STEEL, WELDING AND CONSTRUCTION SPECIFICATIONS FOR SEISMIC STRUCTURES

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ABSTRACT

The Northridge and Kobe earthquakes taught us that toughness is important for structural steel in seismically-active areas. While design drawings describe the geometry of a structure, material properties are defined in the construction specification, although this document is often overlooked. For structural joints with significant loading, a fracture control plan (FCP) can provide the basis for the construction specification requirements. The background of some relevant FCPs are reviewed.

Two examples of construction specifications for steel structures subject to earthquakes are compared. The first was developed for a fixed platform structure offshore from New Zealand that is subject to severe cyclic loading from storms and to high maximum loads from seismic events. The second example is from a typical medium-rise, steel framed building in San Francisco that is subject to earthquakes. The conservative basis for the first specification arises from the crack initiation and propagation action of cyclic stresses from storms, the possibility of a maximum seismic load on the resulting cracks, and the precedent of marine and offshore steel selection and welding rules. The liberal basis for the second is shown to stem from the absence of cyclic loading and the precedent of U.S. building codes and construction practice.

A four-tiered approach to material selection, welding procedures and practice, and construction specification requirements is presented. The first tier, which is represented by simple structures with primarily only dead load, uses common structural steels and consumables welded to AWS D1.1. The second tier, which includes modest structures with some dynamic loads, requires the use of toughness-tested steels and welding consumables. The third tier, which includes bridges and high-rise buildings subject to significant seismic or other loads, requires welding procedures qualified with impact toughness testing. The fourth tier includes critical areas of some offshore and similar structures that are subject to both cyclic and seismic loading and that are essentially inaccessible during their design life. Steels with fracture toughness (e.g., CTOD) that is pre-production qualified at the steel mill are used with welding procedures qualified by fracture toughness tests, in addition to impact tests, and controlled using appropriate additional essential variables for this fourth tier. This system is intended to clarify the development of project specifications by the engineer.

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INTRODUCTION

Construction specifications are an important but often overlooked part of engineered structures. Design drawings describe the geometry and details of a structure, but the materials, quality level and workmanship are generally defined in the specifications. New drawings are always generated for new structures, but specifications for new buildings are often copied from previous projects without significant review. For example, the specification for a moderately-sized public building that was designed in 1998 referred to AWS 1.0, Code for Welding in Building Construction, which was replaced by the D1.1 Structural Welding Code in 1972, 25 years ago! Clearly, the underlying basis for many of the construction requirements in the building industry in California has been past precedent within the industry and not design requirements derived by the engineer.

However, the trend in structural design has been away from purely elastic design, and moment frame connections are evaluated by their post-yielding behavior. Is it realistic to assume that materials and fabrication practice developed for static applications with stresses well below yield will still provide acceptable performance under dynamic, yielding conditions? To answer this question, it may be of value to compare the design philosophies of specifications for two structures in strong seismic areas: an offshore platform subject to fatigue by wave action and a moderately-sized building. The physical and economic environments of these two structures differ radically, but they also share a number of similarities.

One major difference between these structures is the industry within which they are designed and built. Toughness has been a significant consideration in the marine environment since Liberty ships broke in two during World War II. On the other hand, the FEMA 267 Interim Guidelines (Ref. 1), which was published in 1995 after the Northridge Earthquake, was the first clear statement of the need for toughness in the California building industry. Maybe more importantly, the design of offshore tubular structures has long acknowledged the effect of materials, fabrication and inspection on post-yielding behavior, while this has yet to be addressed in a systematic way for buildings. For example, an emergency rule change adopted by ICBO (Ref. 2) to the 1994 Uniform Building Code nullified automatic approval of the standard beam-column moment connection, requiring instead that prototype tests be performed. However, the effect of materials, welding and inspection are not explicitly incorporated into the prototype procedure, even though the effect may be significant.

The effect of toughness on the performance of designed connections is explicitly acknowledged in some other industries. In shipbuilding rules (e.g., ABS MODU Rules, Ref. 3), the required toughness of the base material is determined from the service temperature, thickness and structural application category. The Bridge Welding Code (AWS D1.5, Ref. 4) was separated from the structural welding code (AWS D1.1, Ref. 5) to a large extent because requirements related to the toughness of bridge steels were not considered relevant to other structures.

