_s T_{8/5} F_M F_L \\ &= 0.254(0.0833 \text{ in./s})(12.0 \text{ s})(1.25)(1) \\ &= 0.317 \end{aligned}$$

$$\begin{aligned} C_y &= \frac{q_s F_N F_M F_L}{135{,}000 t_{eff}} \\ &= \frac{(28{,}800 \text{ J/in.})(2)(1.25)(1)}{135{,}000(0.615)} \\ &= 0.867 \end{aligned}$$

The estimated extent of the flange in the cross-flange direction, x_i, and the through-thickness directions, y_i, that is affected by temperatures exceeding 500°C (932°F) during welding:

$$\begin{aligned} x_i &= C_x x_T \\ &= 0.317(6.3 \text{ in.}) \\ &= 2.00 \text{ in.} \end{aligned}$$

$$\begin{aligned} y_i &= C_y y_T \\ &= 0.867(0.85 \text{ in.}) \\ &= 0.737 \text{ in.} \end{aligned}$$

8.5 REPAIR OF DEFORMED ELEMENTS

8.5.1 Cold Bending

AISI Research Report RP98-1, "Fabrication Guidelines for Cold Bending" (AISI, 2022), and Brockenbrough (2006) provide guidance for cold-bending members and plates. As the preface to AISI Research Report RP98-1 indicates, "to avoid cracking plates during cold bending it is necessary to adopt a suitable minimum inside bend radius, which typically varies with plate thickness and grade." Table 8-8 displays the guide's updated recommendations for minimum radii of cold-bent members based on a recent experimental program accounting for the tensile strength of higher-strength plates.

The AISI guidance is based on ASTM A36/A36M steel data and factored based on the measured elongation of material tensile test samples tested to failure, which averaged from 14 to 46% over a 2 in. length. The guidance suggests that strain limits associated with cracking failure occur at 0.5 to 0.8 in./in. for ½-in.-thick ASTM A36/A36M steel. The radius-to-thickness limits for other steel plate thicknesses were proportioned based on the bend test procedures from ASTM A6/A6M.

The AISI guidance relies heavily on extrapolating a limited dataset and may warrant additional direct experimental verification or project-specific testing for cold bending thicker and higher-strength plates.

Related work by Alan Pense at Lehigh University investigated the effects of cold working on the material properties of steel. A summary of his experimental work, delivered at the 2012 National Steel Bridge Alliance (NSBA) World Bridge Symposium (Pense et al., 2012), conveyed that cold working material causes the yield and tensile stress of the material to increase (yield stress, on average, increases 20 to 21% for the tested steels when subjected to 1 to 3% elongation); however, related effects of strain hardening cause ductility and material toughness to decrease. Similarly, the transition temperature of the material CVN toughness increases in cold-worked materials. Pense et al. concluded that stress relieving is ineffective at restoring the ductility of cold-worked materials because strain aging of the material has altered the metallurgical properties of the deformed

Table 8-8. Suggested Minimum Inside Bend Radius for Cold Forming (AISI, 2022)				
Material	Thickness, t, in.			
	$t \leq$ ¾ in.	¾ in. $< t \leq$ 1 in.	1 in. $< t \leq$ 2 in.	2 in. $< t \leq$ 3 in.
ASTM A36/A36M	1.5t	1.5t	1.5t	2.0t
ASTM A572/A572M Gr. 50	1.5t	1.5t	2.0t	2.5t
ASTM A588/A588M	1.5t	1.5t	2.0t	2.5t
ASTM A656/A656M Gr. 70	1.5t	1.5t	—	—
ASTM A514/A514M	1.75t	2.25t	4.5t	5.5t

steel. He notes this to be the case whether the bending is induced by cold working or heat bending and that additional bending, as may be employed in an attempt to straighten members, will continue to compound the effects of strain hardening and strain aging until the point where the ductility of the component has been exhausted.

It is reasonable to expect cold bending to be successful for deformed elements that exceed the minimum bend radius limits tabulated in AISI Research Report RP98-1. Material that is straightened by cold or thermal means should be inspected for cracks after bending, and the engineer should consider the effects that reduced ductility may have on the performance of the straightened component in service.

