

**Program to Reduce the Earthquake Hazards of
Steel Moment-Frame Structures**

**Recommended Specifications
and Quality Assurance
Guidelines for Steel Moment-
Frame Construction for
Seismic Applications**

DISCLAIMER

This document provides recommendations to supplement industry standard specifications for structural steel construction of steel moment frames intended for seismic applications. In addition it presents recommended guidelines for implementation of construction quality control and quality assurance programs for these structures. These recommendations were developed by practicing engineers, based on professional judgment and experience, and by a program of laboratory, field and analytical research. While every effort has been made to solicit comments from a broad selection of the affected parties, this is not a consensus document. It is primarily intended as a resource document for organizations with appropriate consensus processes for the development of future design standards and building code provisions. **No warranty is offered, with regard to the recommendations contained herein, either by the Federal Emergency Management Agency, the SAC Joint Venture, the individual Joint Venture partners, or their directors, members or employees. These organizations and their employees do not assume any legal liability or responsibility for the accuracy, completeness, or usefulness of any of the information, products or processes included in this publication. The reader is cautioned to review carefully the material presented herein and exercise independent judgment as to its suitability for application to specific engineering projects.** These recommended criteria have been prepared by the SAC Joint Venture with funding provided by the Federal Emergency Management Agency, under contract number EMW-95-C-4770.

Cover Art. The beam-column connection assembly shown on the cover depicts the standard detailing used in welded steel moment-frame construction prior to the 1994 Northridge earthquake. This connection detail was routinely specified by designers in the period 1970-1994 and was prescribed by the *Uniform Building Code* for seismic applications during the period 1985-1994. It is no longer considered to be an acceptable design for seismic applications. Following the Northridge earthquake, it was discovered that many of these beam-column connections had experienced brittle fractures at the joints between the beam flanges and column flanges.

Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications

SAC Joint Venture

A partnership of
Structural Engineers Association of California (SEAOC)
Applied Technology Council (ATC)
California Universities for Research in Earthquake Engineering (CUREe)

Prepared for SAC Joint Venture Partnership by
Guidelines Development Committee

Ronald O. Hamburger, Chair

John D. Hooper
Robert E. Shaw, Jr.
Lawrence D. Reaveley

Thomas Sabol
C. Mark Saunders
Raymond H. R. Tide

Project Oversight Committee

William J. Hall, Chair

Shirin Ader
John M. Barsom
Roger Ferch
Theodore V. Galambos
John Gross
James R. Harris
Richard Holguin

Nestor Iwankiw
Roy G. Johnston
Len Joseph
Duane K. Miller
John Theiss
John H. Wiggins

SAC Project Management Committee

SEAOC: William T. Holmes
ATC: Christopher Rojahn
CUREe: Robin Shepherd

Program Manager: Stephen A. Mahin
Project Director for Topical Investigations:
James O. Malley
Project Director for Product Development:
Ronald O. Hamburger

SAC Joint Venture

SEAOC: www.seaoc.org
ATC: www.atcouncil.org
CUREe: www.curee.org

June, 2000

THE SAC JOINT VENTURE

SAC is a joint venture of the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), and California Universities for Research in Earthquake Engineering (CUREe), formed specifically to address both immediate and long-term needs related to solving performance problems with welded, steel moment-frame connections discovered following the 1994 Northridge earthquake. SEAOC is a professional organization composed of more than 3,000 practicing structural engineers in California. The volunteer efforts of SEAOC's members on various technical committees have been instrumental in the development of the earthquake design provisions contained in the *Uniform Building Code* and the 1997 *National Earthquake Hazards Reduction Program (NEHRP) Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*. ATC is a nonprofit corporation founded to develop structural engineering resources and applications to mitigate the effects of natural and other hazards on the built environment. Since its inception in the early 1970s, ATC has developed the technical basis for the current model national seismic design codes for buildings; the *de-facto* national standard for postearthquake safety evaluation of buildings; nationally applicable guidelines and procedures for the identification, evaluation, and rehabilitation of seismically hazardous buildings; and other widely used procedures and data to improve structural engineering practice. CUREe is a nonprofit organization formed to promote and conduct research and educational activities related to earthquake hazard mitigation. CUREe's eight institutional members are the California Institute of Technology, Stanford University, the University of California at Berkeley, the University of California at Davis, the University of California at Irvine, the University of California at Los Angeles, the University of California at San Diego, and the University of Southern California. These university earthquake research laboratory, library, computer and faculty resources are among the most extensive in the United States. The SAC Joint Venture allows these three organizations to combine their extensive and unique resources, augmented by consultants and subcontractor universities and organizations from across the nation, into an integrated team of practitioners and researchers, uniquely qualified to solve problems related to the seismic performance of steel moment-frame structures.

ACKNOWLEDGEMENTS

Funding for Phases I and II of the SAC Steel Program to Reduce the Earthquake Hazards of Steel Moment-Frame Structures was principally provided by the Federal Emergency Management Agency, with ten percent of the Phase I program funded by the State of California, Office of Emergency Services. Substantial additional support, in the form of donated materials, services, and data has been provided by a number of individual consulting engineers, inspectors, researchers, fabricators, materials suppliers and industry groups. Special efforts have been made to maintain a liaison with the engineering profession, researchers, the steel industry, fabricators, code-writing organizations and model code groups, building officials, insurance and risk-management groups, and federal and state agencies active in earthquake hazard mitigation efforts. SAC wishes to acknowledge the support and participation of each of the above groups, organizations and individuals. In particular, we wish to acknowledge the contributions provided by the American Institute of Steel Construction, the Lincoln Electric Company, the National Institute of Standards and Technology, the National Science Foundation, and the Structural Shape Producers Council. SAC also takes this opportunity to acknowledge the efforts of the project participants – the managers, investigators, writers, and editorial and production staff – whose work has contributed to the development of these documents. Finally, SAC extends special acknowledgement to Mr. Michael Mahoney, FEMA Project Officer, and Dr. Robert Hanson, FEMA Technical Advisor, for their continued support and contribution to the success of this effort.

TABLE OF CONTENTS

LIST OF FIGURES	xi
LIST OF TABLES.....	xii
1 INTRODUCTION.....	1-1
1.1 Purpose.....	1-1
1.2 Intent	1-3
1.3 Background	1-4
1.4 Application.....	1-11
1.5 Overview	1-11

Part I: Recommended Specifications

1 GENERAL	Part I: 1-1
1.1 Scope.....	1-1
1.2 Governing Specifications.....	1-1
1.2.1 American Institute of Steel Construction.....	1-1
1.2.2 American Welding Society	1-2
1.2.3 Research Council on Structural Connections	1-2
1.2.4 Building Code	1-2
1.3 Definitions.....	1-2
1.3.1 Building Official	1-2
1.3.2 Contractor	1-3
1.3.3 Contract Documents.....	1-3
1.3.4 Engineer	1-3
1.3.5 Heavy Structural Sections.....	1-3
1.3.6 Hold Point	1-3
1.3.7 Nondestructive testing	1-3
1.3.8 Observation Point.....	1-4
1.3.9 Owner.....	1-4
1.3.10 Quality Assurance	1-4
1.3.11 Quality Assurance Agency.....	1-4
1.3.12 Quality Assurance Plan	1-4
1.3.13 Quality Control	1-4
1.3.14 Quality Control Plan	1-4
1.3.15 Special Inspection	1-5
1.3.16 Special Inspector	1-5
1.3.17 Seismic-Force-Resisting System.....	1-5
1.3.18 Seismic Weld Demand and Consequence Categories.....	1-5
1.3.19 Structural Observation	1-6
1.3.20 Structural Steel Framing	1-6
1.3.21 Written Practice	1-6
1.4 Submittals	1-6

1.4.1	Shop Drawings.....	1-6
1.4.2	Erection Drawings.....	1-7
1.4.3	Erection Plans	1-7
1.4.4	Contractors Statement of Responsibility.....	1-8
1.4.5	Certificates of Compliance	1-8
1.4.6	Manufacturer's Test Reports	1-9
1.4.6.1	Structural Steel	1-9
1.4.6.2	Fastening Material	1-9
1.4.6.3	Welding Material.....	1-10
1.4.6.4	Shear Connectors (Shear Studs)	1-10
1.4.7	Procedures.....	1-10
1.4.7.1	Fastener Installation Procedure	1-10
1.4.7.2	Welding Procedure Specifications (WPS).....	1-11
1.4.7.3	Welding Performance Qualification Records (WPQRs).....	1-12
1.4.7.4	Inspector Qualifications.....	1-13
1.4.7.5	Contractor's Quality Control Plan.....	1-13
1.4.7.6	Samples	1-13
1.5	Pre-fabrication / Pre-erection Conferences	1-13
2	PRODUCTS.....	Part I: 2-1
2.1	Structural Steel.....	2-1
2.1.1	Supplemental Requirements for Structural Steel	2-1
2.1.1.1	Wide Flange Structural Sections	2-1
2.1.1.2	Heavy Sections	2-1
2.2	Fasteners	2-2
2.3	Anchor Rods	2-2
2.4	Welding Material	2-3
2.4.1	Supplemental Requirements for Welding Materials.....	2-3
2.4.1.1	Toughness Strength and Elongation	2-3
2.4.1.2	Hydrogen Level	2-4
2.4.1.3	Packaging Requirements	2-4
2.5	Shear Connectors (Shear Studs).....	2-5
3	EXECUTION.....	Part I: 3-1
3.1	Fabrication	3-1
3.1.1	General.....	3-1
3.1.2	Bolted Construction	3-1
3.1.3	Welded Construction	3-2
3.1.4	Headed Shear Studs	3-2
3.2	Erection	3-3
3.2.1	General.....	3-3
3.2.1.1	Storage and Handling	3-3
3.2.1.2	Temporary Shoring and Bracing	3-3
3.2.1.3	Erection Procedures.....	3-3
3.2.1.4	Field Work for Other Trades	3-4

3.2.2	Bolted Construction	3-4
3.2.3	Welded Construction	3-4
3.2.4	Headed Shear Studs	3-4
3.2.5	Corrective Work.....	3-4
3.2.6	Thermal Cutting.....	3-4
3.3	Supplemental Welding Requirements.....	3-5
3.3.1	Welding Personnel Qualification.....	3-5
3.3.1.1	Supplemental Welding Personnel Testing.....	3-5
3.3.1.2	Qualification Period	3-5
3.3.2	Intermix of Filler Metals.....	3-6
3.3.3	Electrode Storage and Exposure Limits.....	3-6
3.3.4	Wind Velocity Limits.....	3-6
3.3.5	Minimum Preheat and Interpass Temperature	3-6
3.3.6	Maximum Preheat and Interpass Temperature	3-7
3.3.7	Nonfusible Backing.....	3-7
3.3.8	Peening.....	3-7
3.3.9	Controlled Cooling.....	3-8
3.3.10	Post Weld Heat Treatment	3-8
4	WELDED JOINT DETAILS	Part I: 4-1
4.1	Backing Bars	4-1
4.1.1	Heavy Section Splices Requiring Removal of Backing Bars	4-1
4.1.2	Moment Connection Joints Requiring Removal of Backing Bars	4-1
4.2	Weld Tabs	4-2
4.2.1	Use of Weld Tabs.....	4-2
4.2.2	Heavy Section Joint Weld Tab Removal and Finish	4-2
4.2.3	Moment Connection Weld Tab Removal and Finish	4-2
4.3	Weld Toes	4-3
4.4	Weld Access Holes	4-3
4.5	Web Weld Details	4-4
4.6	Doubler Plate Details	4-4
4.7	Column Continuity Plate Details	4-4
4.8	Welding Sequence for Moment Connection of Bottom Beam Flange	4-5
4.9	Improved Welded Unreinforced Flange Connection Details.....	4-6
4.9.1	Existing Web Connection Materials and Details	4-6
4.9.2	Replacement of Beam Flange Welds	4-6
4.9.3	New Welding Material.....	4-6
4.10	Haunched Connection Welding Details.....	4-7
4.10.1	Existing Web Connection Materials and Details	4-7
4.10.2	Flange Weld Details.....	4-7
4.10.3	New Welding Material.....	4-7
4.10.4	Haunch Welding Requirements	4-7
4.10.5	Continuity Plates and Stiffener Welding Requirements	4-8
4.11	Cover-Plated Moment-Connection Details.....	4-8
4.11.1	Existing Web Connection Materials and Details	4-8

	4.11.2 Flange Weld Details.....	4-8
	4.11.3 New Welding Material.....	4-8
	4.11.4 Sequence of Assembly.....	4-8
5	FABRICATION.....	Part I: 5-1
5.1	Reduced Beam Sections (RBSs).....	5-1
5.1.1	Holes and Attachments	5-1
5.1.2	RBS Cut Tolerances.....	5-1
5.1.3	Cut Surface Roughness.....	5-1
5.1.4	Gouges and Notches.....	5-1
5.1.5	Welded repair of Notches and Gouges	5-1
5.2	Heavy Sections.....	5-2
5.2.1	General.....	5-2
5.2.2	Access Hole Requirements	5-2
5.2.3	Welding.....	5-2
5.3	End Plate Moment Connections.....	5-2
5.3.1	Flange Welds	5-2
5.3.2	Web Welds.....	5-3
5.3.3	Stiffener Welds	5-3
5.3.4	Weld Access Hole.....	5-3
5.3.5	NDT Requirements	5-3
5.4	Bolted Connections.....	5-3
5.5	Repair of Discontinuities in Main Members.....	5-3
5.5.1	Tack Welds	5-4
5.5.2	Erection Aids	5-5
5.5.3	Air Carbon Arc Cutting and Thermal Cutting	5-5
5.6	K-Area Welding Limitations	5-5
5.7	Surface Finish	5-5
5.7.1	Flush Surfaces.....	5-5
5.7.2	Finish Methods and Values.....	5-5
5.8	Weld Acceptance Criteria	5-6
5.8.1	Engineer's Authority.....	5-6
5.8.2	Magnetic Particle Testing	5-6
5.8.3	Ultrasonic Testing – Flaw Detection	5-6
5.8.4	Ultrasonic Testing – Flaw Sizing.....	5-7
6	QUALITY CONTROL AND QUALITY ASSURANCE	Part I: 6-1
6.1	Specifications.....	6-1
6.1.1	Codes.....	6-1
6.1.2	Quality Assurance Agency Practices	6-1
6.1.3	Magnetic Particle Testing (MT).....	6-1
6.1.4	Ultrasonic Testing (UT).....	6-1
6.1.5	Nondestructive Testing Personnel Qualification	6-1
6.2	Submittals	6-2
6.2.1	Written Practice for Quality Assurance Agencies.....	6-2