FRACTURE CONTROL PLAN (FCP)

Specific toughness requirements are developed from a fracture control plan (FCP). Although there are different approaches to developing an FCP, all base toughness on the anticipated level and loading rate of the stress, the criticality of the joint location and the expected flaw size. For an offshore platform subject to earthquakes and wave-induced fatigue, this may be done explicitly; i.e., the largest flaw that could escape detection during construction in the location of interest grows by fatigue throughout the service life. The toughness required to keep the final flaw from unstable fast fracture when loaded by the extreme seismic event at the end of the design life is the value used for material selection and weld procedure qualification. This fracture mechanics analysis uses the (crack initiation) fracture toughness of the material as measured by K_{IC}, CTOD or other direct method, and such testing is often required.

A more generalized approach to an FCP is found in shipbuilding rules, where the fracture toughness required is correlated to a Charpy V-Notch (CVN) requirement for the steel and weld. This approach is somewhat less rigorous, but more economical and practical for less demanding applications. The shipbuilding requirements, for example, are based on the experience leading to Pellini's Fracture Analysis Diagram, or FAD (Ref. 6). As discussed below, this method relates the required crack arrest temperature for the steel to the service temperature and service conditions through a 'temperature shift' between the toughness testing temperature and the lowest anticipated service temperature (LAST). Although the crack arrest temperature is defined by drop weight or explosion-bulge tests, the temperature shift between the service temperature and the test temperature is correlated to CVN testing. The ship structures FCP is based on ensuring sufficient toughness to arrest cracks that might propagate during service.

In the AWS D1.5 Bridge code, steels with minimum service temperatures down to 0°F have a CVN requirement of 15 ft-lbs at +70°F. The AASHTO Fracture Control Plan on which this was based attempts to preclude unstable crack initiation by ensuring a minimum fracture toughness. In developing this FCP, they reasoned that bridges are loaded by vehicle traffic at intermediate strain rates (i.e., 10^{-3} sec^{-1}), not at the impact strain rates (i.e., 10 sec^{-1}) that occur in CVN testing. Corresponding levels of fracture toughness for these two strain rates were found to occur at a test temperature difference of 120F° for the 50 ksi yield steels tested. Tests also showed that a 15 ft-lb CVN transition temperature at -80°F would provide non-plane-strain (i.e., ductile) behavior at a minimum operating temperature of -30°F, so a shift of 70F° (39C°) above the LAST was assumed. The correlation to a slower loading rate during service permits the test temperature to be above the minimum service temperature. Although the AASHTO FCP is based on linear elastic conditions, the stresses are kept well below yield by the standard design allowables.

The same 15 ft-lbs at +70°F CVN criteria is recommended for building structures in the FEMA 267 Interim Guidelines (section 8.1.4), although the basis is not stated. For loads that result in intermediate strain rates and stresses well below yield, this criteria appears appropriate. Some researchers (e.g., E.J. Kaufmann et al, Ref. 7) suggest that loading during an earthquake is on the order of 10^{-2} sec^{-1}, which is still within an intermediate loading rate range. However, others (e.g., Kurobane et al, Ref. 8) measured higher strain rates during earthquakes (0.1sec^{-1}). These approach dynamic strain rates that are beyond the assumptions of the AASHTO FCP basis. Moreover, areas that are expected to accommodate yielding exceed the loading assumptions of the AASHTO FCP. Therefore, for locations that may experience high strain rates and/or high strains, the AASHTO 15 ft-lbs at +70°F criteria is not appropriate, and another basis for determining toughness requirements should be used.

The approach to temperature shifts used in the shipbuilding industry does not factor in strain rate differences and is not limited to linear elastic conditions. This approach is best visualized by Pellini's FAD diagram, as shown in Fig. 1 (from Ref. 6), and is characterized by temperature shifts (between the LAST and test temperature) that depend on loading and criticality. Generalized shipbuilding rules (e.g., Ref. 3) suggest that the CVN test temperature for critical steel should be about 30C° below the LAST, primary steel should be tested 10C° below the LAST, and secondary (non-structural) steel should be tested at the LAST. As shown in Fig. 1, the FAD indicates that fracture initiation will not occur for small cracks subjected to yield level dynamic loading for steels tested just above the LAST. Moreover, with a 30C° test temperature shift, even large flaws will be arrested at stresses close to the yield stress, and smaller flaws will exhibit only ductile tearing when strained beyond yield.