8.5.2 Heat Straightening

FHWA Report No. FHWA-IF-08-999, *Guide for Heat Straightening of Damaged Steel Bridge Members* (FHWA 2008), provides detailed guidance on the heat straightening of members. As discussed in the guide, heat straightening is a repair method that involves the gradual straightening of members by applying successive cycles of heating and cooling to the plastically deformed regions of damaged steel; the resulting thermal expansion and contraction result in the gradual straightening of the member. The following conditions characterize the process:

- The temperature of the steel does not exceed either (1) the lower critical temperature (the lowest temperature at which molecular changes occur) or (2) the temper limit for quenched and tempered steels.
- The stresses produced by applied external forces do not exceed the yield stress of the steel in its heated condition.
- Only the regions in the vicinity of the plastically deformed zones are heated.

When these conditions are met, the material properties undergo relatively small changes, and the steel's performance remains essentially unchanged after heat straightening. Properly conducted heat straightening is a safe and economical procedure for repairing damaged steel. Heat-straightening repairs have been conducted for strains up to 100 times the yield strain.

The engineer should be aware that heat straightening is distinctly different than hot mechanical straightening or hot working, and the field procedures should be aligned appropriately. Hot mechanical straightening or hot working methods involve an external force applied after heating to straighten the damaged member. The results of hot mechanical straightening or hot working are unpredictable and can result in fracture, reduced member strength, and additional mechanical deformations. Such approaches are typically inappropriate for repairing damaged structural steel and should be considered only for non-load-carrying elements or when replacement or other methods are impossible.

The FHWA guide presents methods of heat straightening that apply to various categories of deformations, including:

- Category S (strong-axis bending)
- Category W (weak-axis bending)
- Category T (torsional twisting)
- Category L (local damage)

The categories align to heating patterns applicable to each bending type. See FHWA (2008) for more details. AWS D1.7/D1.7M provides additional guidance on heat straightening.

8.5.3 Damaged Anchor Rods

As discussed in AISC Design Guide 1 (Kanvinde et al., 2024), field straightening of rods should be limited to those that are 36 ksi material or less and with a bend angle less than 45°; the maximum temperature permitted for hot bending ASTM F1554 Gr. 36 (ASTM, 2020) material is 1,200°F. Anchor repair or extension by welding should only be performed for material grades that are specified as weldable—for example, those purchased to ASTM F1554 Gr. 55 Supplement 1.

Beyond these limited cases, anchor rod repairs may require supplementing the condition with post-installed anchorage or rebuilding the base condition. In space-limited

Table 8-9. Common Painting Systems for Various Environmental Conditions	
Dry interiors where steel is protected	None
Interiors, normally dry	None or latex
Exteriors, normally dry	Oil base or latex
Frequent fresh-water wetting, spray, immersion	Vinyl, coal-tar epoxy, epoxy
Frequent salt-water wetting, spray, immersion	Zinc-rich, vinyl, coal-tar epoxy, epoxy
Fresh-water or salt-water immersion	Vinyl, coal-tar epoxy
Chemical exposure, neutral (pH 5 to 10)	Vinyl, chlorinated rubber, zinc-rich
Chemical exposure, acid (pH 2 to 5)	Vinyl, coal-tar epoxy, chlorinated rubber
Chemical exposure, alkali (pH 10 to 12)	Coal-tar epoxy, chlorinated rubber
Chemical exposure to mild solvents	Epoxy
Note: This information is compiled from Newman (2000).	

conditions or when the anchorage is controlled by concrete strength, the most practical solution may involve shortening the steel column and building a concrete pedestal or jacket on top of the existing concrete base to supplement or replace the anchorage.

8.6 COATINGS

When additional exposure is likely or aesthetic concerns require remediation of corroded steel, there are several methods of surface preparation and coatings for the repair of corroded steel. Detailed guidance on coating system selection and design is available from publications by the Association for Materials Protection and Performance (AMPP)—formerly the Society for Protective Coatings (SSPC). The information presented in Table 8-9 is a cursory introduction to some of the applicable terms and general considerations.

8.6.1 Surface Preparation

Successful coating performance requires a properly prepared surface. Surface preparation should be specified to meet measurable criteria, including removing rust that painters may otherwise consider to be "tightly adhered."

Surface preparation may be accomplished by mechanical or chemical means. The selection of the appropriate method of surface preparation and coating will depend on the specific conditions and requirements of the repair project and the type and extent of corrosion damage on the steel surface.