6.2.1.1	Bolting Inspection Procedures.....	6-2
6.2.1.2	Welding Inspection Procedures.....	6-2
6.2.1.3	Shear Connector (Stud) Inspection Procedures.....	6-3
6.2.2	Written Practice for Nondestructive Testing Agencies.....	6-3
6.3	Inspector Qualifications	6-3
6.3.1	Special Inspector Qualifications	6-3
6.3.2	Welding Inspector Qualifications	6-3
6.3.3	NDT Personnel Qualifications.....	6-4
6.3.3.1	Supplementary NDT Personnel Qualification Testing.....	6-4
6.3.4	Bolting Inspector Qualifications	6-4
6.4	Contractor Tasks	6-4
6.5	Quality Assurance Agency Tasks	6-4
6.6	Welding Inspection	6-4
6.7	Nondestructive Testing of Welded Joints	6-6
6.7.1	Magnetic Particle Testing	6-6
6.7.2	Ultrasonic Testing.....	6-6
6.7.3	NDT Delay Periods.....	6-10
6.7.4	Nondestructive Testing Requirements	6-10
6.7.4.1	Heavy Sections	6-11
6.7.4.2	Lamellar Tearing	6-11
6.7.4.3	Column Splices.....	6-11
6.7.4.4	Column Webs at Continuity Plates	6-13
6.7.4.5	Column Webs at Doubler Plates	6-13
6.7.4.6	Weld Access Holes.....	6-13
6.7.4.7	Reduced Beam Section Repairs.....	6-13
6.8	Bolting Inspection	6-13
6.9	Shear Stud Welding Inspection.....	6-14
6.10	Special Inspection	6-14
APPENDIX A: WELD METAL / WELDING PROCEDURE SPECIFICATION		
	TOUGHNESS VERIFICATION PROCEDURES	Part I: A-1
APPENDIX B SUPPLEMENTAL WELDER QUALIFICATION TESTING		
	PROCEDURE.....	Part I: B-1
APPENDIX C SUPPLEMENTAL CHARPY V-NOTCH TOUGHNESS		
	TESTING FOR COMBINATIONS OF FILLER METALS	Part I: C-1
APPENDIX D SUPPLEMENTAL TESTING FOR EXTENDED EXPOSURE LIMITS		
	FOR FLUX-CORED ARC WELDING FILLER METALS.....	Part I: D-1
APPENDIX E SUPPLEMENTAL ULTRASONIC TECHNICIAN TESTING.....		
		Part I: E-1
APPENDIX F SUPPLEMENTAL MAGNETIC PARTICLE TESTING		
	PROCEDURES	Part I: F-1

Part II: Recommended Quality Assurance Guidelines

1	GENERAL	Part II: 1-1
1.1	Scope	1-1
1.2	Terms	1-2
1.2.1	Quality Control	1-2
1.2.2	Quality Assurance	1-2
1.2.3	Quality Control Plan	1-3
1.2.4	Quality Assurance Plan	1-3
1.2.5	Contract Documents.....	1-4
1.2.6	Owner.....	1-5
1.2.7	Engineer	1-5
1.2.8	Contractor	1-5
1.2.9	Building Official	1-6
1.2.10	Quality Assurance Agency.....	1-6
1.2.11	Special Inspector	1-6
1.2.12	Special Inspection	1-6
1.2.12.1	Continuous Special Inspection	1-7
1.2.12.2	Periodic Special Inspection	1-7
1.2.13	Nondestructive testing	1-7
1.2.14	Structural Observation	1-7
1.2.15	Written Practice	1-8
1.2.16	Hold Point	1-9
1.2.17	Observation Point.....	1-9
1.3	Pre-Job Meeting.....	1-9
1.4	Fabrication and Erection Drawings.....	1-10
2	CONTRACTOR QUALIFICATIONS AND QUALITY TASKS.....	Part II: 2-1
2.1	Scope	2-1
2.2	Contractor Quality Control Plan	2-1
2.3	Contractor's Statement of Responsibility	2-2
2.4	Certification Programs	2-2
2.4.1	AISC Quality Certification	2-2
2.4.2	AISC Erector Certification.....	2-4
2.4.3	Building Code Evaluation Services	2-5
2.4.4	Alternatives to Certification.....	2-5
2.4.4.1	International Standardization Organization (ISO) 9000.....	2-4
2.4.4.2	Jurisdiction Evaluation	2-5
2.4.4.3	Individual Evaluation	2-5
2.5	Contractor Obligations.....	2-5
3	QUALITY ASSURANCE AGENCY QUALIFICATIONS AND QUALITY ASSURANCE TASKS	Part II: 3-1
3.1	Scope	3-1
3.2	Evaluation of Quality Assurance Agencies.....	3-1

3.3	Quality Assurance Agencies Qualifications.....	3-1
3.3.1	Testing Laboratory Accreditation Programs	3-1
3.3.1.1	National Voluntary Laboratory Accreditation Program (NVLAP).....	3-2
3.3.1.2	American Association for Laboratory Accreditation (A2LA).....	3-2
3.3.1.3	International Conference of Building Officials Evaluation Services, Inc. (ICBO ES)	3-2
3.3.1.4	Building Officials and Code Administrators Evaluation Services, Inc. (BOCA ES).....	3-2
3.3.1.5	Southern Building Code Congress International, Public Safety Testing, and Evaluation Services, Inc.	3-2
3.3.1.6	National Evaluation Services (NES)	3-3
3.3.1.7	American Welding Society (AWS)	3-3
3.3.1.8	American Council of Independent Laboratories (ACIL).....	3-3
3.3.2	Alternatives to Certification.....	3-3
3.4	Written Practice	3-3
3.4.1	Written Practice for Quality Assurance Agencies.....	3-3
3.4.2	Written Practice for Nondestructive Testing Agencies.....	3-4
3.5	Qualification of Inspection Personnel	3-4
3.5.1	Special Inspection Qualification and Inspection	3-5
3.5.1.1	ICBO Certified Special Inspector for Structural Steel and Welding	3-5
3.5.2	Welding Inspector Qualification and Certification	3-5
3.5.3	NDT Personnel and Qualification.....	3-7
3.5.3.1	NDT Level Requirements.....	3-7
3.5.3.2	Levels of NDT Qualification.....	3-8
3.5.3.2.1	Trainee	3-8
3.5.3.2.2	NDT Level 1	3-8
3.5.3.2.3	NDT Level II	3-8
3.5.3.2.4	NDT Level III	3-8
3.5.3.3	Recertification of NDT Personnel	3-9
3.5.3.4	Suspension of Certification	3-9
3.5.3.5	Revocation of Certification	3-9
3.6	Inspection Tasks – General	3-10
3.7	Special Inspector Tasks.....	3-10
3.7.1	Quality Assurance Plan	3-10
3.7.2	Material	3-10
3.7.3	Contractor’s QC Program	3-11
3.7.4	Inspector Qualifications	3-11
3.7.5	Steel Frame	3-11
3.7.6	Welding and Bolting	3-11
3.7.7	Records	3-11
3.7.8	Reports	3-11

4	STRUCTURAL STEEL	Part II: 4-1
4.1	Specification of Material.....	4-1
4.2	Submittals	4-1
4.3	Material Control.....	4-1
4.4	Quality Control – Structural Steel.....	4-1
4.5	Extent of Steel Inspection	4-2
4.6	Structural Steel Inspection Tasks.....	4-2
5	WELDING	Part II: 5-1
5.1	Specifications.....	5-1
5.1.1	Welding Material	5-1
5.1.2	Weld Acceptance Criteria	5-1
5.2	Submittals	5-1
5.2.1	Welding Material	5-1
5.2.1.1	Supplemental CVN Toughness Verification Tests.....	5-2
5.2.2	Welding Procedure Specifications.....	5-2
5.2.3	Other Submittals	5-3
5.3	Material Control	5-3
5.3.1	Storage	5-3
5.4	Quality Control and Assurance	5-4
5.5	Quality Control Tasks	5-5
5.5.1	Welding Quality Control Programs	5-5
5.6	Extent of Welding Inspection and Nondestructive Testing	5-5
5.7	Welding Inspector Tasks.....	5-6
5.8	NDT Technician Tasks	5-7
6	BOLTING	Part II: 6-1
6.1	Specifications.....	6-1
6.1.1	Bolting Material	6-1
6.1.2	Installation Requirements	6-1
6.2	Submittals	6-1
6.2.1	Material Certifications	6-1
6.2.2	Bolting Installation Procedures.....	6-1
6.2.3	Bolting Inspection Procedures	6-1
6.3	Material Controls	6-2
6.4	Quality Control and Quality Assurance	6-2
6.4.1	Extent of Bolting Inspection	6-2
6.4.2	Bolting Quality Control Program.....	6-2
6.5	Bolt Installer Qualification.....	6-3
6.6	Bolting Inspection Tasks.....	6-3
	REFERENCES, BIBLIOGRAPHY, AND ACRONYMS.....	R-1
	SAC PROJECT PARTICIPANTS	S-1

LIST OF FIGURES

Figure 1-1	Typical Welded Moment-Resisting Connection Prior to 1994.....	1-5
Figure 1-2	Common Zone of Fracture Initiation in Beam-Column Connection	1-6
Figure 1-3	Fractures of Beam to Column Joints.....	1-6
Figure 1-4	Column Fractures.....	1-7
Figure 1-5	Vertical Fracture through Beam Shear Plate Connection	1-7

Part I: Recommended Specifications

Figure B-1	Supplemental Welder Qualification Test Assembly	B-2
------------	---	-----

Part II: Recommended Quality Assurance Guidelines

Figure 1-1	Structural Observation Checklist	1-8
Figure 2-1	Fabricator Evaluation Checklist.....	2-6
Figure 2-2	Erector Evaluation Checklist	2-11
Figure 3-1	Quality Assurance Agency Evaluation Checklist	3-12
Figure 4-1	Structural Steel Material Inspection.....	4-3
Figure 5-1	Checklist for Prefabrication Inspection for Welding	5-8
Figure 6-1	Bolting Material Inspection Record	6-5
Figure 6-2	Bolt Installer Qualification.....	6-7
Figure 6-3	Bolt Pre-Installation Assembly Tests.....	6-8
Figure 6-4	Bolted Joint Inspection Record	6-10

LIST OF TABLES

Part I: Recommended Specifications

Table 6-1	Process and Visual Welding Inspection Categories.....	6-6
Table 6-2	Process and Visual Welding Inspection Tasks	6-7
Table 6-3	Nondestructive Testing Requirements	6-12
Table A-1	WPS Toughness Verification Test - Welding and Preheat Conditions (°F)	A-1

Part II: Recommended Quality Assurance Guidelines

Table 5-1	Seismic Weld Demand Categories.....	5-10
Table 5-2	Seismic Weld Consequence Categories.....	5-11
Table 5-3	Process and Visual Inspection Categories.....	5-11
Table 5-4	Nondestructive Testing	5-12
Table 5-5	Welding Inspection Checklist	5-13

1. INTRODUCTION

1.1 Purpose

This report, *FEMA-353 – Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications* has been prepared by the SAC Joint Venture, under contract to the Federal Emergency Management Agency, to indicate those standards of workmanship for structural steel fabrication and erection deemed necessary to achieve reliably the design performance objectives contained in the set of companion publications prepared under this same contract:

- *FEMA-350 – Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, which provides recommended criteria, supplemental to *FEMA-302, 1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, for the design and construction of steel moment-frame buildings and provides alternative performance-based design criteria;
- *FEMA-351 – Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings*, which provides recommended methods to evaluate the probable performance of existing steel moment-frame buildings in future earthquakes and to retrofit these buildings for improved performance; and
- *FEMA-352 – Recommended Postearthquake Evaluation and Repair Criteria for Welded, Steel Moment-Frame Buildings*, which provides recommendations for performing postearthquake inspections to detect damage in steel moment-frame buildings following an earthquake, evaluating the damaged buildings to determine their safety in the postearthquake environment, and repairing damaged buildings.

The recommended design criteria contained in these three companion reports are based on the material and workmanship standards contained in this document, which also includes discussion of the basis for the quality control and quality assurance criteria contained in the recommended specifications.

This document has been prepared in two parts.

Part I, Recommended Specifications, provides recommended supplemental requirements, for typical project specifications for structural steel fabrication and erection, which should be included in specifications for the construction of steel moment frames designed for seismic applications. These recommendations have not been subjected to a formal consensus adoption process, nor was formal review or approval obtained from technical committees of the Structural Engineers Association of California (SEAOC). However, these recommendations have received extensive review by practicing engineers, researchers, fabricators and erectors, and the standards of construction indicated by these recommendations were presumed to exist during the development of the design criteria contained in *FEMA-350*, *FEMA-351*, and *FEMA-352*. It is anticipated that these recommendations will be submitted for consideration by the applicable standards committees of the American Institute of Steel Construction (AISC), American Society

for Testing and Materials (ASTM), American Society for Nondestructive Testing (ASNT), and American Welding Society (AWS). These organizations are expected to subject these recommendations to a consensus review process, the result of which may be the modification of the industry standard specifications to incorporate these recommendations, perhaps in modified or abbreviated form. In the interim, it is recommended that the applicable portions of these Recommended Specifications be included in construction documents where the design is based on *FEMA-350*, *FEMA-351*, or *FEMA-352*.

Part II, Recommended Quality Assurance Guidelines, has been prepared to provide design professionals, building officials, and contractors with recommended procedures for performing quality control and quality assurance functions in the construction of steel moment frames designed for seismic applications. These recommendations are non-mandatory but are deemed appropriate to achieving the construction standards presumed in the design criteria presented in *FEMA-350*, *FEMA-351* and *FEMA-352*, and may be used as a resource in developing Quality Assurance Plans as required in *FEMA-302*, *1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*.

This document is not intended to serve as a complete model for direct adoption as building code provisions, nor is it intended to supplant existing building code, design specification or material specification provisions, except as specifically noted in the text. Because building codes, design specifications, and material specifications are constantly being revised, users of this document are cautioned to review its provisions to verify their suitability and compliance with current governing standards. This document has been prepared using standards in existence or soon to be adopted at the time of publication.

Some of the material contained in this document replicates provisions of existing codes and standards. This has been done for the following reasons: to provide a complete document using a style typical of building project specifications; to highlight critical portions of existing standards that are commonly missed because the contents are contained within larger codes, specifications and standards, and may be atypical of common requirements for general steel construction; and to establish the basis for the provisions themselves.

Detailed explanations of the basis for these recommended specifications and quality assurance guidelines, hereinafter referred to as *Recommendations*, may be found in a series of State of the Art Reports prepared in parallel with these criteria. These reports include:

- *FEMA-355A – State of the Art Report on Base Metals and Fracture*. This report summarizes current knowledge of the properties of structural steels commonly employed in building construction, and the production and service factors that affect these properties.
- *FEMA-355B – State of the Art Report on Welding and Inspection*. This report summarizes current knowledge of the properties of structural welding commonly employed in building construction, the effect of various welding parameters on these properties, and the effectiveness of various inspection methodologies in characterizing the quality of welded construction.

- *FEMA-355C – State of the Art Report on Systems Performance.* This report summarizes an extensive series of analytical investigations into the demands induced in steel moment-frame buildings designed to various criteria, when subjected to a range of different ground motions. The behavior of frames constructed with fully restrained, partially restrained and fracture-vulnerable connections is explored for a series of ground motions, including motion anticipated at near-fault and soft-soil sites.
- *FEMA-355D – State of the Art Report on Connection Performance.* This report summarizes the current state of knowledge of the performance of different types of moment-resisting connections under large inelastic deformation demands. It includes information on fully restrained, partially restrained, and partial strength connections, both welded and bolted, based on laboratory and analytical investigations.
- *FEMA-355E – State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes.* This report summarizes investigations of the performance of steel moment-frame buildings in past earthquakes, including the 1995 Kobe, 1994 Northridge, 1992 Landers, 1992 Big Bear, 1989 Loma Prieta and 1971 San Fernando events.
- *FEMA-355F – State of the Art Report on Performance Prediction and Evaluation.* This report describes the results of investigations into the ability of various analytical techniques, commonly used in design, to predict the performance of steel moment-frame buildings subjected to earthquake ground motion. Also presented is the basis for performance-based evaluation procedures contained in the design criteria and guideline documents, *FEMA-350* to *FEMA-353*.