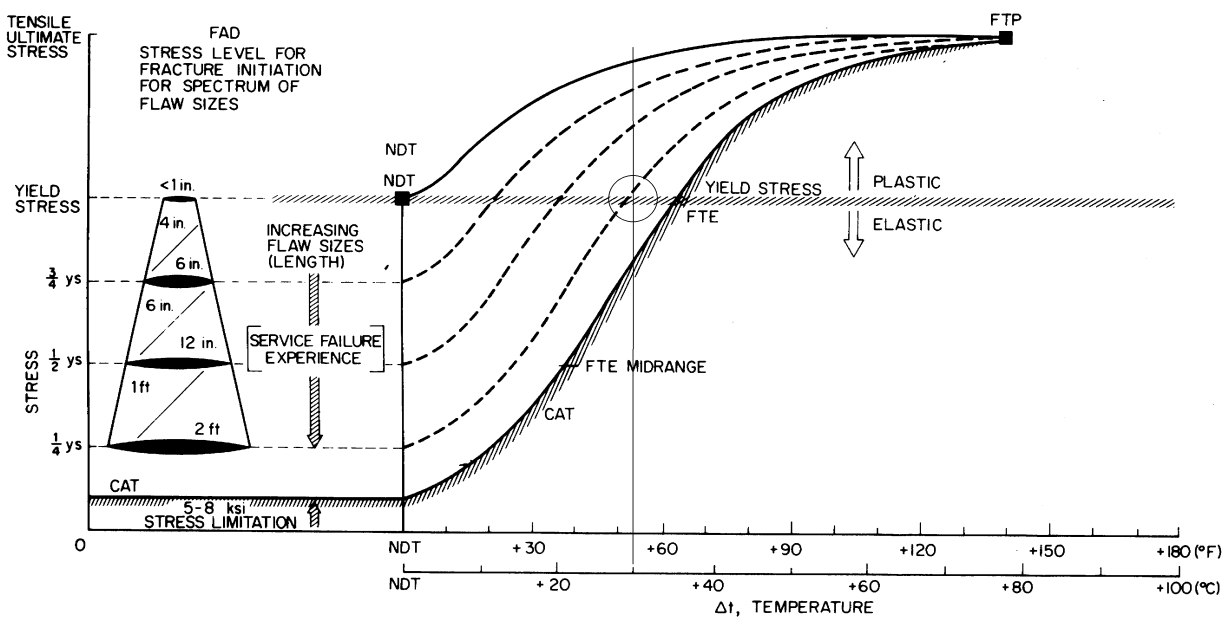

Figure 1
Fracture Analysis Diagram

Although many fracture control plans are based on CVN or other dynamic testing, there is a fundamental difference between those based on mitigating unstable crack initiation and those based on crack arrest. Ships are dynamically loaded structures that can have miles of welds, so crack arrest is important. Offshore structures have a minimum number of members and welds, so it is important to not let unstable crack initiation occur. Often however, offshore structures in onerous conditions meet both requirements to minimize the possibility of significant downtime.

SPECIFICATIONS

An FCP is primarily implemented through the construction specification and the codes and standards that the specification invokes. It is useful to look at project specifications for two very different steel structures. The first is for an offshore platform structure in an extremely onerous environment and represents an application where significant conservatism is warranted in the application of materials and workmanship. An example of such an area is offshore New Zealand, where frequent storms produce a wave environment that results in fatigue damage at rates similar to the North Sea. As an illustration, the Maui A structure waited after being partially installed offshore New Zealand for nearly a year and a half in the late 1970s before the storms subsided long enough to complete the installation. Unlike the North Sea, however, this area is seismically active, so not only does the environment tend to initiate and propagate cracks at local stress concentrations, but these cracks can be loaded by strains beyond yield in an earthquake.