Mechanical preparation involves abrasive blasting or grinding to remove surface contaminants and corrosion products. Mechanical preparation is often the most effective method for preparing steel surfaces for repair.

Chemical cleaning involves using acids or other chemicals to dissolve surface contaminants and corrosion products. Chemical cleaning can be effective for removing heavy rust and other contaminants but may not be suitable for all types of steel surfaces.

8.6.2 Coating Systems

Coating systems typically require multiple coats for full effectiveness. A primer provides first protection until other coatings are applied and provides bonding to existing steel. The primer is essential to the overall performance of the system. A top coat provides a coating system's overall protection, and an additional intermediate coat is often applied to add to a system's protection and durability. The primer, top coat, and intermediate coat should be specified using compatible systems.

Oil-based coatings may be used as primers and cured by air oxidation. Oil-based coatings may be acceptable for mild environments but often must be combined with zinc-rich primers. Coatings with zinc-rich primers are most useful on steel exposed to the exterior, such as lintels.

Latex coatings are resin suspended in water. When the water evaporates, the resin fuses together. Latex coatings should only be specified in normally dry environments.

Epoxy coatings form a chemical reaction between two components. These systems may not perform well when exposed to direct sunlight. Epoxy coatings may be preferred in interior conditions and are often used for repairing corroded steel in marine environments, as they provide excellent protection against saltwater corrosion.

Conventional rust reformers are not typically effective at preventing the recurrence of corrosion in structures.

8.7 COMPOSITE REPAIRS

Concrete jacketing, filling, and bonding of existing noncomposite steel members to concrete can be effective ways to improve an existing member's strength. Any composite system needs to consider force transfer between the existing and added components, the materials' long-term creep, and the materials' strength when subjected to fire.

Existing concrete encasement found in early steel buildings was provided primarily as fire protection and may not have been intended for strength. Such encasements may be low strength and not sufficiently confined or detailed to achieve composite action. Supplemental confinement of the encasement may allow for some consideration of composite strength.

In addition to concrete composites, there have been recent technical advances in bonded repair materials, such as carbon fiber-reinforced polymers. Such systems lose strength when subjected to high temperatures and cannot be relied upon to maintain their strength when exposed to fire.

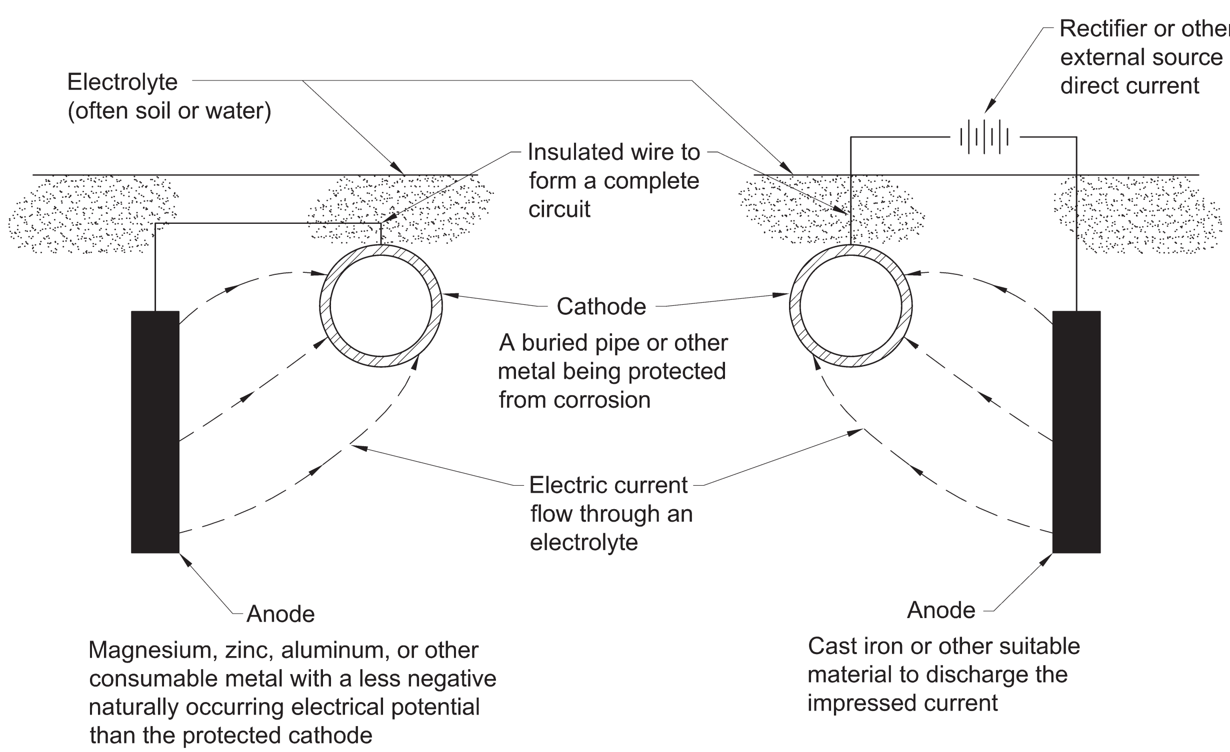
Repair procedures for composite structures are discussed in ACI 562, *Assessment, Repair, and Rehabilitation of Existing Concrete Structures—Code and Commentary* (ACI, 2021).