In addition to the recommended design criteria and the State of the Art Reports, a companion document has been prepared for building owners, local community officials and other non-technical audiences who need to understand this issue. *A Policy Guide to Steel Moment-Frame Construction (FEMA-354)*, addresses the social, economic, and political issues related to the earthquake performance of steel moment-frame buildings. *FEMA-354* also includes discussion of the relative costs and benefits of implementing the recommended criteria.

1.2 Intent

The recommended specifications contained in Part I are primarily intended as a resource document for organizations engaged in the development of industry standard codes and consensus standards for the construction of buildings intended to resist the effects of earthquake ground shaking. These specifications have been developed by professional engineers and researchers, based on the findings of a large multi-year program of investigation and research into the performance of steel moment-frame structures. This process included broad external review by practicing engineers, researchers, fabricators, and the producers of steel and welding consumables, including two workshops convened to obtain direct comment from these stakeholders on the proposed recommendations. These recommended specifications are intended to provide engineers, specifiers, and those involved in the development of project specifications with guidance in writing provisions appropriate for their particular projects. The provisions of this document have been established to provide a concise, detailed set of requirements for steel

moment-frame construction in seismic applications. Currently, there are numerous and widely diverse requirements being specified by practicing engineers and building authorities for steel moment-frame construction in seismic applications. These *Recommendations* provide a basis for standardization of those requirements within the industry, consistent with design recommendations developed by this project.

The recommended quality assurance guidelines contained in Part II are intended to provide engineers, building officials, inspection agencies and contractors with an understanding of the important features of a quality control and quality assurance program for steel moment-frame construction for seismic applications. It provides information and recommended guidelines relative to the appropriate qualifications and roles of the various parties engaged in the construction of these structures. In addition, it provides a recommended system of classification for welded joints in seismic applications that may be used to select and specify appropriate quality control and quality assurance measures for these joints. This system has been used to develop the recommended quality assurance categories referenced in the design criteria documents produced by this project, including *FEMA-350*, *FEMA-351* and *FEMA-352*. This same classification system is used in Part I of these *Recommendations* to determine the applicable recommended quality control and quality assurance specification provisions.

1.3 Background

For many years, the basic intent of the building code seismic provisions has been to provide buildings with an ability to withstand intense ground shaking without collapse, but potentially with some significant structural damage. In order to accomplish this, one of the basic principles inherent in modern code provisions is to encourage the use of building configurations, structural systems, materials and details that are capable of ductile behavior. A structure is said to behave in a ductile manner if it is capable of withstanding large inelastic deformations without significant degradation in strength, and without the development of instability and collapse. The design forces specified by building codes for particular structural systems are related to the amount of ductility the system is deemed to possess. Generally, structural systems with more ductility are designed for lower forces than less ductile systems, as ductile systems are deemed capable of resisting demands that are significantly greater than their elastic strength limit. Starting in the 1960s, engineers began to regard welded steel moment-frame buildings as being among the most ductile systems contained in the building code. Many engineers believed that steel moment-frame buildings were essentially invulnerable to earthquake-induced structural damage and thought that should such damage occur, it would be limited to ductile yielding of members and connections. Earthquake-induced collapse was not believed possible. Partly as a result of this belief, many large industrial, commercial and institutional structures employing steel moment-frame systems were constructed, particularly in the western United States.

The Northridge earthquake of January 17, 1994 challenged this paradigm. Following that earthquake, a number of steel moment-frame buildings were found to have experienced brittle fractures of beam-to-column connections. The damaged buildings had heights ranging from one story to 26 stories, and a range of ages spanning from buildings as old as 30 years to structures just being erected at the time of the earthquake. The damaged buildings were spread over a large

geographical area, including sites that experienced only moderate levels of ground shaking. Although relatively few buildings were located on sites that experienced the strongest ground shaking, damage to buildings on these sites was extensive. Discovery of these unanticipated brittle fractures of framing connections, often with little associated architectural damage to the buildings, was alarming to engineers and the building industry. The discovery also caused some concern that similar, but undiscovered, damage may have occurred in other buildings affected by past earthquakes. Later investigations confirmed such damage in a limited number of buildings affected by the 1992 Landers, 1992 Big Bear and 1989 Loma Prieta earthquakes.

In general, steel moment-frame buildings damaged by the 1994 Northridge earthquake met the basic intent of the building codes. That is, they experienced limited structural damage, but did not collapse. However, the structures did not behave as anticipated and significant economic losses occurred as a result of the connection damage, in some cases, in buildings that had experienced ground shaking less severe than the design level. These losses included direct costs associated with the investigation and repair of this damage as well as indirect losses relating to the temporary, and in a few cases, long-term, loss of use of space within damaged buildings.

Steel moment-frame buildings are anticipated to develop their ductility through the development of yielding in beam-column assemblies at the beam-column connections. This yielding may take the form of plastic hinging in the beams (or, less desirably, in the columns), plastic shear deformation in the column panel zones, or through a combination of these mechanisms. It was believed that the typical connection employed in steel moment-frame construction, shown in Figure 1-1, was capable of developing large plastic rotations, on the order of 0.02 radians or larger, without significant strength degradation.

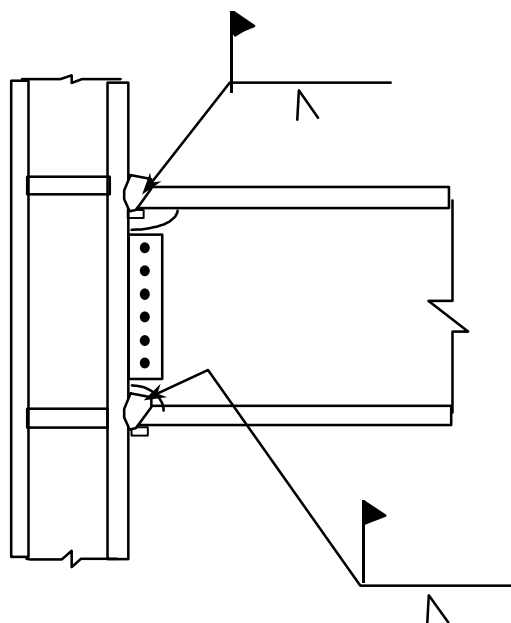


Figure 1-1 Typical Welded Moment-Resisting Connection Prior to 1994

Observation of damage sustained by buildings in the 1994 Northridge earthquake indicated that, contrary to the intended behavior, in many cases, brittle fractures initiated within the connections at very low levels of plastic demand, and in some cases, while the structures remained essentially elastic. Typically, but not always, fractures initiated at, or near, the complete joint penetration groove weld between the beam bottom flange and column flange (Figure 1-2). Once initiated, these fractures progressed along a number of different paths, depending on the individual joint conditions.

In some cases, the fractures progressed completely through the thickness of the weld, and when fire protective finishes were removed, the fractures were evident as a crack through exposed faces of the weld, or the metal just behind the weld (Figure 1-3a). Other fracture patterns also developed. In some cases, the fracture developed into a crack of the column flange material behind the complete joint penetration groove weld (Figure 1-3b). In these cases, a portion of the column flange remained bonded to the beam flange, but pulled free from the

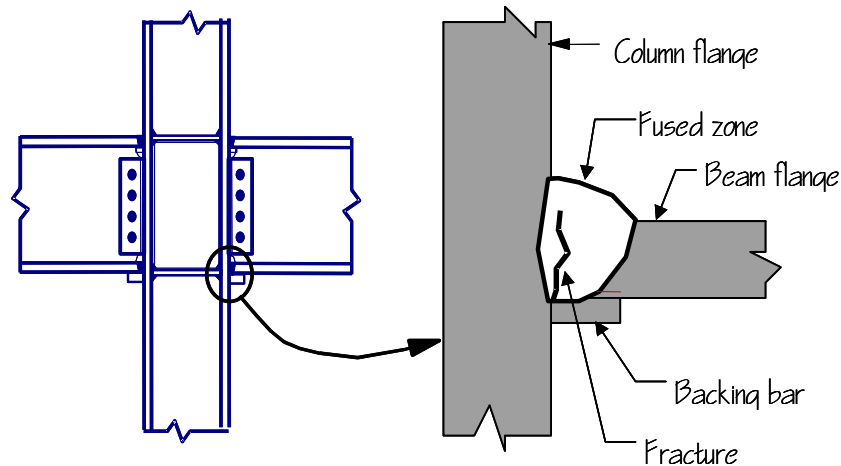


Figure 1-2 Common Zone of Fracture Initiation in Beam-Column Connection



a. Fracture at Fused Zone



b. Column Flange "Divot" Fracture

Figure 1-3 Fractures of Beam to Column Joints

remainder of the column. This fracture pattern has sometimes been termed a “divot” or “nugget” failure.

A number of fractures progressed completely through the column flange, along a near-horizontal plane that aligns approximately with the beam lower flange (Figure 1-4a). In some cases, these fractures extended into the column web and progressed across the panel zone (Figure 1-4b). Investigators have reported some instances where columns fractured entirely across the section.



a. Fractures through Column Flange



b. Fracture Progresses into Column Web

Figure 1-4 Column Fractures

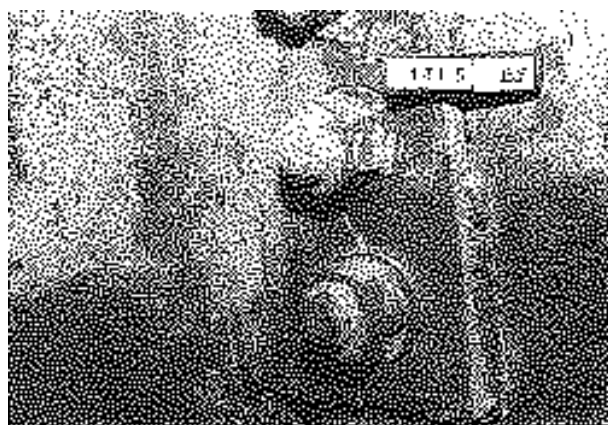


Figure 1-5 Vertical Fracture through Beam Shear Plate Connection

Once such fractures have occurred, the beam-column connection has experienced a significant loss of flexural rigidity and strength to resist those loads that tend to open the crack. Residual flexural strength and rigidity must be developed through a couple consisting of forces transmitted through the remaining top flange connection and the web bolts. However, in providing this residual strength and stiffness, the bolted web connections can themselves be subject to failures. These include fracturing of the welds of the shear plate to the column, fracturing of supplemental welds to the beam web or fracturing through the weak section of shear plate aligning with the bolt holes (Figure 1-5).

Despite the obvious local strength impairment resulting from these fractures, many damaged buildings did not display overt signs of structural damage, such as permanent drifts or damage to architectural elements, making reliable postearthquake damage evaluations difficult. In order to determine reliably if a building has sustained connection damage it is necessary to remove architectural finishes and fireproofing, and perform detailed inspections of the connections. Even if no damage is found, this is a costly process. Repair of damaged connections is even more costly. At least one steel moment-frame building sustained so much damage that it was deemed more practical to demolish the building than to repair it.

Initially, the steel construction industry took the lead in investigating the causes of this unanticipated damage and in developing design recommendations. The American Institute of Steel Construction (AISC) convened a special task committee in March, 1994 to collect and disseminate available information on the extent of the problem (AISC, 1994a). In addition, together with a private party engaged in the construction of a major steel building at the time of the earthquake, the American Institute of Steel Construction (AISC) participated in sponsoring a limited series of tests of alternative connection details at the University of Texas at Austin (AISC, 1994b). The American Welding Society (AWS) also convened a special task group to investigate the extent that the damage related to welding practice and to determine if changes to the welding code were appropriate (AWS, 1995).

In September, 1994, the SAC Joint Venture, AISC, the American Iron and Steel Institute and National Institute of Standards and Technology jointly convened an international workshop (SAC, 1994) in Los Angeles to coordinate the efforts of the various participants and to lay the foundation for systematic investigation and resolution of the problem. Following this workshop, FEMA entered into a cooperative agreement with the SAC Joint Venture to perform problem-focused studies of the seismic performance of steel moment-frame buildings and to develop recommendations for professional practice (Phase I of SAC Steel Project). Specifically, these recommendations were intended to address the following: the inspection of earthquake-affected buildings to determine if they had sustained significant damage; the repair of damaged buildings; the upgrade of existing buildings to improve their probable future performance; and the design of new structures to provide reliable seismic performance.

During the first half of 1995, an intensive program of research was conducted to explore more definitively the pertinent issues. This research included literature surveys, data collection on affected structures, statistical evaluation of the collected data, analytical studies of damaged and undamaged buildings, and laboratory testing of a series of full-scale beam-column assemblies representing typical pre-Northridge design and construction practice as well as various repair, upgrade and alternative design details. The findings of these tasks formed the basis for the development of *FEMA-267 – Interim Guidelines: Evaluation, Repair, Modification, and Design of Welded Steel Moment Frame Structures*, which was published in August, 1995. *FEMA-267* provided the first definitive, albeit interim, recommendations for practice, following the discovery of connection damage in the 1994 Northridge earthquake.

In September 1995 the SAC Joint Venture entered into a contractual agreement with FEMA to conduct Phase II of the SAC Steel Project. Under Phase II, SAC continued its extensive

problem-focused study of the performance of moment resisting steel frames and connections of various configurations, with the ultimate goal of develop seismic design criteria for steel construction. This work has included: extensive analyses of buildings; detailed finite element and fracture mechanics investigations of various connections to identify the effects of connection configuration, material strength, and toughness and weld joint quality on connection behavior; as well as more than 120 full-scale tests of connection assemblies. As a result of these studies, and independent research conducted by others, it is now known that the typical moment-resisting connection detail employed in steel moment-frame construction prior to the 1994 Northridge earthquake, and depicted in Figure 1-1, had a number of features that rendered it inherently susceptible to brittle fracture. These included the following:

- The most severe stresses in the connection assembly occur where the beam joins to the column. Unfortunately, this is also the weakest location in the assembly. At this location, bending moments and shear forces in the beam must be transferred to the column through the combined action of the welded joints between the beam flanges and column flanges and the shear tab. The combined section properties of these elements, for example the cross sectional area and section modulus, are typically less than those of the connected beam. As a result, stresses are locally intensified at this location.
- The joint between the bottom beam flange and the column flange is typically made as a downhand field weld, often by a welder sitting on top of the beam top flange, in a so-called “wildcat” position. To make the weld from this position each pass must be interrupted at the beam web, with either a start or stop of the weld at this location. This welding technique often results in poor quality welding at this critical location, with slag inclusions, lack of fusion and other defects. These defects can serve as crack initiators, when the connection is subjected to severe stress and strain demands.
- The basic configuration of the connection makes it difficult to detect hidden defects at the root of the welded beam-flange-to-column-flange joints. The backing bar, which was typically left in place following weld completion, restricts visual observation of the weld root. Therefore, the primary method of detecting defects in these joints is through the use of ultrasonic testing. However, the geometry of the connection also makes it very difficult for ultrasonic testing to detect flaws reliably at the bottom beam flange weld root, particularly at the center of the joint, at the beam web. As a result, many of these welded joints have undetected significant defects that can serve as crack initiators.
- Although typical design models for this connection assume that nearly all beam flexural stresses are transmitted by the flanges and all beam shear forces by the web, in reality, due to boundary conditions imposed by column deformations, the beam flanges at the connection carry a significant amount of the beam shear. This results in significant flexural stresses on the beam flange at the face of the column, and also induces large secondary stresses in the welded joint. Some of the earliest investigations of these stress concentration effects in the welded joint were conducted by Richard, et al. (1995). The stress concentrations resulting from this effect resulted in severe strength demands at the root of the complete joint penetration welds between the beam flanges and column flanges, a region that often includes significant discontinuities and slag inclusions, which are ready crack initiators.