The FCP for critical components in a structure such as this is based on bounding the size of the largest flaw that could escape detection at a critical area, and then propagating this flaw using Paris Law fracture mechanics methods of fatigue computation over the number of cycles estimated to be encountered by all wave states, including storms, that the structure will incur during its life. The weld metal, heat affected zone (HAZ), and parent metal are required to have sufficient fracture toughness (e.g., CTOD) at the lowest anticipated service temperature to ensure that this final flaw size will not initiate unstable crack propagation. Requirements for material selection, welding procedure development and fabrication procedures are then included in the project specification to ensure that this level of toughness is achieved in the as-fabricated structure. As discussed below, extensive controls are necessary to achieve this goal.

The second specification that will be examined is for a medium highrise building in San Francisco. Cyclic loading is generally negligible and temperatures are mild for this application, so this was historically considered to be a benign environment. Fabrication requirements reflected in earlier specifications were typically limited to generic steels (e.g., A36, A572/Gr. 50) that were welded to unmodified AWS D1.1 and inspected with ‘continuous’ visual weld inspection and ultrasonic examination of full penetration welds. Toughness was not considered important, and actual fabrication practice was often based on a lower standard than even D1.1, reflecting the lack of concern over the as-fabricated condition. A comparison of material property aspects of these two specifications is summarized in Table 1.

The New Zealand platform specification for materials, welding, fabrication, quality control and inspection was over 60 pages long. In addition to the explicit definition of material and weldment properties and fabrication tolerances, there is a decided emphasis on forcing the contractor to demonstrate compliance with all specification requirements. A contractor quality control (QC) department is mandated, and this group takes responsibility for specification compliance. Typically, the owner would have significant (QA) inspection staff to monitor this effort, but fabricated steel would not be evaluated for acceptance without written acceptance by the fabrication QC. This reduces the required size of the QA and frees them to focus on problem areas and other tasks. Although this approach is actually mandated in D1.1, it was seldom followed in practice for buildings in California, and fabricators rarely had a fully qualified ‘fabrication/erection’ inspector as required by the code.

Materials for the NZ platform are purchased with extensive toughness requirements. The steel composition and processing are ‘pre-production qualified’ at the rolling mill, as defined in API RP 2Z, to have adequate CTOD toughness in the heat affected zone (HAZ) of welds, as well as in the as-delivered plate steel. HAZ CTOD testing is very complex and expensive, resulting in a prolonged testing program to obtain the required number of valid tests. Few steel producers have actually delivered steel to this specification since these applications are rare, so this steel is expensive and has a long lead time from the mill. However, purchasing steel to API RP 2Z eliminates the need for CTOD testing by the fabrication contractor, which removes this expensive and time-consuming qualification from the contractor’s critical path, leaving them with CTOD testing of only the weld metal. Mill pre-production qualification testing has been the subject of controversy (Ref. 9), but it has proved to be beneficial for some critical projects.

For the NZ platform, welding procedure qualification is performed for virtually all structural welding, and ‘prequalified’ welding procedures as defined in AWS D1.1 and commonly used for building construction, are not generally permitted. The welding procedure qualification establishes the voltage, current and other welding parameters for production welding. In fact, the list of essential variables, i.e., those that require requalification if the specific tolerance is exceeded, is much more extensive than found in D1.1. This reflects the fact that many factors affect the fracture toughness beyond those that affect weld metal soundness and strength, which is the basis for prequalifiation in D1.1. Examples of essential variables in the NZ platform specification that would not be found in D1.1 are consumable brand name and wire size, heat input limitations, maximum interpass temperature, bevel geometry, and depth of backgouging.