8.8 CATHODIC PROTECTION

As discussed earlier, corrosion is an electrochemical reaction wherein an uncharged metal atom loses one or more of its electrons and becomes a charged metal ion that transfers from an anode to a cathode in an electrochemical reaction. Cathodic protection involves minimizing the flow of ions between an existing anode and cathode by inducing a stronger electrical current from an external source.

Cathodic protection can be an effective method for preventing or inhibiting the progression of steel corrosion in specific environments but requires the free transfer of ions between an anode and a cathode in an electrolyte. Common electrolytes are soil, concrete, or water, making cathodic protection effective in buried, encased, and submerged conditions, with buried pipelines being the most common application. The electrical charge required for cathodic protection may be provided through galvanic or impressed current systems, as schematically described in Figure 8-6.

Galvanic cathodic protection relies on differences in the electrical potential of materials to create a cathode and anode



(a) Galvanic anode system *(b) Impressed current system*

Fig. 8-6. Cathodic protection systems.

in a circuit. Sacrificial anodes with higher electrical potential than the material being protected (for steel, often zinc or magnesium) are electrically connected in the presence of an electrolyte to form a complete circuit. The current density degrades with time and requires consideration of the need for anode replacement within the expected service life of the structure. AMPP standard NACE SP0216-2023, *Galvanic Cathodic Protection of Reinforcing Steel in Atmospherically Exposed Concrete Structures* (AMPP, 2023), is a useful guide to designing galvanic cathodic protection systems.

Impressed current cathodic protection follows similar principles but relies on externally applied electrical current to achieve differences in electrical potential. The anode in an impressed current system may be cast iron or other similarly suitable material that is connected to a rectifier or other external source of direct current. Impressed current cathodic protection is an active system that requires ongoing maintenance and electrical continuity for effectiveness, and alternative systems should be considered when ongoing maintenance cannot be ensured.

Effective cathodic protection system designs require knowledge of the electrical properties of the components and the installed environment. As such, the systems should be designed by specialized consultants experienced in these system designs.

Chapter 9
Construction Documents

9.1 DESIGN DOCUMENTS

AISC *Specification* Section A4 provides a list of items that are required to be defined on design documents. For repair or remediation projects, the engineer should consider providing the following additional project information when applicable:

- Properties of existing and remedial materials used for the project
- The scope of repair or rehabilitation
- Description of the codes and standards to which the repair or alteration design conforms, including the applicable code year(s)
- IEBC compliance method
- Design-basis code criteria for rehabilitation/repair design per IEBC for:
  - Additions
  - Alterations, including level (i.e., level 1, 2, or 3)
  - Change of occupancy
  - Repairs, including structural damage classification (i.e., substantial or less than substantial)
- Intended remaining service life
- Special inspection requirements beyond AISC *Specification* Chapter N
- Special quality assurance and quality control requirements beyond AISC *Specification* Chapter N
- Maintenance program requirements
- Monitoring requirements
- Types and frequency of future inspection or monitoring
- Types and frequency of future maintenance
- Unique welding or bolting requirements
- Delegated design requirements for shoring
- Required installation sequences

9.2 SPECIAL DESIGN REQUIREMENTS FOR REPAIRS AND STRENGTHENING

Many repair and strengthening designs require specific installation procedures for successful implementation to ensure that load is transferred between existing and new parts efficiently and that temporary conditions associated with a repair are appropriately managed. Designs that rely on unique construction means and methods for demolition or installation should provide an adequate description of those requirements. Loads for temporary conditions delegated to others, such as unique construction shoring requirements, should be provided on the design documents. To avoid contractual concerns during implementation, in some cases it may be preferable to provide installation details as guidance rather than requirements and allow contractors to choose the final sequence and installation approach in accordance with their preferred means and methods.