- In order that the welding of the beam flanges to the column flanges be continuous across the thickness of the beam web, this detail incorporates a series of weld access holes in the beam web, at the beam flanges. Depending on their geometry, severe strain concentrations can occur in the beam flange at the toe of these weld access holes. These strain concentrations can result in low-cycle fatigue and the initiation of ductile tearing of the beam flanges after only a few cycles of moderate plastic deformation. Under large plastic flexural demands, these ductile tears can quickly become unstable and propagate across the beam flange.
- Steel material at the center of the beam-flange-to-column-flange joint is restrained from movement, particularly in connections of heavy sections with thick column flanges. This condition of restraint inhibits the development of yielding at this location, resulting in locally high stresses on the welded joint, which exacerbates the tendency to initiate fractures at defects in the welded joints.
- Design practice in the period 1985-1994 encouraged design of these connections with relatively weak panel zones. In connections with excessively weak panel zones, inelastic behavior of the assembly is dominated by shear deformation of the panel zone. This panel zone shear deformation results in a local kinking of the column flanges adjacent to the beam-flange-to-column-flange joint, and further increases the stress and strain demands in this sensitive region.

In addition to the above, additional conditions contributed significantly to the vulnerability of connections constructed prior to 1994.

- In the mid-1960s, the construction industry moved to the use of the semi-automatic, self-shielded, flux-cored arc welding process (FCAW-S) for making the joints of these connections. The welding consumables that building erectors most commonly used inherently produced welds with very low toughness. The toughness of this material could be further compromised by excessive deposition rates, which unfortunately were commonly employed by welders. As a result, brittle fractures could initiate in welds with large defects, at stresses approximating the yield strength of the beam steel, precluding the development of ductile behavior.
- Early steel moment frames tended to be highly redundant and nearly every beam-column joint was constructed to behave as part of the lateral-force-resisting system. As a result, member sizes in these early frames were small and much of the early acceptance testing of this typical detail was conducted with specimens constructed of small framing members. As the cost of construction labor increased, the industry found that it was more economical to construct steel moment-frame buildings by moment-connecting a relatively small percentage of the beams and columns and by using larger members for these few moment-connected elements. The amount of strain demand placed on the connection elements of a steel moment frame is related to the span-to-depth ratio of the member. Therefore, as member sizes increased, strain demands on the welded connections also increased, making the connections more susceptible to brittle behavior.
- In the 1960s and 1970s, when much of the initial research on steel moment-frame construction was performed, beams were commonly fabricated using A36 material. In the

1980s, many steel mills adopted more modern production processes, including the use of scrap-based production. Steels produced by these more modern processes tended to include micro-alloying elements that increased the strength of the materials so that despite the common specification of A36 material for beams, many beams actually had yield strengths that approximated or exceeded that required for grade 50 material. As a result of this increase in base metal yield strength, the weld metal in the beam-flange-to-column-flange joints became under-matched, potentially contributing to its vulnerability.

At this time, it is clear that in order to obtain reliable ductile behavior of steel moment-frame construction a number of changes to past practices in design, materials, fabrication, erection and quality assurance are necessary. The recommended criteria contained in this document, and the companion publications, are based on an extensive program of research into materials, welding technology, inspection methods, frame system behavior, and laboratory and analytical investigations of different connection details. The guidelines presented herein are believed to be capable of addressing the vulnerabilities identified above and providing for frames capable of more reliable performance in response to earthquake ground shaking.

1.4 Application

These *Recommendations* supersede the quality assurance guidelines contained in *FEMA-267, Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures*, and the *Interim Guidelines Advisories, FEMA-267A and FEMA-267B*. It is intended to be used in coordination with and in supplement to the locally applicable building code and those national standards referenced by the building code. Building codes are living documents and are revised on a periodic basis. These *Recommendations* have been prepared based on the provisions contained in the 1997 *FEMA-302 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, the 1997 *AISC Seismic Specification* (AISC, 1997) and the 1998 *AWS D1.1 Structural Welding Code - Steel*, as these documents form the basis for the 2000 edition of the *International Building Code*. Some users may wish to apply the recommendations contained herein to specific engineering projects, prior to the adoption of these recommendations by future codes and standards. Such users are cautioned to consider carefully any differences between the aforementioned documents and those actually enforced by the building department having jurisdiction for a specific project and to adjust these recommendations accordingly. These users are also warned that these recommendations have not undergone a consensus adoption process. Users should thoroughly acquaint themselves with the technical data upon which these recommendations are based and exercise their own independent engineering judgment prior to implementing these recommendations in practice.

1.5 Overview

The following is an overview of the general contents of chapters contained in these *Recommendations*, and their intended use. Many of the specification provisions contained in Part I are already included in industry standard specification and building code requirements.

When this document is used as a basis for the development of project specifications, users should include those paragraphs specifically applicable to the individual project. This includes provisions of a general nature that may be necessary for the overall project, as well as specific provisions applicable to the connection systems and details being utilized. The recommended specifications contained in Part I are not intended as a stand-alone specification. They are intended to be used as a supplement to, and in coordination with, a complete specification for structural steel construction.

The user should also review the incorporated provisions for compliance with local building code requirements, particularly in those jurisdictions that may not have adopted the latest model specifications, and those with specific provisions not found in model specifications. In some cases, a variance may be needed to utilize these provisions where revisions to the existing codes and standards are recommended.

Recommendations for new requirements applicable to the construction of seismic-force-resisting systems are indicated in these *Recommendations* by underlined text.

Part I – Recommended Specifications

- **Chapter 1: General.** This chapter provides definitions for use throughout Part I. It also contains provisions pertaining to submittal of information regarding material, material certifications, procedures, personnel records, quality control plan, and samples. The use of a pre-fabrication, pre-erection conference is also established.
- **Chapter 2: Products.** This chapter lists the applicable material specifications, both generic and specific, for structural steel, welding material, bolting material, and shear connectors.
- **Chapter 3: Execution.** This chapter provides generic and specific recommendations for the fabrication and erection of the structural steel frames, bolting, and welding. Included are several specific recommendations regarding welding operations that are not currently in the welding codes.
- **Chapter 4: Welded Joint Details.** This chapter provides detailing and welding recommendations for specific types of welded joints, including backing bars, weld tabs, reinforcing fillet welds, weld access holes, web connections, doubler plates, continuity plates, cover-plated connections, welded overlay connections, and haunched connections.
- **Chapter 5: Fabrication Details.** This chapter provides detailing and quality recommendations for steel fabrication, welding, cutting, bolting, and repairs. Quality recommendations for beam-flange-to-column-flange moment-connection welds are provided in this chapter.
- **Chapter 6: Quality Control and Quality Assurance.** This chapter lists governing specifications and practices, the requirements and recommendations for a Written Practice for quality assurance and nondestructive testing, inspector qualifications, nondestructive testing technician qualifications, and a detailed list of quality control and quality assurance tasks for welding, bolting, and shear connector inspection. Specific recommendations for nondestructive testing for various joints are included.

- **Appendices.** Several appendices are provided with details for various recommended test procedures for welding material, welding personnel qualification, and nondestructive testing technician qualification. Recommended provisions for magnetic particle testing procedures are also provided.

Part II – Quality Assurance Guidelines

- **Chapter 1: General.** This chapter provides discussion of various terms relating to the process of ensuring that workmanship and materials conform to the applicable standards. In addition it describes the various phases of the quality process and the recommended role of various participants in these phases.
- **Chapter 2: Contractor Qualifications and Quality Tasks.** This chapter describes methods for determining whether contractors have adequate qualifications to perform the work, and also provides recommended contractor responsibilities in the quality process.
- **Chapter 3: Quality Assurance Agency Qualifications and Quality Assurance Tasks.** This chapter describes methods for determining whether inspection agencies and testing laboratories that perform quality control and assurance tasks, and their personnel engaged in these tasks, have adequate qualifications for this work. It also includes recommendations for inspection agency work scope as part of the quality process.
- **Chapter 4: Structural Steel.** This chapter provides recommendations for procedures to ensure that structural steel materials meet the applicable standards. Checklists for assisting in the verification of steel quality are included.
- **Chapter 5: Welding.** This chapter presents recommended methods for ensuring that welded joints meet the applicable materials and workmanship standards. Included in this chapter is a description of the basis for determining the quality assurance category for a welded joint, used in Part I of these *Recommendations* as an index to recommended quality assurance measures.
- **Chapter 6: Bolting.** This chapter presents recommended procedures for determining whether bolted joints meet the applicable materials and workmanship requirements.
- **References, Bibliography, and Acronyms.**

Commentary: The Recommended Specifications and Quality Assurance Guidelines contained in these Recommendations are applicable to the construction of moment-resisting frames used as the seismic-force-resisting elements of buildings and other structures, designed in accordance with the recommendations of FEMA-350, FEMA-351 and FEMA-352. As used in these Recommendations, the terms “steel structure” or “steel framing” apply only to the structural steel elements of the seismic-force-resisting system as defined in FEMA-302. The quality assurance guidelines and quality control guidelines contained in these Recommendations could be extended to the construction of other types of seismic-force-resisting systems and even to elements of structures that are not intended to be part of the seismic-force-resisting system. However,

extension of these recommendations to these other construction types and elements could result in significant construction cost premiums. These Recommendations are deemed appropriate to application to the construction, upgrade and repair of Special Moment Frames and Ordinary Moment Frames used in seismic-force-resisting systems due to the severe service conditions anticipated for these frames and the proven sensitivity of their performance to construction quality. Such determination has not been made for other elements or systems. Further discussion may be found in FEMA-354, A Policy Guide to Steel Moment-Frame Construction.

PART I

RECOMMENDED SPECIFICATIONS

Note: Recommendations for new requirements applicable to the construction of seismic-force-resisting systems are indicated in these *Recommendations* by underlined text.

Part I: RECOMMENDED SPECIFICATIONS

1. GENERAL

1.1 Scope

The provisions of Part I apply to fabrication, erection and inspection of steel moment frames as follows:

- new Ordinary Moment Frames (OMF) or Special Moment Frames (SMF) designed in accordance with the recommended criteria of *FEMA-350*;
- upgrade of existing steel moment frames designed in accordance with the recommended criteria of *FEMA-351*; or
- repair of damaged steel moment frames designed in accordance with the recommended criteria of *FEMA-352*.

Fabrication, erection and inspection shall be as shown on the approved drawings, including schedules, notes and details, and as further provided in the specifications of Part I.

Commentary: Throughout Part I, references are made to sections in Part II of these Recommendations, Chapters 1 to 6, where pertinent supplemental information may be found. These references are provided for information only and the supplemental information itself should not typically be included within the body of specifications, nor should the references to this information.

1.2 Governing Specifications

Structural steel framing shall comply with the requirements of the governing specifications listed in the subsections that follow.

Commentary: In some cases, several alternative specifications may apply. In these cases, the alternatives are preceded by a sequence letter, e.g., (a), (b), (c). In such cases, the specifier should select the entry appropriate to the individual project. Specifications without a sequence letter should always be included in project specifications.

1.2.1 American Institute of Steel Construction

- (a) *Load and Resistance Factor Design Specification for Structural Steel Buildings*, December 1, 1993, with *Supplement No. 1*, January 30, 1998; or
- (b) *Metric Load and Resistance Factor Design Specification for Structural Steel Buildings*, December 1, 1994, with *Supplement No. 1*, January 30, 1998; or
- (c) *Specification for Structural Steel Buildings (Allowable Stress Design and Plastic Design)*, June 1, 1989.

- *Seismic Provisions for Structural Steel Buildings*, April 15, 1997, with *Supplement No. 1*, February 15, 1999;
- *Code of Standard Practice for Steel Buildings and Bridges*, March 7, 2000.

1.2.2 American Welding Society

- (a) AWS D1.1-98 *Structural Welding Code* – Steel, 1998; or
- (b) AWS D1.1:2000 *Structural Welding Code* – Steel, 2000.

1.2.3 Research Council on Structural Connections

- (a) *LRFD Specification for Structural Joints Using ASTM A325 and A490 Bolts*, June 3, 1994; or
- (b) *Allowable Stress Design Specification for Structural Joints Using ASTM A325 or A490 Bolts*, November 13, 1985, reaffirmed June 3, 1994.

1.2.4 Building Code

- (a) *ICC International Building Code* – 2000; or
- (b) *ICBO Uniform Building Code* – 1997; or
- (c) *BOCA National Building Code* – 1999; or
- (d) *SBCCI Standard Building Code* – 1997; or
- (e) the locally applicable building code.

Commentary: In addition to referencing the above building codes, these Recommendations make reference to, or are referenced by, the following Federal Emergency Management Agency publications. Reference to the following publications should not normally be included in project specifications.

FEMA-350 – Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings

FEMA-351 – Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings

FEMA-352 – Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings

1.3 Definitions

1.3.1 Building Official

The term Building Official is applied to the regulatory agency having jurisdiction of the building. (See Part II, Section 1.2.9)

1.3.2 Contractor

The Contractor is the firm performing the fabrication or erection of the structural steel framework. The Contractor may be the steel fabricator responsible for the shop fabrication of the structural steel, and/or the steel erector, responsible for all field erection and field welding. (See Part II, Section 1.2.8)

1.3.3 Contract Documents

The Contract Documents include the design drawings, including schedules, notes and details, the project specifications, and the Quality Assurance Plan. (See Part II, Section 1.2.5)

1.3.4 Engineer

The Engineer is the registered design professional in responsible charge of the design of the structural steel framing system. (See Part II, Section 1.2.7)

Commentary: Section 202 of the International Building Code (IBC) defines a Registered Design Professional as being the person responsible for design. Throughout this specification, the term Engineer is used for this person.

1.3.5 Heavy Structural Sections

The following rolled sections and built-up sections shall be considered as Heavy Sections, subject to special notch toughness, fabrication, welding and inspection requirements as specified herein.

- ASTM A6 Group 3 shapes with flanges thicker than 1-1/2 in.
- ASTM A6 Group 4 and Group 5 shapes
- Welded built-up members with web or flange plates exceeding 2 inches in thickness.

Commentary: The requirements of this section as applied to Group 3 shapes with flanges thicker than 1-1/2 in. is contained in the AISC Seismic Provisions (AISC, 1997, 1999). The requirement for Group 4 and 5 shapes is contained in AISC LRFD and the AISC Seismic Provisions.