QC Aspects of Weldment Toughness

Weld procedure qualification tests that include fracture toughness testing are very expensive, and HAZ CTOD toughness characterization typically takes 2 to 4 months to complete. However, the major cost in specifying a fracture toughness criterion is in the QC effort during production. The QC system must be sufficiently effective to ensure that the level of toughness tested will be reflected in the as-fabricated structure and that the structure will not have flaws that exceed the limitations defined in the FCP. The intent of a workmanship approach, as exemplified by the AWS D1.1 code and used in building construction, is to establish a basic quality level, not to ensure that all flaws greater than a specific size will be detected and repaired. The fracture control plan for critical welds in some specialized structures does indeed mandate that all flaws greater than a certain size be detected and eliminated. Moreover, this size may be based on a level of fracture toughness that requires toughness testing and other assurances that the level is met in the as-fabricated structure. As the required level of toughness increases, additional essential variables become necessary to ensure that the toughness of the as-fabricated structure reflects the results of the toughness testing. Controls on welding voltage, heat input, maximum interpass temperature and other variables add significant expense to the cost of fabrication. Examples of such differences can be seen by comparing the AWS Structural Welding Code, D1.1, to the Bridge Code, D1.5, which is based on toughness criteria. A few of these differences are summarized in Table 2.

4-TIER SYSTEM

Very little guidance is given to the structural engineer on the appropriate level of toughness required for a given project, let alone weld joint. Moreover, attempts to understand toughness requirements by looking at other codes can be confusing and misleading, as we have seen. One code requires CVN tests to be performed well above the LAST, while others permit only secondary, non-structural steels to be tested at temperatures not exceeding the LAST. And yet, the ductility and toughness of materials for structures that are designed and/or validated by their post-yielding behavior (e.g., moment frame joints) have a decided influence on the response behavior.

A four-tier system to help determine the toughness and other requirements for a specific application is proposed below. This system is based on relevant fracture control plan strategies and experience in other structural applications and is summarized in Table 3.

Tier 1

Tier 1 represents the vast majority of structural steel fabrication and simply follows D1.1. For buildings, this is modified by the local building code, but this tier does not require special steels, consumables or qualifications. As noted in the D1.1 code, the use of prequalified base materials and weld metals with prequalified welding procedures should be expected to provide matching strength and ductility that are compatible with basic, static designs. Moreover, the workmanship standard that is set out in the D1.1 code should provide a uniform level of quality that is expected to be satisfactory for these structures.

Tier 2

With the clarity of hindsight, the many committees and other groups studying the steel fractures in building moment frames after the Northridge earthquake generally agreed that improved toughness is desirable in these joints. However, the FEMA 267 Guidelines were not very clear on what properties are appropriate and did not recommend a general approach. In fact, AASHTO CVN test temperatures are suggested, even though high loading rates and post-yielding conditions are indicated for these joints elsewhere in the Guideline document. Tier 2 attempts to follow the suggestions in FEMA 267 as they are currently implemented by many in the building industry. Building structures are generally designed and built using rolled sections such as wide flange beams and heavy section columns. For the larger and more heavily loaded structures that require relatively thick flanges and webs (i.e., ASTM A6 Groups 3, 4 and 5), the number of suppliers has been decreasing until there are only a few sources for these in the U.S. A proposed replacement for ASTM A572 ("Standard Specification for Steel for Structural Shapes Used in Building Framing") was developed following the Northridge earthquake that ensures enhanced material properties, including higher toughness, limited yield strength range, increased ductility and limitations on sulfur and other chemical elements. However, the schedule and cost implications of adding these supplementary requirements has been severe in many recent projects. Considering this, some project specifications specify standard steels without CVN requirements (such as ASTM A572, Grade 50) but require the weld metal to conform to a filler metal specification that includes CVN testing, e.g., to meet 27J at –18°C (20 ft-lbs at 0°F) in the standardized test. For example, (AWS A5.20) E70T-1 FCAW wires would then be acceptable but E70T-4 wires would not. While this should result in some improvement, weld metal toughness can be very dependent on welding procedure variables, so the level of toughness actually achieved in the as-fabricated structure is unknown. Moreover, although some improvement can generally be expected when using a single type of toughness-tested consumable, recent studies (Refs. 10, 11) have confirmed that significant toughness degradation can occur when combining consumable classifications in the same weldment. Thus, because no welding procedure qualification is performed, the actual level of toughness is unknown, and this tier may not be suitable for use with a rigorous FCP. However, the following specification additions can significantly improve the general quality of a Tier 2 structure.