9.2.1 Construction Sequence Requirements

When specific sequences or construction activities are required to implement the design, recommended sequences should be indicated on the construction documents.

9.2.2 Construction and Shoring Requirements

When shoring is required to implement the design, the recommended shoring location and loading should be indicated on the construction documents.

9.2.3 Loads during Retrofit

When loads are required to be limited during or prior to installation, the load limitations should be indicated on the construction documents. Similarly, when loads are required for the contractor to complete delegated design items, the loads should be provided on construction documents.

9.3 QUALITY ASSURANCE REQUIREMENTS

Inspection requirements in the IBC and AISC *Specification* Chapter N are generally written to address new construction projects. Additional and in-progress inspections may be required for repair projects. When required, the contract documents should identify hold points for inspection.

Symbols

A Original cross-sectional area of the uncorroded member (net or gross), in.^{2}

A_d Remaining cross-sectional area of the corroded member (net or gross), in.^{2}

A_g Gross area of the cross section, in.^{2}

C_x Normalizing coefficient to account for welding and joint variables in the *x*-axis

C_y Normalizing coefficient to account for welding and joint variables in the *y*-axis

E Modulus of elasticity of the member, ksi

E_{ci} Modulus of elasticity of cast iron, ksi

F_{avg} Average of test values, ksi

F_{cr} Critical stress, ksi

F_e Elastic buckling stress, ksi

F_L Weld length factor

F_M Base metal grade factor

F_{min} Equivalent specified minimum strength, ksi

F_N Weld factor

F_u Specified minimum tensile strength, ksi

F_y Specified minimum yield stress, ksi

I Amperage from weld procedure specification, A

I Original moment of inertia of the uncorroded member, in.^{4}

I_d Remaining moment of inertial of the corroded member, in.^{4}

L_c Unbraced length of compression member, in.

P_{cl} Minimum compressive strength of a cast iron column, kips

R Adjustment factor

RCF_L Local residual capacity factor

RCF_{Lb} Residual local bending capacity factor

RCF_{Lc} Residual local compressive capacity factor

RCF_{Lt} Residual local tensile capacity factor

RCF_{Lv} Residual local shear capacity factor

RCF_M Member residual capacity factor

RCF_{Mb} Residual bending capacity factor

RCF_{Mc} Residual compressive capacity factor

RCF_{Mt} Residual tensile capacity factor

RCF_{Mv} Residual shear capacity factor

S_d Remaining section modulus of the corroded member, in.^{3}

T Critical temperature isotherm under consideration, °F

$T_{8/5}$ Cooling time from 800°C to 500°C (1,470°F to 932°F), s

U Voltage from weld procedure specification, V

Z Original plastic section modulus of the uncorroded member, in.^{3}

a Original crack length, in.

b Width of element, in.

k Lower tolerance limit factor, a function of n, p, and γ

l_w Length of weld, in.

n Number of samples (statistical sample size)

p Proportion of test data falling above the lower limit

q_s Heat input in relation to arc power, J/in.

r Radius of gyration, in.

r_d Reduced radius of gyration of the corroded member, in.

t Thickness, in.

t_{eff} Effective thickness, in.

$t_{2/3}$ Limiting thickness accounting for 2D versus 3D heat flow, in.

t_d Remaining thickness of the corroded element, in.

t_w Original web thickness of the uncorroded member, in.

t_{wd} Remaining web thickness of the corroded member, in.

v_s Welding velocity from weld procedure specification, in./s

x_i Affected width in the longitudinal axis of the weld, in.

x_T Temperature parameter in the longitudinal axis of the weld, in.

y_i Affected width transverse to the axis of the weld, in.

y_T Temperature parameter transverse to the axis of the weld, in.

γ Confidence interval

η Relative heat efficiency factor

σ_{test} Standard deviation of the sample of test values

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