1.3.6 Hold Point

A stage in the work beyond which the work shall not proceed until the designated inspector has completed specified inspection tasks. (See Part II, Section 1.2.16 and Table 5-5)

1.3.7 Nondestructive Testing

Nondestructive testing (NDT) includes magnetic particle testing (MT), penetrant testing (PT), radiographic testing (RT), and ultrasonic testing (UT). The terms nondestructive examination (NDE) and nondestructive testing (NDT) are synonymous. (See Part II, Section 1.2.13)

1.3.8 Observation Point

A stage in the work beyond which the work may proceed after the workman has completed his or her own inspection, with special inspection provided by the designated special inspector through observation of the work operations on a routine, occasional, random basis. (See Part II, Section 1.2.17 and Table 5-5)

1.3.9 Owner

The Owner is the entity responsible for providing Quality Assurance. (See Part II, Section 1.2.6)

1.3.10 Quality Assurance

Quality Assurance (QA) consists of those inspection and testing services to be performed by an agency or firm other than the Contractor to ensure that the material and workmanship of structural steel construction meet the project specifications, applicable standards, and the Quality Assurance Plan. (See Part II, Section 1.2.2)

1.3.11 Quality Assurance Agency

The Quality Assurance (QA) Agency is the independent testing organization selected by the Owner, or Owner's designee, to perform the Quality Assurance inspection and testing required by the Quality Assurance Plan, specifications and contract documents. (See Part II, Section 1.2.10)

1.3.12 Quality Assurance Plan

The Quality Assurance Plan ("Plan") is the written requirements containing the set of procedures that are to be followed to confirm compliance with these requirements. The Quality Assurance Plan shall be part of the Contract Documents. (See Part II, Section 1.2.4)

Commentary: QA Plans are required by IBC Section 1705.1

1.3.13 Quality Control

Quality Control includes those functions to be performed by the Contractor to ensure that the material and workmanship of structural steel construction meet the project specifications, applicable standards, and the Quality Assurance Plan. (See Part II, Section 1.2.1)

1.3.14 Quality Control Plan

The Quality Control (QC) Plan is the written statement, prepared by the Contractor, of all measures that will be taken to ensure that construction conforms to the applicable standards, project specifications, and the Quality Assurance Plan. (See Part II, Sections 1.2.3, and 2.2)

Commentary: QC Plans are required by IBC Section 1704.2.1

1.3.15 Special Inspection

Special Inspection is a Quality Assurance activity comprising inspection of the installation, fabrication, erection or placement of components and connections requiring special expertise to ensure adequacy. (See Part II, Section 1.2.12)

Commentary: Special Inspection is defined in IBC Section 1702. Note that Special Inspection is one of several types of inspection that should be performed as part of structural steel construction. Refer to Part II of these Recommendations for supplemental information on the various types of inspection that should be performed.

1.3.16 Special Inspector

The Special Inspector is the individual or firm employed by the Owner, to provide the special inspection required. In most cases, the Special Inspector and the Quality Assurance Agency are the same entity. (See Part II, Section 1.2.11)

1.3.17 Seismic-Force-Resisting System

The Seismic-Force-Resisting System (SFRS) is defined as the assembly of structural elements in the building that resists seismic forces. Included in the SFRS are the columns, beams, girders and braces, and the connections between these elements, specifically designed to resist seismic forces, as designated on the contract documents. The SFRS does not include members that provide out-of-plane bracing to components of the SFRS, nor does it include other structural members designed to resist only gravity loads.

Commentary: The construction documents should clearly identify those members and connections that are part of the Seismic-Force-Resisting System (SFRS) so that the extent of applicability of special fabrication, erection and inspection requirements may be fully understood by all parties. Failure to designate the extent of the SFRS could result in substantial construction cost premiums.

1.3.18 Seismic Weld Demand and Seismic Weld Consequence Categories

Structural welds that are a part of the Seismic-Force-Resisting System are identified on the design drawings as Seismic Weld Demand Category A, B or C, and as Seismic Weld Consequence Category H, M and L. (See Part II, Tables 5-1 and 5-2)

Commentary: The required extent of inspection and special inspection of welded joints in this specification is keyed to the Seismic Weld Demand Category, the Seismic Weld Consequence Category and the primary loading direction. The contract documents must clearly identify the Seismic Weld Demand Category and Seismic Weld Consequence Category for all welded joints in the SFRS. In addition, each such welded joint should be identified as to whether the primary loading of the joint is longitudinal or transverse. If a welded joint is subjected to loading in both directions, it should be classified as that direction in which the

loading is most severe. If the loading severity in both directions is comparable, loading direction should be classified as transverse.

FEMA-350, FEMA-351, and FEMA-352 provide recommended Seismic Weld Demand Category, Seismic Weld Consequence Category, and Loading Direction classifications for welds in prequalified connections and repairs. These classifications are of the form AH/T where the first letter (A, B, or C) represents the Seismic Weld Demand Category, the second letter (H, M, or L) represents the Seismic Weld Consequence Category and the final letter (T or L) represents the primary loading direction. Refer also to Part II of these Recommendations, Chapter 5, for additional information on this classification system.

1.3.19 Structural Observation

Structural Observation is the visual observation of the structural system by a registered design professional, for general conformance to the approved construction documents, at significant construction stages and at completion of the structural system. (See Part II, Section 1.2.14)

Commentary: Structural Observation is defined in IBC Section 1702 and in FEMA-302, Section 3.5.

1.3.20 Structural Steel Framing

Structural steel framing shall be as defined in the AISC *Code of Standard Practice*, Section 2.1.

1.3.21 Written Practice

The Written Practice is the Quality Assurance Agency's procedures for determining and ensuring the acceptability of the structure in accordance with the applicable codes, standards, specifications and procedures, including general inspection, material controls, visual welding inspection, and bolting inspection. The Written Practice also describes the selection and administration of inspection personnel, and the training, experience and examination requirements for qualification and certification of inspection personnel. (See Part II, Sections 1.2.15, and 3.4)

1.4 Submittals

1.4.1 Shop Drawings

The Contractor shall submit to the Engineer detailed, coordinated and checked shop drawings for all structural steel for review prior to the start of fabrication and/or erection. (See Part II, Section 1.4) Shop drawings shall show:

1. size and location of all structural members and connection material,

2. complete information necessary for the fabrication of members including cuts, copes, holes, doubler plates, stiffeners, and camber,
3. type, size and location of bolts and welds,
4. surface preparation and finishes,
5. details of assembly,
6. the Seismic Weld Demand Category and Seismic Weld Consequence Category of welded joints in the Seismic-Force-Resisting System,
7. identification of shop-installed high-strength bolted connections as snug-tight, pretensioned or slip-critical, as required by the Contract Documents, and
8. identification of Welding Procedure Specification (WPS) applicable to each shop weld.

Welded connections shall be identified using standard AWS symbols as given in AWS A2.4, *Standard Symbols for Welding, Brazing and Nondestructive Testing*, and shall clearly distinguish between shop and field welds.

1.4.2 Erection Drawings

The Contractor shall submit to the Engineer detailed, coordinated and checked erection drawings for all structural steel for review prior to the start of fabrication and/or erection. (See Part II, Section 1.4) Erection drawings shall be submitted that show:

1. location of all structural material,
2. identification mark of members,
3. orientation and relation of members to appropriate grid lines,
4. setting elevations for column bases,
5. standard and special details for bolted and welded field connections,
6. Seismic Weld Demand Category and Seismic Weld Consequence Category for field-welded joints in the Seismic-Force-Resisting System,
7. identification of field-installed high strength bolted connections as snug-tight, pretensioned or slip-critical, as required by the Contract Documents, and
8. identification of WPS applicable to each field weld.

Welded connections shall be identified using standard AWS symbols as given in AWS A2.4, *Standard Symbols for Welding, Brazing and Nondestructive Testing*, and shall clearly identify field welds.

1.4.3 Erection Plans

Prior to starting erection, the Contractor shall submit to the Engineer, for record purposes, a description of the methods, scheme of erection, sequence of erection, and type of equipment proposed for use in erecting the structural steel work. Included shall be design information, plans

and elevations showing proposed temporary bracing and shoring systems, with the magnitudes, locations and directionality of construction loads imposed on the permanent structure. The Engineer will have no responsibility to review this submittal nor to approve of the Contractors means and methods.

The steel erector shall also submit for record purposes, written procedures indicating field welding sequences for each type of welded moment connection, and field welding sequences for moment connections at each level. The Engineer will have no responsibility to review this submittal nor to approve of the Contractors means and methods.

Commentary: The Engineer is not obligated to review and provide comment on these submittals. Rather, the purpose of the submittal is to ensure that the Contractor actually does prepare these documents prior to performing the work.

1.4.4 Contractor's Statement of Responsibility

Each Contractor responsible for the work shall submit a written Statement of Responsibility to the Owner and Building Official prior to commencement of the work. (See Part II, Section 2.3) The statement shall contain the following:

- acknowledgment of awareness of the special requirements contained in the Quality Assurance Plan,
- acknowledgment that control will be exercised to obtain conformance with the construction documents,
- procedures for exercising control within the Contractor's organization, the method and frequency of reporting, and the distribution of reports, and
- identification and qualifications of the persons exercising such control and their positions in the organization.

Commentary: The requirement for Contractor's Statement of Responsibility is contained in IBC Section 1705.3 and FEMA-302, Section 3.2.2.

1.4.5 Certificates of Compliance

Coincident with the submittal of manufacturer's test reports and certifications for structural steel, fasteners, welding filler metals, and shear studs, the Contractor shall submit a letter stating that the Contractor has reviewed the submitted manufacturer's certifications and test reports, and that the materials being furnished for the project are in conformance with the applicable standards, specifications and project documents. (See Part II, Sections 4.2, 5.2.1, and 6.2.1)

Commentary: Requirements for Certificates of Compliance are contained in IBC Section 1702.1 and Table 1704.3.

1.4.6 Manufacturer's Test Reports

1.4.6.1 Structural Steel

The fabricator shall submit, for record, copies of material test reports, commonly called mill test reports, for all structural steel in the Seismic-Force-Resisting System. Material test reports shall comply with the requirements of ASTM A6. Material test reports shall be accompanied by a Certificate of Compliance from the fabricator. (See Part II, Section 4.2)

Commentary: Requirements for Manufacturer's Test Reports for structural steel are contained in IBC Section 1702.1 and Table 1704.3(3). However, submittal to the Engineer is not required under the IBC.

1.4.6.2 Fastening Material

The Contractor supplying fastener products shall submit, for record, copies of all Manufacturer's Certifications for all fastener components, including bolts, nuts, washers, and direct tension indicators, if used in the Seismic-Force-Resisting System. Fastener Manufacturer's Certifications shall be accompanied by a Certificate of Compliance from the Contractor. (See Part II, Section 6.2.1)

Manufacturer certifications for bolts shall contain at least the following information:

- heat analysis, heat number, and a statement certifying that heats having prohibited elements intentionally added were not used to produce the bolts,
- results of hardness, tensile, and proof load tests, as required and performed,
- if galvanized, the results of rotational capacity tests, including the test method used (solid plate or tension measuring device) and the lubricant present,
- if galvanized, measured zinc coating weight or thickness,
- results of visual inspection for bursts,
- statement of compliance with dimensional and thread fit requirements,
- lot number and purchase order number,
- complete mailing address of responsible party, and
- title and signature of the individual assigned certification responsibility by the company officers.

For A490 bolts, only the Production Lot Method of testing and certification is acceptable. The Shipping Lot Method is unacceptable.

Commentary: Requirements for Manufacturer's Test Reports for fastening materials are contained in IBC Section 1702.1 and Table 1704(1). The suggested list of information to be provided is based on the certification requirements contained in ASTM A325. The IBC does not require submittal of these reports.

1.4.6.3 Welding Material

The Contractor supplying welding filler metal and shielding gas products shall submit copies of all Manufacturer's Certifications for all electrodes, fluxes and shielding gasses to be used. Certifications shall satisfy the applicable AWS A5 requirements. The welding material's Manufacturer's Certifications shall be accompanied by a Certificate of Compliance from the Contractor supplying the materials. (See Part II, Section 5.2.1)

The Contractor shall also submit the applicable manufacturer's supplemental certifications that the product meets any additional requirements of the project beyond that required by standard AWS A5 specification. Should the welding material manufacturer not supply such supplemental certifications as required, the Contractor shall have the necessary testing performed and provide the applicable test reports.

Commentary: Basic requirements for Manufacturer's Certificates for welding materials are contained in IBC Section 1702.1 and Table 1704.3(4).

1.4.6.4 Shear Connectors (Shear Studs)

The Contractor shall submit the following items:

- stud manufacturer's Manufacturer's Certification that the studs, as supplied, meet the requirements of AWS D1.1, Sections 7.2 and 7.3,
- certified copies of the stud manufacturer's test reports covering the last completed set of in-plant quality control mechanical tests for the diameter supplied, and
- certified material test reports from the steel supplier indicating diameter, chemical properties and grade on each heat number supplied.

The stud manufacturer's Manufacturer's Certification shall be accompanied by a Certificate of Compliance from the Contractor. (See Part II, Sections 4.2 and 5.2)

Commentary: Requirements for Manufacturer's Certification of Shear Connectors are contained in IBC 1702.1 and AWS D1.1, Section 7.3.3. AWS D1.1 does not require submittal of these certifications unless requested by the Engineer.

1.4.7 Procedures

Procedures shall assign responsibility to a person or position in the organization and shall contain enough detail to be useful to the workforce without reference to governing specifications. The procedures need not act as work instructions. Procedures shall be dated and indicate the person or position that has the authority to maintain the procedure.

1.4.7.1 Fastener Installation Procedures

The Contractor shall submit written procedures for the pre-installation testing, installation, snugging, pretensioning , and post-installation inspection of fasteners. The procedures shall meet

all requirements of the Research Council on Structural Connections (RCSC) Specification and the project documents. Procedures need be submitted only for the methods of installation to be used by the Contractor, which may include the turn-of-nut, calibrated wrench, twist-off type tension control bolt, and direct tension indicator methods. (See Part II, Section 6.2.2)

1.4.7.2 Welding Procedure Specifications (WPSs)

The Contractor shall submit all Welding Procedure Specifications (WPSs) to be used by the Contractor on the project. For WPSs that are not prequalified per AWS D1.1, the supporting Procedure Qualification Record (PQR) shall also be submitted with the WPS. The Contractor shall also submit the manufacturer's product data sheets for all welding material to be used. The data sheets shall describe the product, limitations of use, recommended welding parameters, and storage and exposure requirements, including baking and rebaking, if applicable. (See Part II, Section 5.2.2)

Welding Procedure Specifications (WPSs) shall be categorized by and specify the following items:

- steel specifications and grades to be welded,
- thickness range of material to be joined,
- type of joint,
- type of weld (groove, fillet, plug, slot),
- size of weld, and
- position of welding.

Based upon the application, the WPS shall specify, as a minimum, the following items, as applicable for the welding process:

- power supply (constant current or constant voltage),
- welding electrode, flux, and shielding gas classifications,
- welding electrode and flux manufacturer and trade name,
- electrode diameter,
- voltage (except for shielded metal arc welding),
- current (amperage) or wire feed speed,
- electrical stick-out or contact tube-to-work distance (wire fed),
- travel speed,
- minimum preheat and interpass temperatures,
- maximum preheat and interpass temperatures (if applicable),
- number and placement of passes,

- technique (stringer or weave bead),
- shielding gas flow rate,
- for groove welds, the joint configuration and tolerances, and
- other pertinent information specific to the weld to be made.