- For FCAW in particular, the specification should require that the manufacturer's brand name is stated on the welding procedure specification (i.e., a new WPS document is required for each consumable). The combination of voltage, current, stick-out, etc., that is within the range recommended by the manufacturer for one brand name of wire will generally not be the same as that for another, so a WPS is not meaningful unless it is limited to a single brand name of wire. This specification requirement will also make the review and use of the WPS more meaningful than if it just met the minimum requirements of D1.1.
- Similarly, separate parameters should be stated for each wire diameter.
- Many cracks found in Northridge buildings after the earthquake originated from lack of penetration welding defects under the beam web in the lower flange of beam-to-column welds. Although many of these may have been avoided with proper fit-up inspection to ensure that the root opening met the WPS tolerance, this indicates that special training is needed for this weld. A specification requirement for welder qualification that includes a mock-up of a web with access hole over the qualification butt weld will help to ensure that the welders are sufficiently well trained. An example arrangement that permits the use of standard D1.1 qualification acceptance criteria is shown in Fig. 2. Arrangements such as this have been successfully used (Ref. 12), and permit standard D1.1 acceptance criteria to be imposed. An additional bend specimen can be taken directly below the web for a more demanding evaluation of weld soundness in this area.
- Other useful specification requirements for improving the beam-to-column weld include:
    - Make the first pass of each layer against the column flange (to reduce lamellar tearing)
    - Start the lower flange weld at the center below the web and complete it on the run-off tab (to reduce the possibility of defects below the web)
    - Complete each layer on both sides of the web before proceeding (to reduce residual stresses).

- Work done 30 years ago (Ref. 13) and confirmed in the FEMA 267 Guidelines demonstrated that UT cannot reliably detect lack of fusion in the root of the lower flange-to-column weld under the web unless the backing bar is removed. Thus, a specification requirement for backgouging the lower flange root should be considered for all such significant connections.
- FEMA 267 does not recommend removal of the backing on the upper flange of the beam-to-column weld. However, if this is required, consideration should be given to modifying the mock-up in Fig. 2 to include a simulated web and column below the test plate.

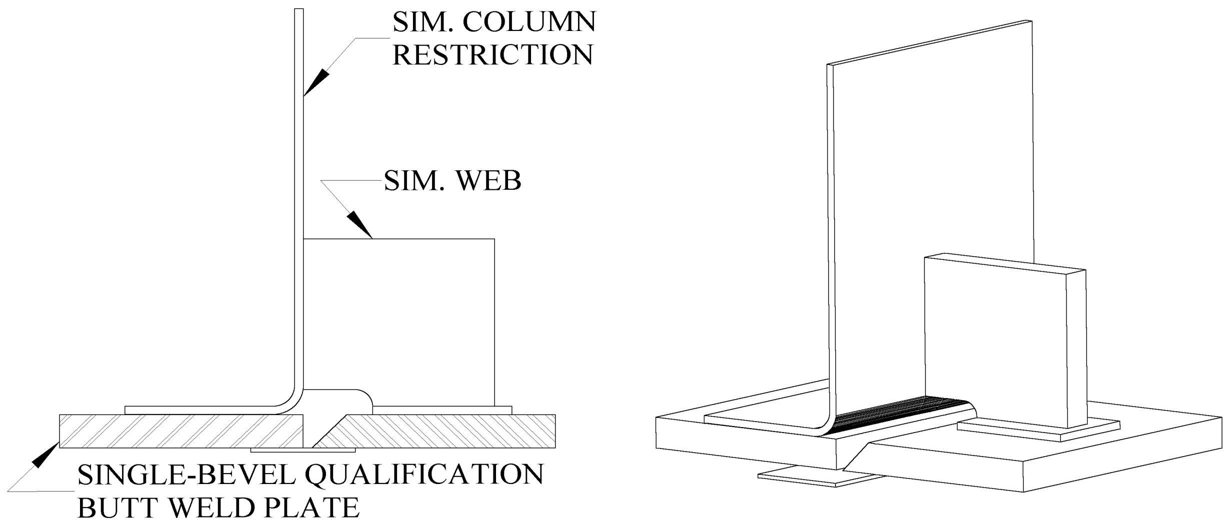

Fig. 2. Welder Qualification Mock-Up

Tier 3

Tier 3 includes the use of both CVN-tested base metals and welding procedure qualification with CVN testing. The example given in Table 3 is D1.5 because of its similarity to D1.1 but with CVN testing required. However, D1.5 is based on the AASHTO FCP as modified by DOT, CalTrans and other rules and by bridge building industry practice. Because of the assumption of intermediate loading rates and limited stresses, this precedent may not be applicable to other structures in this tier, and a modified D1.1 may be more appropriate. Although D1.1 has provisions for removing CVN specimens from weld procedure qualification tests, additional essential variables are necessary to ensure that the level of toughness determined in the PQR test will be reflected in the as-fabricated weldment. Some examples were presented in Table 2. Requirements that should be added to the project specification for FCAW, and even other processes when lower temperature toughness is required, include the following.

- Maximum interpass temperature not exceeding 30C° (50F°) more than that tested
- Minimum preheat temperature not less than 15C° (25F°) below that tested for each layer
- Maximum heat input limited by qualification weld
- No change in the electrode wire brand name
- No change in the electrode wire diameter

- No significant change (e.g., > 9mm [3/8 in.]) in the wire stickout
- No significant change in the joint detail

Because of the more stringent controls that are necessary compared to fabrication using prequalified WPSs, a higher level of quality control, and therefore quality assurance, is required. For fabrication in this tier, ignoring the D1.1 (section 6) requirement for a 'Fabrication / Erection Inspector' and relying on the owner's ('verification') inspector (which was the *de facto* system in the California building industry) will not generally provide sufficient reliability. Significant changes such as these to the D1.1 practice are pervasive and require a QC system that includes training of welders and supervisors. Thus, the practice of hiring an outside CWI as the fabrication or erection inspector will also generally not be sufficient to ensure the required level of quality. However, high toughness is typically only necessary at a limited number of locations within a given structure, and the engineer can identify these areas as the only locations where such controls are required.

Tier 4

Tier 4 is applicable to critical components of large structures that are not redundant; not easily accessible; subject to in-service fatigue, corrosion or other degradation; and possibly loaded locally beyond the yield strength. Often in highly engineered structures, some weld joints may be designed by finite element analysis or other detailed modeling method, in which case particular effort must be made to ensure that both the completed joint geometry and the as-fabricated weld properties reflect the design assumptions. Regardless of the design approach, such weldments must exhibit the level of fracture toughness required by the FCP, and close attention must be paid to material procurement and to the development and application of weld procedures to ensure this. Base metals with (API RP 2Z) pre-production qualified CTOD properties in the weld HAZ provide a cost-effective approach that minimizes the impact on the fabrication schedule because the fabricator only needs to qualify the CTOD toughness of the weld metal. However, each consumable and combination of consumables, including repair welds, should be tested.

Additional essential variables are needed beyond the Tier 3 list because the fracture toughness of a weldment is far more sensitive to procedural variables than is CVN toughness. Examples of additional essential variables that are needed to ensure that the as-fabricated fracture toughness reflects the PQR test results include the following.

- Minimum and maximum heat input welding process limitations
- Weld bead and layer sequence
- Depth of backgouging on the second side welded
- Maximum strain of formed base steel

The QC and QA effort to achieve this level of quality significantly exceeds that for Tier 3. Training of fabrication and QC personnel is essential because all of the workforce involved with Tier 4 joints must participate in the effort. This is a very expensive, schedule-demanding approach that is only appropriate for a limited number of structural components. However, it has been used extensively in areas such as North Sea offshore platform structures and nuclear power plant components because of the adverse environmental conditions and the highly evolved structural design methods used.

CONCLUSIONS

1. Although new to the structural design of buildings, base metal and weldment toughness requirements have long been in place in other structural steel applications.

2. The toughness requirements for a given structure are engineered within a fracture control plan (FCP) that combines local loading, inspection requirements and the philosophical approach to determine the required level of toughness.

3. Where the level of toughness determined in the FCP cannot be assumed for normal steel materials and fabrication practice, the project specification must define the additional requirements.

4. A good project specification states the necessary additions and modifications to the base code clearly and unambiguously.

5. An approach to developing project specification requirements, particularly for base metal and weld procedure toughness testing, is proposed that comprises a system of 4 tiers. Components for any major structure can be evaluated to determine which tier they fall into, and project specifications can then reflect the relevant requirements.