Tolerances, or the acceptable range of values, applicable to the various welding parameters shall also be noted in the WPS.

Commentary: AWS D1.1 Section 4.1.1 requires WPS submittal only for nonprequalified procedures. The AISC Seismic Provisions, Section 7.3a requires submittal for all procedures.

1.4.7.3 Welding Performance Qualification Records (WPQRs)

The Contractor shall submit written Welding Performance Qualification Records (WPQRs) for all welding personnel under the Contractor's supervision who will be performing services on the project. The WPQR shall document the successful completion of the appropriate welding personnel qualification test. All welder qualification testing shall be performed in accordance with the current or a previous version of AWS D1.1. The Contractor shall also submit additional documentation that the welder has passed all designated supplemental welder qualification testing required for the types of welding to be performed. (See Section Part II, 5.2.3)

Should the WPQR testing have been performed more than six months prior to the start of the welding by the welder, documentation shall also be submitted showing that the welder has continued to use the applicable welding process on an ongoing basis since the test was conducted, with no lapse in service exceeding six months.

The Welding Performance Qualification Records shall, as a minimum, provide the following information:

- welding process,
- type of weld,
- welding position,
- thickness of test plate,
- thickness and nominal diameter of tubular test piece, if applicable,
- if vertical position, whether upward or downward progression,
- electrode group (F-number) classification (if SMAW),
- test results,
- signature of Contractor's individual responsible for the test, and
- date of test.

If the supplemental testing in Appendix B is required, the Contractor shall submit additional WPQRs for those tests.

Commentary: AWS D1.1, Section 4.1.2 sets the requirements for WPQRs, although submittal is not required under that document.

1.4.7.4 Inspector Qualifications

The Contractor shall submit written qualifications for all inspectors to be assigned Quality Control functions for the structural steel work, including general inspection, bolting inspection, welding inspection, and nondestructive testing. (See Part II, Section 2.2)

1.4.7.5 Contractor's Quality Control Plan

The Contractor shall submit the contractor's written Quality Control Plan, including any required modifications to satisfy the requirements for the project, and the Quality Assurance Plan. (See Part II, Section 2.2)

Commentary: IBC Section 1704.2.1 requires that the Contractor have a Quality Control Plan, although it does not require submittal.

1.4.7.6 Samples

Material samples shall be provided as requested by the Engineer or Quality Assurance Agency. Routine physical verification of structural steel, fastener, or welding material composition or mechanical properties is not required. (See Part II, Section 4.5)

Commentary: It is not common to require submittal of sample materials of structural steel or connection materials. If the Engineer desires such samples, this requirement and the number and type of samples required must specifically be identified in the Contract Documents.

1.5 Pre-Fabrication / Pre-Erection Conferences

Prior to performing any fabrication or erection work, the Engineer, Quality Assurance Agency, and Special Inspector, together with Steel Fabricator personnel and Steel Erector personnel supervising the shop, field and Quality Control work shall hold a Pre-Fabrication and Pre-Erection Conference to review welding procedures, bolting procedures, and inspection requirements for all welding and bolting operations. (See Part II, Section 1.3)

Commentary: Conferences between the Engineer, Special Inspector and fabrication and erection personnel are not required by code but are an effective means of ensuring that all responsible parties understand the quality requirements of the project and how they are to be monitored. It is advisable to hold this conference early in the project, before any actual construction work starts. In most cases, it may be appropriate to hold separate conferences with shop fabrication and field erection personnel. Welding Procedure Specifications,

*Quality Control (QC) Plan and Quality Assurance (QA) Written Practice
submittals and reviews should be made before the conferences.*

Part I

2. PRODUCTS

2.1 Structural Steel

Structural steel, of the strength and grade shown on the design drawings shall conform to the requirements of the following ASTM standards, as applicable.

- (a) A36 – *Specification for Carbon Structural Steel*
- (b) A572 – *Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*
- (c) A913 – *Specification for High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self-Tempering Process*
- (d) A992 – *Standard Specification for Steel for Structural Shapes for Use in Building Framing*

Commentary: Project specifications should list only those material specifications that are actually intended to be incorporated on the project. Those specifications that are not applicable to a specific project should be deleted. It is recommended that structural shapes used in new steel moment-frame seismic-force-resisting systems conform either to ASTM 913, or ASTM 992. Refer to FEMA-350 for additional information on appropriate material specifications for various applications.

2.1.1 Supplemental Requirements for Structural Steel

2.1.1.1 Wide-Flange Structural Sections

All wide-flange steel members comprising the Seismic-Force-Resisting System shall conform to ASTM A992 or ASTM A913. Shapes of ASTM A572, grade 50, mill-certified to AISC Technical Bulletin No. 3 requirements, may be substituted for A992 with the Engineer's approval. Shapes of ASTM A913, grade 50, shall satisfy ASTM Supplementary Requirement S75.

2.1.1.2 Heavy Sections

Heavy Sections, as defined in Section 1.3.15, shall be supplied with Charpy V-notch (CVN) testing in accordance with ASTM A6, *Supplementary Requirement S5*. The test shall meet a minimum average absorbed energy value of 20 ft-lbf at +70°F. Testing shall be in accordance with the current *AISC Specification*.

All Heavy Section column flanges located at welded moment connections shall be ultrasonically examined, prior to welding, for evidence of laminations, inclusions, or other discontinuities in accordance with ASTM A435, *Straight Beam Ultrasonic Examination of Steel Plates*, or ASTM A898, *Straight Beam Ultrasonic Examination of Rolled Steel Structural Shapes*, as applicable. The area to be tested is a zone 3 in. above and below each beam flange connection. For plates, any discontinuity causing a total loss of back reflection that cannot be

contained within a circle the diameter of which is 3 inches, or one-half the plate thickness, whichever is greater, shall be rejected. For shapes, ASTM 898 Level I criteria are applicable.

If beams in the Seismic-Force-Resisting System are moment-connected to the weak axis of columns, the column web shall be similarly examined to the above criteria, if greater than 2 inches thick.

Commentary: This requirement is applicable to structural steel in building applications that are normally enclosed and heated. For applications in cold regions, where structural steel will be exposed to low service temperatures, supplemental toughness requirements may be appropriate. Additional information on these toughness requirements may be found in the Commentary to Section A.3C of the AISC LRFD Specifications.

2.2 Fasteners

Fasteners shall conform to the requirements of the following ASTM standards, as applicable, for the strength, grade and type shown on the design drawings:

- (a) A325 – *Specification for Structural Bolts, Steel, Heat-Treated, 120/105 ksi Minimum Tensile Strength*
- (b) A490 – *Specification for Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength*
- (c) A563 – *Specification for Carbon and Alloy Steel Nuts*
- (d) F436 – *Specification for Hardened Steel Washers*
- (e) F959 – *Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners*
- (f) F1852 – *Specification for “Twist-Off” Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*

Commentary: Bolts connecting structural steel components of the Seismic-Force-Resisting System should normally conform to ASTM A325, ASTM A490, or ASTM F1852 pretensioned as for slip-critical applications. Other bolt materials may be used for connection of other components of the steel structure or for connection of the Seismic-Force-Resisting System to the foundation. See also Section 3.2.2.

2.3 Anchor Rods

Anchor rods shall conform to the requirements of the following ASTM standard:

- (a) F1554 - *Specification for Anchor Bolts, Steel, 36, 55, and 105 ksi Yield Strength*

Other material may be used for anchor rods as permitted by current AISC Specifications.

2.4 Welding Material

Welding materials shall conform to the following AWS specifications, as applicable:

- (a) ANSI/AWS A5.1-91, *Specification for Carbon Steel Electrodes for Shielded Metal Arc Welding*
- (b) ANSI/AWS A5.5-96, *Specification for Low-Alloy Steel Electrodes for Shielded Metal Arc Welding*
- (c) ANSI/AWS A5.17/A5.17M-97, *Specification for Carbon Steel Electrodes and Fluxes for Submerged Arc Welding*
- (d) ANSI/AWS A5.18-93, *Specification for Carbon Steel Electrodes and Rods for Gas Shielded Arc Welding*
- (e) ANSI/AWS A5.20-95, *Specification for Carbon Steel Electrodes for Flux-Cored Arc Welding*
- (f) ANSI/AWS A5.23/A5.23M-97, *Specification for Low-Alloy Steel Electrodes and Fluxes for Submerged Arc Welding*
- (g) ANSI/AWS A5.25/A5.25M-97, *Specification for Carbon and Low-Alloy Steel Electrodes and Fluxes for Electroslag Welding*
- (h) ANSI/AWS A5.26/A5.26M-97, *Specification for Carbon and Low-Alloy Steel Electrodes for Electrogas Welding*
- (i) ANSI/AWS A5.28-96, *Specification for Low-Alloy Steel Electrodes and Rods for Gas Shielded Arc Welding*
- (j) ANSI/AWS A5.29-98, *Specification for Low-Alloy Steel Electrodes for Flux-Cored Arc Welding*
- (k) ANSI/AWS A5.32/A5.32M-97, *Specification for Welding Shielding Gases*

2.4.1 Supplemental Requirements for Welding Materials

2.4.1.1 Toughness, Strength and Elongation

All welds on members comprising the Seismic-Force-Resisting System, except as noted below for joining of material conforming to ASTM A913, grade 65, shall employ weld filler metals classified for nominal 70 ksi tensile strength, referred to as E70 electrodes, meeting the following minimum mechanical property requirements:

1. CVN toughness of 20 ft-lb at 0°F, using AWS A5 classification test methods
2. CVN toughness: 40 ft-lb at 70°F, using the test procedures prescribed in Appendix A

3. yield strength: 58 ksi minimum, using both the AWS A5 classification test (for E70 classification electrodes) and the test procedures prescribed in Appendix A
4. tensile strength: 70 ksi minimum, using both the AWS A5 classification test (for E70 classification electrodes) and the test procedures prescribed in Appendix A
5. elongation: 22% minimum, using both the AWS A5 classification test and the test procedures prescribed in Appendix A.

Welded joints of ASTM A913, grade 65 material shall be made with weld filler metals classified as E80 electrodes with a minimum yield strength of 68 ksi, minimum tensile strength of 80 ksi, and a minimum elongation of 19%. CVN toughness requirements of (1) and (2) above shall apply.

Commentary: For applications where the Seismic-Force-Resisting System is subjected to service temperatures below 50°F following completion of the construction, the CVN toughness requirement for item (2) above should be conducted at a test temperature not less than 20°F above the lowest anticipated service temperature.

For all other joints, welding materials shall meet the requirements of the designated A5 classification test. A minimum CVN toughness is not required, except for the specific joints and designs described in Table 2.3 in the AISC LRFD Specification for Structural Steel Buildings dated December, 1999.

2.4.1.2 Hydrogen Level

For welded joints designated as Seismic Weld Demand Category A or B, welding filler metals, as supplied by the manufacturer, shall meet the requirements for H16 (16 ml diffusible hydrogen per 100 grams deposited weld metal) as tested using the mercury or gas chromatograph method as specified in AWS A4.3, *Standard Methods for Determination of the Diffusible Hydrogen Content of Martensitic, Bainitic, and Ferritic Steel Weld Metal Produced by Arc Welding*. The manufacturer's Certificate of Conformance shall be considered adequate proof that the supplied electrodes meet this requirement, and no additional testing of filler metal samples or of production welds is required.

For all other welds on members of the Seismic-Force-Resisting System, the weld filler metals shall be considered classified as low-hydrogen under the provisions of AWS D1.1.

Commentary: All electrodes and fluxes for FCAW, GMAW, SAW, GTAW, ESW and EGW processes are considered low-hydrogen. For SMAW, electrodes listed in AWS Table 3.1 for steels in Group II or higher are considered low-hydrogen.

2.4.1.3 Packaging Requirements

Packaging of weld filler metals shall conform to the requirements of AWS D.1.1. FCAW electrodes shall be received in moisture-resistant packages that are undamaged. They shall be protected against contamination and injury during shipment and storage. Electrode packages shall

remain effectively sealed against moisture until the electrode is required for use. When removed from protective packaging and installed on machines, care shall be taken to protect the electrodes and coatings, if present, from deterioration or damage. Modification or lubrication of an electrode after manufacture for any reason is not permitted, except that drying shall be permitted when recommended by the manufacturer.

2.5 Shear Connectors (Shear Studs)

Stud Shear Connectors shall be *AWS D1.1* “Type B” headed studs made from *ASTM A108*, Grade 1015 or 1020, cold-finished carbon steel, with dimensions complying with *AWS D1.1* for the type and sizes shown.

Part I

3. EXECUTION

3.1 Fabrication

3.1.1 General

Structural steel shall be shop fabricated in accordance with the applicable *AISC Specification*, *AISC Code of Standard Practice*, and *AWS D1.1*, as modified by the provisions of the contract documents.

High-strength structural steel shall be identified according to grade.

Members shall be marked and match-marked as needed for field assembly. Members shall be fabricated for delivery in a sequence that will expedite erection and minimize field handling of structural steel.

Cuts, holes, and openings in structural steel members, shown on the design drawings, shall also be shown on the shop drawings for structural steel and shall be done in the shop.

The Contractor shall cooperate fully with requests from inspection and testing personnel for access to the connections and joints to be inspected or tested. This includes beam and column turning in the shop, weld backing removal when nondestructive examination indicates rejectable conditions, and access to platforms or scaffolding as required to perform the work safely. Weld backing removal shall also be performed where required by the Contract Documents. When weld backing must be removed for test verification, the inspection must be performed in a timely manner so as not to hinder production nor require duplication of effort.

The Contractor shall conduct all necessary corrections of deficiencies in material and workmanship. Repair procedures shall be submitted to the Engineer for review and acceptance. The Contractor shall comply with requests to correct deficiencies, when such corrections are required by the Engineer or other designated responsible party. The Quality Assurance Inspector and NDT Technician shall be apprised of any repairs made by the Contractor.

3.1.2 Bolted Construction

Standard holes shall be used, unless otherwise indicated on the Drawings. Holes shall be drilled or punched at right angles to the surface of the metal, not more than 1/16 inch larger than the fastener diameter, except where oversize holes are specified. Making or enlarging holes by burning is prohibited, unless hole slotting is required between two punched or drilled holes. Center piercing with a torch and reaming to the required diameter is permitted. Material having a thickness in excess of the connector diameter plus 1/8 inch shall be drilled rather than punched. Burrs that would prohibit solid seating of the connected parts shall be removed.

Connections shall be assembled in accordance with the *RCSC Specification*, using provisions for pretensioned joints, unless snug-tight bolts are indicated on the Drawings. Faying surfaces of

pretensioned joints shall be prepared as for Class A or better slip-critical joints unless otherwise indicated on the Drawings.

3.1.3 Welded Construction

Each welder working on the project shall be assigned an identification symbol or mark. Each welder shall mark or stamp this identification symbol at each weldment completed. Stamps, if used, shall be the low-stress type.

Welding personnel shall be qualified in accordance with AWS D1.1, Section 4, Parts A and C. WPQR testing performed more than six months prior to the start of the welding by the welder is acceptable, provided written documentation is submitted showing that the welder has continued to use the applicable welding process on an ongoing basis since the test was conducted, with no lapse in service exceeding six months. Welders whose work routinely exhibits poor workmanship shall be requalified before performing further welding.

WPSs shall be available to welders and inspectors prior to and during the welding process. Prior to welding, joint fit-up shall be verified by the welder for conformance with the WPS and AWS D1.1.

For all complete joint penetration (CJP) and partial joint penetration (PJP) groove welds subjected to ultrasonic testing (UT), a visible mark, “for UT,” shall be accurately placed on the steel a distance of 4 inches away from the root of the edge preparation.

Welding shall be performed in accordance with the appropriate WPS for the joint.

Groove welds shall be complete joint penetration groove welds, unless specifically designated otherwise on the Drawings. Groove preparation details are at the Contractor's selection, subject to qualification, if required, in accordance with AWS D1.1.

Weld tabs shall be in accordance with AWS D1.1, Section 5.31, as modified by Section 4.2 of this specification. End dams shall not be used.

Backing bars shall be in accordance with AWS D1.1, Section 5.10, as modified by Section 4.1 of this specification.

Faces of fillet and groove welds exposed to view shall have as-welded surfaces that are reasonably smooth and uniform. No finishing or grinding shall be required, except where clearances or fit of other items may so necessitate, or as preparation for coating.

3.1.4 Headed Shear Studs

Shear studs shall be welded in accordance with AWS D1.1, Section 7, and the manufacturer's recommendations. Shear studs shall not be attached in the area designated in Section 5.5 of this specification.

3.2 Erection

3.2.1 General

Structural steel shall be erected in accordance with the *AISC Specification* and the *AISC Code of Standard Practice*, as modified by the project documents.

Where erection requires performing fabrication work on site, the work shall conform to the applicable standards of Section 3.1, Fabrication.

The Contractor shall cooperate fully with requests from inspection and testing personnel for access to the connections and joints to be inspected or tested. This includes weld backing removal when non-destructive testing indicates rejectable conditions, and access to platforms or scaffolding as required to perform the work safely. When such services are required, the inspection must be performed in a timely manner so as not to hinder production nor require duplication of effort.

The Contractor shall perform all necessary corrections of deficiencies in material and workmanship. The Contractor shall comply with requests to correct deficiencies, when such corrections are required by the Engineer or other designated responsible party. The Quality Assurance Inspector and NDT Technician should be apprised of any repairs to be made by the Contractor, prior to performance.

3.2.1.1 Storage and Handling

Structural steel shall be stored and handled in a manner that prevents damage or distortion. Do not store materials on the structure in a manner that might cause distortion or damage to members of the supporting structure. Store materials to permit easy access for inspection and identification. Keep steel members off the ground by using blocking, cribbing, platforms, or other supports.

3.2.1.2 Temporary Shoring and Bracing

Contractor's erection scheme and temporary bracing and shoring shall provide lateral support and stability for all columns until permanent lateral-support systems (for example: wind bracing bents and horizontal deck/slab diaphragms) are in place and complete. Provide temporary shoring and bracing members in vertical and horizontal planes as required, with connections of sufficient strength to bear the imposed loads. Remove temporary members and connections when permanent support systems are in place and final connections are made.

3.2.1.3 Erection Procedures

The Contractor shall be responsible for the control of all erection procedures and sequences including but not limited to temperature differentials and weld shrinkage.

3.2.1.4 Field Work for Other Trades

There shall be no field cutting of structural steel members in the Seismic-Force-Resisting System for the work of other trades without written prior approval of the Engineer.

3.2.2 Bolted Construction

Fasteners shall be stored in a protected place. Except for ASTM F1852 “twist-off” type assemblies, clean and relubricate bolts, nuts and washers that become dry or rusty before use. F1852 fastener components may be relubricated following the manufacturer’s written instructions, and must be retested after relubrication and prior to use to verify suitability for installation.

Do not use flame cutting to align bolt holes except as permitted by RCSC specifications.

Ream holes that must be enlarged to admit bolts. Do not enlarge holes by more than 1/32 in. Additional reaming beyond 1/32 in. shall not be performed without approval of Engineer. When reaming beyond 1/32 in. is approved, drill or ream to the next larger hole size and use the next larger size bolt.

Bolt holes may be made by punching or drilling. Bolt holes may also be made by thermal cutting to a smaller diameter, followed by reaming to the required diameter.

3.2.3 Welded Construction

See Section 3.1.3.

3.2.4 Headed Shear Studs

See Section 3.1.4.

3.2.5 Corrective Work

There shall be no field cutting or alteration of structural steel members or connections in the Seismic-Force-Resisting System without prior review by the Engineer. Structural elements having fabrication errors or which do not satisfy tolerance limits shall be repaired. Submit drawings showing reasons for, and details of, proposed corrective work for approval by the Engineer prior to performing corrective work. Corrective work shall be performed in accordance with the requirements of the Contract Documents. Pre-approved repair and correction procedures may be used when authorized by the Engineer for specific conditions.

3.2.6 Thermal Cutting

Thermal cutting torches may be used in the field for correcting fabrication errors in structural framing only with the approval of the Engineer. When thermal cutting is permitted, cutting shall be done only with a mechanically guided torch or a torch controlled using a guide bar, except as permitted below:

1. Thermal-cut edges that are not welded and will be free of substantial stresses, as determined by the Engineer, may be cut manually with an unguided torch provided the specified AISC edge distances to holes are maintained.
2. Other thermal-cut edges and edges to be welded may be cut manually with an unguided torch to a line within 1/8 in. of the finished dimension, with final removal of material completed by chipping or grinding to produce a surface quality equal to that of the base metal edges.

Cuts shall be smooth and regular in contour, and free of notches. The radius of reentrant corners shall be no less than one-half inch. Cuts performed to modify beam copes may be made with a smaller radius, but shall not be cut square.

Commentary: Thermal cutting of main members of the seismic-force-resisting system in the field should be performed with caution. If members are under load, when thermally cut, the resulting loss of strength could result in damage or failure of the member. Particular caution should be exercised with regard to thermal cutting in areas anticipated to experience yielding or buckling during earthquake response as the formation of martensite or the presence of excessive surface roughness or nicks resulting from such cutting can serve as initiators for fractures under high-strain cyclic loading. If proper precautions are taken, including shoring of heavily loaded members, use of preheat prior to cutting, and grinding to remove surface defects, field thermal cutting can be safely performed.

3.3 Supplemental Welding Requirements

3.3.1 Welding Personnel Qualification

3.3.1.1 Supplemental Welding Personnel Testing

Welders and welding operators performing work on welds classified as Seismic Weld Demand Categories A or B shall pass Supplemental Welder Qualification Testing, as prescribed in Appendix B, on special test joint mock-ups. Testing shall be performed using the process to be used in the work, with the WPS set at the highest deposition rate to be used in the work. FCAW-S and FCAW-G shall be considered separate processes for welding personnel qualification. Tack welders need not perform such Supplemental Testing.

3.3.1.2 Qualification Period

Welding personnel required to be tested using the Supplemental Welding Personnel Testing prescribed in Section 3.3.1.1 shall be qualified by test within 12 months prior to beginning welding on the project.

Should the 12-month period elapse during welding on the project, the Supplemental Welder Qualification remains valid. It is not required to repeat the testing during the course of the project, unless the quality of the workmanship for that welder routinely fails to meet the applicable project weld quality standards.

3.3.2 Intermix of Filler Metals

For welded joints requiring CVN toughness in Seismic Weld Demand Categories A and B, when FCAW-S filler metals will be used in combination with filler metals for other processes, including FCAW-G, supplemental toughness testing shall be conducted as prescribed in Appendix C.

3.3.3 Electrode Storage and Exposure Limits

FCAW electrodes shall be received and stored in the original, undamaged manufacturer packaging, until ready for use. Electrodes in packages that have had the internal plastic wrapping damaged shall not be used for welds in Seismic Weld Demand Category A or B. Modification or lubrication of an electrode after manufacture is not permitted, except that drying is permitted as recommended by the manufacturer.

The exposure time limit for all FCAW electrodes shall be based upon the results of tests as prescribed in Appendix D. These tests may be conducted by the electrode manufacturer or supplier, by the Contractor, or by an independent testing agency or laboratory with suitable equipment.

In lieu of testing, when welding is to be suspended for more than 8 hours, electrodes shall be removed from the machines and stored in an electrode wire oven maintained at a temperature between 250° and 550° F, or as recommended by the manufacturer. Electrodes not consumed within 24 hours of accumulated exposure outside closed or heated storage shall not be used for welds in Seismic Weld Demand Category A or B. Electrode spools shall be identified to facilitate monitoring of total atmospheric exposure time. FCAW electrodes that have been exposed for periods exceeding an accumulated 24 hours may be dried if manufacturer's testing and recommendations show that drying is effective at removing moisture and restoring electrodes to their designated diffusible hydrogen level.

3.3.4 Wind Velocity Limits

In lieu of AWS D1.1-98, Section 5.12.1, GMAW, FCAW-G, GTAW and EGW shall not be performed when the wind velocity in the immediate vicinity of the weld exceeds three miles per hour. Welding performed within an enclosed area, and not subject to drafts may be deemed to satisfy this requirement.

SMAW, FCAW-S, and SAW may be performed without limitation to wind velocity, provided the wind does not affect the appearance of the molten weld puddle.

3.3.5 Minimum Preheat and Interpass Temperature

Minimum preheat and interpass temperatures shall be provided for all welds, including tack welds, in accordance with AWS D1.1, Table 3.2. The Contractor may specify higher minimum temperatures, if desired, as a part of the Contractor's WPS for a particular application. In such cases, the WPS minimum preheat and interpass temperatures shall be provided. For welds in

Seismic Demand Categories A, B, or C, preheat and interpass temperatures shall be in the range tested in accordance with Appendix A.

Preheat and interpass temperatures lower than those required by AWS D1.1, Table 3.2, are permitted provided the WPS has been qualified by test, and the WPS and PQR have been accepted by the Engineer.

Minimum preheat and interpass temperatures shall be verified at a distance of 3 in. from the weld, at the point of arc initiation or for materials over 3 in. in thickness, at a distance equal to the thickness of the part.

3.3.6 Maximum Preheat and Interpass Temperature

The maximum preheat and maximum interpass temperature permitted is 550°F, measured at a distance of 1 in. from the point of arc initiation. This maximum temperature may not be increased by the WPS, regardless of qualification testing.

Commentary: Limitations on maximum preheat and interpass temperature are imposed because laboratory data indicates that high heat input results in degradation of weld toughness. Use of qualified-by-test procedures with high heat inputs are not permitted because standard procedure qualification tests do not include evaluation of CVN toughness.

3.3.7 Nonfusible Backing

The use of nonfusible backing materials, including ceramic and copper, is permitted only with satisfactory welder qualification testing performed using the type of backing proposed for use, using the test plate shown in AWS D1.1-98, Figure 4.21, except that groove dimensions shall be as provided in the WPS and PQR. Should the joint to be welded include welding a beam flange to a column flange through an access hole, the Supplementary Welder Qualification Test of Appendix B shall be performed using the type of backing proposed for use.

For nonfusible weld tabs and short segments of nonfusible backing bars used at the ends of welds between shear tabs and column faces, or at the ends of continuity plate welds, special welding personnel and welding procedure qualification testing is not required. The welder shall be trained in the proper welding techniques for using such nonfusible weld tabs and backing bars prior to performing such welding on the project.

3.3.8 Peening

The use of peening is permitted at the Contractor's option, but is not required. See AWS D1.1-98, Section 5.27. If the Contractor elects to use peening, a written procedure for performing peening shall be incorporated into the WPS for the joints to be peened.

3.3.9 Controlled Cooling

The use of controlled cooling is permitted at the Contractor's option, but is not required. If the Contractor elects to use controlled cooling, a written procedure for controlling cooling after welding shall be incorporated into the WPS for the joints to receive such treatment. The procedure shall include the method of heating, the maximum temperature permitted, cooling rate range to be provided, and the method and frequency of temperature measurements.

The use of insulating blankets after completion of welding, without the addition of heat, is permitted at the Contractor's option, but is not required. For the use of insulating blankets only, a written procedure and temperature measurements are not required.

3.3.10 Post Weld Heat Treatment

The use of Post Weld Heat Treatment (PWHT) is permitted at the Contractor's option, but is not required. The use of PWHT shall meet the Stress-Relief Heat Treatment limitations of AWS *D1.1-98*, Section 5.8. If the Contractor elects to use PWHT, a written procedure for performing PWHT shall be incorporated into the WPS for the joints to be treated.

Controlled cooling, or the application of heat immediately following completion of a joint to maintain a nominal temperature at or below 550°F, is not considered PWHT.

Part I

4. WELDED JOINT DETAILS

4.1 Backing Bars

The use of backing bars shall be in accordance with AWS D1.1-98, Section 5.10. All tack welds attaching backing bars to the steel prior to the welding of the joint shall be made within the joint. Preheat for such tack welds as required by AWS D1.1, Table 3.2, or by the WPS, as applicable, shall be provided. Backing bars may remain in place unless required to be removed by Section 4.1.1 or 4.1.2 of this specification, by AWS D1.1-98, Section 5.10.4, or by AISC Specification Section J1.5.

4.1.1 Heavy Section Splices Requiring Removal of Backing Bars

All welded tension splices of Heavy Sections, as defined in Section 1.3.5, shall have the backing bars removed. Where fusible backing material is used, the root pass area shall be backgouged after backing bar removal, and backwelded until flush or with slight reinforcement. The surface shall then be ground smooth, to a surface roughness not to exceed 500 microinches. Notches and gouges shall be repaired to the requirements of AWS D1.1-98, Section 5.15.4.4.

4.1.2 Moment Connection Joints Requiring Removal of Backing Bars

Backing bars shall be removed from the joint when required on the design drawings. Following removal of backing, the root pass shall be backgouged to sound weld metal, and backwelded.

Backing bar removal may be performed by air carbon arc cutting (CAC-A), commonly referred to as air carbon arc gouging (ACAG), grinding, chipping, or thermal cutting. The process shall be controlled to minimize gouging and removal of base metal except for material immediately adjacent to the weld.

Following backgouging, the root shall be backwelded. A reinforcing fillet weld with a minimum leg size of 5/16 inch or the root opening plus 1/16 inch, whichever is larger, shall be provided. The reinforcing fillet weld need not be ground. Following completion of the reinforcing fillet weld, MT shall be performed on the fillet weld and the immediately adjacent area.

Should these joints be made with nonfusible backing, the backing material shall be removed, any unacceptable weld discontinuities removed by backgouging, the root backwelded as needed, the reinforcing fillet added as described above, and MT performed on the completed fillet weld and the immediately adjacent area. If visual inspection of the root shows no unacceptable discontinuities, no backgouging and backwelding is required.

Commentary: Backing removal is required at the beam-bottom-flange-to-column joint of the following types of prequalified moment-resisting connections: Welded

Unreinforced Flange (WUF) connections, Reduced Beam Section (RBS) connections, and Free Flange (FF) connections, Welded Flange Plate (WFP) connections, Bolted Flange Plate (BFP) connections. In addition, backing should be removed from any connections subject to project-specific qualification testing, if the backing was removed from the qualification test specimens.

4.2 Weld Tabs

4.2.1 Use of Weld Tabs

Welds shall be terminated at the end of a joint in a manner that will ensure sound welds. Whenever necessary, this shall be done by use of weld tabs, also called extension bars and run-off tabs. Weld tabs shall extend beyond the edge of the joint a distance equal to a minimum of the part thickness, but not less than one inch. Weld tabs shall be oriented parallel to the joint preparation and to the weld direction. No weld dams are allowed.