6. The additional costs associated with each tier increase significantly. However, the tier approach provides a rational basis for developing a realistic FCP for specific structural components.

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PROPERTY	S.F. BUILDING	N.Z. PLATFORM
LAST	+40°F	+32°F
Steel: Base Steel Spec	ASTM A572, Gr. 50 (with no supplementary requirements)	API SPEC 2H, Gr. 50 with Sup. Rqmts: S-7 (low nitrogen content) & S-11 (Pre-production CTOD qualification)
Steel: Through-Thickness Properties	N/A	S-4 (30% min. reduction of area in Z direction for chords)
Steel: Lamination UT	N/A	S-3 (ASTM A578, Level II UT for chords)
Steel: CVN	20J at 21°C (15 ft-lb at +70°F)	41J at -40°C (30 ft-lb at -40°F) S-3 for t>40mm (1-5/8in.): (per-plate CVN)
Steel: CTOD of HAZ	N/A	S-11 (API RP 2Z Pre-Production CTOD of 0.25mm at 10°C [0.010in. at +14°F])
WM: Weld Metal Spec	AWS A5.x	AWS A5.x
WM: Hydrogen Limits	AWS A5.x	10ml/100g production batch tested
WM: CVN	AWS A5.x Classification with 27J at -18°C (20ft-lbs at 0°F)	AWS A 5.x
WPQT: Weld Procedure Qualification Test	Prequalified or tested per D1.1	Qualified per D1.1 plus CVN from weld metal, weld root, fusion line (FL), FL+1mm, FL+5mm; repair WM, FL & HAZ.
WPQT: CVN	N/A	41J at -30°C (30 ft-lb at -22°F)
WPQT: WM CTOD	N/A	0.25mm at 0°C (0.010in. at 32°F)
WPQT: HAZ CTOD	N/A	N/A if heat input is within pre-qualified range

Table 1. Comparison of Material Property Requirements

SUBJECT	D1.1 REQUIREMENT	D1.5 REQUIREMENT
Base Metal	Wide selection in Table 3.2	CVN-tested ASTM A 709 or (Sec. 5.4.1) qualification if based on grade qualified
Filler Metal / FCAW & SAW Flux	Consumable qualified qualifies all consumables meeting same generic specification (AWS A5.x)	Qualified consumable is limited to same manufacturer's brand name (Table 5.1)
Plate Thickness	One test (1in. plate) qualifies unlimited thickness	Two tests required for unlimited thickness (Table 5.2)
Heat Input	No requirement	Qualification limits production parameters (Table 5.2)
Duration of qualification validity	Indefinite	60 Months

Table 2. Example Comparisons Between D1.1 and D1.5

TIER:	1 Basic	2 Buildings	3 Bridges, Major connections in large structures	4 Critical (inaccessible, non-redundant, fatigue, seismic)
LOADING	Dead + wind	Moderate seismic	Fatigue or Seismic	Fatigue + seismic
STRUCTURES	Most structures & buildings	Moderate high rise buildings	Bridges, large buildings & other structures	Critical, non-redundant parts of large structures
WELDING CODE	D1.1	D1.1 Modified	D1.5; D1.1 mod.	Custom
STEEL MATERIAL	D1.1 Prequalified	D1.1 Prequalified (possibly with CVNs)	CVN tested*	CTOD Qual. by Mill to API RP 2Z
WELD METAL	AWS A5.x Standard Classification	A5.x Standard Classification with CVNs	CVN tested*	CVN + CTOD tested
WPS / PQR	Prequalified WPSs	Prequalified WPSs	PQRs qualified by test with CVNs	PQRs qualified by test with CVNs + CTODs
NDE	D1.1, Sec. 6: Static	D1.1, Sec. 6: Static	D1.1, Sec. 6: Cyclic	Acceptance criteria & extent validated by FCP
QC	D1.1	D1.1	D1.1 Modified	QC Manual & Audited Program
* CVN test temperature per AASHTO/D1.5 for intermediate strain rate loading and/or low stresses, or shipbuilding practice for higher strain rates and/or high stresses.				

Table 3. Summary of 4-Tier System