Except as noted in the following sections, weld tabs shall meet the requirements of AWS D1.1-98, Section 5.31. Fusible weld tabs may remain in place unless their removal is required by the following sections, by AWS D1.1-98, Section 5.31, or by AISC Specification Section J1.5. Nonfusible weld tabs may be used in applications and locations where qualified in accordance with AWS D1.1, Section 4.

4.2.2 Heavy Section Joint Weld Tab Removal and Finish

All welded tension splices in Heavy Sections, as defined in Section 1.3.5, shall have the weld tabs removed and ground smooth to a finish of 500 microinches or better. Gouges and notches shall satisfy the requirements of AWS D1.1-98, Section 5.15.4.4, or shall be repaired to the provisions of AWS D1.1-98, Section 5.15.4.4.

4.2.3 Moment Connection Weld Tab Removal and Finish

Weld tabs shall be removed when required by the design drawings. Weld tab removal may be performed by air carbon arc cutting (CAC-A), grinding, chipping, or thermal cutting to within 1/8 inch of the base metal surface. For weld tabs used on continuity plates, removal within 1/4 inch of the plate edge is adequate. The process shall be controlled to minimize gouging and removal of base metal except for that material immediately adjacent to the weld.

The edges where the weld tabs have been removed shall be finished to a surface roughness value of 500 microinches or better. Grinding to a flush condition is not required. Gouges and notches are not permitted, and must be removed by grinding. The transitional slope of any area where gouges and notches have been removed shall not exceed 1:5. Material removed by grinding that extends more than 1/16 inch below the surface of the base metal shall be filled with weld metal using approved weld repair procedures.

The contour of the weld at the ends shall provide a smooth transition, free of notches and sharp corners. A minimum radius at the corner need not be provided.

Following removal and finishing to the required smoothness and contour, and the completion of any necessary repairs, the exposed ends of the weld shall be inspected using magnetic particle testing (MT).

Commentary: Weld tabs should be specified to be removed from each end of both the top and bottom beam flanges for the following prequalified beam-to-column connections: Welded Unreinforced Flange (WUF) connections, Reduced Beam Section (RBS) connections, Welded Flange Plate (WFP) connections, Free Flange (FF) connections, Bolted Flange Plate (BFP) connections, Welded Bottom Haunch (WBH) connections, Welded Top and Bottom Haunch (WTBH) connections, Welded Cover Plated Flange (WCPF) connections, and Improved Welded Unreinforced Flange (IWURF) connections. Weld tabs should also be removed from connections subject to project specific qualification if the tabs were removed from the qualification test specimens.

4.3 Weld Toes

Weld toes, whether for groove welds or fillet welds, shall provide a smooth transition between the weld and base metal. The as-welded profile is adequate provided it satisfies the criteria of AWS D1.1, Section 5.24. No grinding is required.

4.4 Weld Access Holes

Weld access holes shall meet the dimensional, surface finish, and testing requirements of AISC LRFD Specification Section J1.6 and AWS D1.1-98, Section 5.17.1 and Figure 5.2, except as otherwise required by the Contract Documents. The provisions in AWS D1.1 Section 5.17.2 and Figure 5.2, Note 1 shall apply to all Heavy Sections as defined in this document.

The access hole shall be ground smooth to a surface roughness value not to exceed 500 microinches, and shall be free of notches and gouges. For this purpose, a notch or gouge is any depression deeper than the overall surface roughness.

Notches or gouges present from thermal cutting shall be removed by grinding, faired to a slope of not more than 1:5 against a straight cut surface, or to a radius of not less than 3/8 inch if in the curved portion of the cut surface. The depth of notches and gouges that may be repaired by grinding is not limited, provided the required dimensions, including tolerances, of the access hole are maintained.

Notches deeper than can be repaired by grinding (as above) may be repaired by welding. Prior to welding, the notch or gouge shall be ground to provide a smooth contour with a radius not less than 3/8 inch. The repair area shall be preheated to a temperature between 400°F and 550°F, measured at the point of welding immediately prior to welding. Filler metal meeting the

requirements for Seismic Weld Demand Categories A and B shall be used. A written repair WPS for the application shall be followed. Following completion of welding, the area shall be ground smooth and flush to meet the contour and finish requirements for the access holes, with fairing of the welded surface to adjoining surfaces.

Prior to acceptance, the weld access hole shall be inspected using magnetic particle testing (MT) or liquid penetrant testing (PT) and shall be free of cracks. If a welded repair has been performed, magnetic particle testing (MT) shall be performed.

4.5 Web Weld Details

Unless otherwise shown on the drawings, shear tabs may be welded to the column using fillet welds, partial joint penetration (PJP) groove welds with reinforcing fillet welds, a combination of fillet weld and PJP groove weld, or a CJP groove weld.

When required by the design drawings, the beam web shall be welded to the shear tab using fillet welds. A minimum clear distance of ½ inch shall be provided between the weld access hole and the toe of the fillet weld connecting the shear tab and beam web.

Fillet welds should terminate a distance equal to or greater than the leg size from the beam end.

4.6 Doubler Plate Details

Web doubler plates, if required, may be welded using any of the three details of the *AISC Seismic Provisions*, Figure C-9.3.

When Figure C-9.3(a) is used, the edges of the doubler plate shall be chamfered to an adequate bevel to facilitate access to the root of the weld. A square-edge plate and square groove weld between doubler plate and column is not acceptable. No grinding of the completed weld is required.

When Figure C-9.3(b) is used, the plate shall be chamfered to miss the radius of the column. The fillet weld, in both throat and leg size, as a minimum, shall be equal to or larger than the chamfer dimensions used for the doubler plate. No grinding of the completed weld is required.

4.7 Column Continuity Plate Details

Continuity plates, also referred to as stiffeners, shall have clips, sized to avoid interference with the radius of the column. Against the column flange, the size of the clip may exceed the radius (k_1 dimension) by no more than 1/2 inch. Along the web, the clip shall extend a distance of approximately 1-1/2 inches beyond the published k dimension. The clip shall be detailed to facilitate suitable weld terminations for both the flange weld and the web weld, with a minimum radius of ½ inch.

The weld between the continuity plate and the column flange, unless otherwise shown, shall be a CJP groove weld for the full length of the groove preparation. The joint may use backing

bars, or may be made without backing provided the root is backgouged and backwelded. If backing bars are used and remain in place, they shall receive a reinforcing fillet weld between the backing bar and column flange. No fillet weld should be placed between backing bar and continuity plate. The fillet weld size need not exceed the minimum size requirements of AWS *D1.1*, Table 5.8.

Weld terminations near the end of the column flange tips may be completed using weld tabs. Weld tabs, if used, may be steel or nonfusible material. After welding completion, weld tabs shall be removed to within $\frac{1}{4}$ inch of the continuity plate edge and the surface finished to a surface roughness not to exceed 500 microinches. Removal or grinding to the flush condition is not required. Following finishing, the edge shall be inspected using magnetic particle testing.

For column flange to continuity plate welds, weld terminations near the radius of the column need not be made using weld tabs. The use of small nonfusible weld tabs to assist in weld terminations is permitted. Weld tabs shall be removed following completion of welding, but no grinding is required.

The weld between the continuity plate and column web may be made with groove welds, fillet welds, or a combination of the two. The weld termination should be held back from the end of the joint at each end a distance of approximately $\frac{1}{4}$ inch.

4.8 Welding Sequence for Moment Connection of Bottom Beam Flange

When welding the bottom flange to the column flange of welded moment-resisting connections, the following sequence shall be followed:

1. When welding from Side A (one side of the beam), the root pass shall begin beyond the center of the joint on Side B, reaching past the beam web (or web plate, for FF connections) through the weld access hole (or opening, for FF connections). After the arc is initiated, travel shall progress toward the edge of the Side A beam flange, and the weld shall be terminated on the Side A weld tab.
2. The Side A root pass, and the root pass deposit on Side B, shall be thoroughly cleaned and visually inspected by the welder to ensure fusion, soundness, and freedom from cracks, slag inclusions and excessive porosity. The resulting bead profile shall be suitable for obtaining good fusion by the subsequent root pass to be initiated from Side B. If the profile is not conducive to good fusion, the start of the first root pass shall be ground, gouged, chipped, or otherwise prepared to ensure adequate profile to achieve fusion.
3. The second half of the weld joint, from Side B, shall have the root pass applied before any other weld passes are performed. The arc shall be initiated in the area of the start of the first Side A root pass, and travel shall progress to the end of the joint, terminating on the Side B weld tab.
4. The above sequence shall be repeated for subsequent weld layers, and each weld layer shall be completed on both sides of the joint before a new layer is deposited. The order of

operations (Side A, then Side B, or vice versa) is not restricted and may vary for each weld layer.

Weld passes shall be placed in horizontal layers. Each pass shall be thoroughly cleaned of slag and wire brushed. Each pass shall be visually inspected by the welder, as described above in Step 2.

Both top and bottom beam flanges should be completely welded prior to any supplemental welding to the beam web or shear tab, unless otherwise detailed in the approved erection plan and the WPS.

Commentary: The above welding sequence is recommended for the following types of joints: Welded Unreinforced Flange (WUF) connections, Reduced Beam Section (RBS) connections, Welded Free Flange (FF) connections, Welded Bottom Haunch (WBH) connections, and Welded Top and Bottom Haunch (WTBH) connections.

4.9 Improved Welded Unreinforced Flange Connection Details

The following provisions are applicable to upgrading existing connections using improvements to the existing beam flange welds only.

4.9.1 Existing Web Connection Materials and Details

The existing web connection and web access holes may remain as constructed.

4.9.2 Replacement of Beam Flange Welds

Where required on the design drawings, the existing beam-to-column flange welds shall be removed by air carbon arc cutting (CAC-A), chipping or grinding until only base metal remains. The joint shall be prepared for new groove welding, using a joint detail that uses a backing bar. The actual root opening is permitted to exceed the prequalified root opening of AWS D1.1, Figure 3.4, provided a split-layer technique is used for the placement of the root pass. The groove angle of the joint shall satisfy the prequalified groove angles of AWS D1.1 Figure 3.4, unless otherwise qualified under the provisions of AWS D1.1, Section 4.

At the top flange, if the backing bar is left in place, a 5/16 inch reinforcing fillet shall be placed between the backing bar and the column. No weld should be placed between the backing bar and beam flange. At the bottom flange, following the provisions of Section 4.1.2 of this specification, the backing bar shall be removed, the weld root backgouged to solid weld metal, and the area backwelded until at least flush. A 5/16 inch reinforcing fillet shall be placed between the weld root and the column at the bottom of the bottom flange.

Continuity plates, if added, shall conform to the requirements of Section 4.8 of this specification.

4.9.3 New Welding Material

All new welding filler metals shall meet the requirements of Section 2.4 of this specification. Weld tab removal shall be as required in Section 4.2.

4.10 Haunched Connection Welding Details

The following provisions are applicable to upgrading existing connections with either a bottom haunch or a top and bottom haunch, as shown on the design drawings.

4.10.1 Existing Web Connection Materials and Details

The existing web connection and web access holes may remain as constructed.

4.10.2 Flange Weld Details

Where shown on the design drawings, the existing beam-to-column flange welds shall be removed by air carbon arc cutting (CAC-A), chipping or grinding until only base metal remains. The joint shall be prepared for new groove welding, using a joint detail that uses a backing bar. The actual root opening is permitted to exceed the prequalified root opening of AWS D1.1, Figure 3.4, provided a split-layer technique is used for the placement of the root pass. The groove angle of the joint shall satisfy the prequalified groove angles of AWS D1.1 Figure 3.4, unless otherwise qualified under the provisions of AWS D1.1, Section 4.

At the top flange, if the backing bar is left in place, a 5/16 inch reinforcing fillet shall be placed between the backing bar and the column. No weld should be placed between the backing bar and beam flange. At the bottom flange, following the provisions of Section 4.1.2 of this specification, the backing bar shall be removed, the weld root backgouged to solid weld metal, and the area backwelded until at least flush. A 5/16 inch reinforcing fillet shall be placed between the weld root and the column at the bottom of the bottom flange. No fillet weld should be placed between the backing bar and beam flange.

4.10.3 New Welding Material

All new welding filler metals shall meet the requirements of Section 2.4 of this specification. Welding shall be executed in conformance with the requirements of Chapter 3 of this specification. Weld tab removal shall be as required in Section 4.2 of this specification.

4.10.4 Haunch Welding Requirements

The groove welds between column and WT haunch may be made with or without backing, but if backing is used, an access hole meeting the minimum dimensional requirements of AWS D1.1, Section 5.17.1 and AWS D1.1 Figure 5.2 shall be used. Backing bars, if used, may remain in place, but if left in place, shall receive a reinforcing fillet. The weld between the WT stem and the beam flange shall be a CJP groove weld. Backing bars, if used, may remain in place, and no reinforcing fillet is required.

The weld between the WT stem and the column flange shall be a CJP groove weld. Backing bars, if used, may remain in place, but if left in place, a reinforcing fillet is required.

4.10.5 Continuity Plates and Stiffener Welding Requirements

New continuity plates added to the column shall meet the requirements of Section 4.7 of this specification. Stiffeners added to the beam web shall meet the requirements of Section 4.7 of this specification, except that the welds between the stiffener and the beam web may be fillet welds of minimum 5/16 inch leg size.

4.11 Cover-Plated Moment-Connection Details

The following provisions are applicable to upgrading existing connections with cover plates when shown on the design drawings.

4.11.1 Existing Web Connection Materials and Details

The existing web connection and web access holes may remain as constructed.

4.11.2 Flange Weld Details

Where shown on the design drawings, the existing beam-to-column flange welds shall be removed by air carbon arc cutting (CAC-A), chipping or grinding until only base metal remains. The joint shall be prepared for new groove welding, using a joint detail that uses a backing bar. The actual root opening is permitted to exceed the prequalified root opening of AWS D1.1, Figure 3.4, provided a split-layer technique is used for the placement of the root pass. The groove angle of the joint shall satisfy the prequalified groove angles of AWS D1.1 Figure 3.4, unless otherwise qualified under the provisions of AWS D1.1, Section 4.

4.11.3 New Welding Material

All new welding filler metals shall meet the requirements of Section 2.4 of this specification. Welding shall be executed in conformance with the requirements of Chapter 3 of this specification. Weld tab removal shall be as required in Section 4.2 of this specification.

4.11.4 Sequence of Assembly

Following completion of the replacement groove welds between the existing beam flanges and the column flange, the top flange groove welds shall have the cap pass surface of the weld ground flush with the surface of the beam flange to enable the cover plate to fit flat against the beam flange. At the bottom flange, the backing bar shall be removed, the weld root backgouged to solid weld metal, and the area backwelded until at least flush. The weld at the bottom of the bottom flange shall then be ground flush with the surface of the beam flange to enable the cover plate to fit flat against the beam flange. Care should be used to avoid over-grinding the existing weld below the surface of the beam flange and creating a void between cover plate and flange. The weld shall be tested using UT, using the percentage testing required for Seismic Weld Demand Category BH/T joints, prior to placement of the cover plate. MT of the cap pass of this

weld is not required. After acceptance of the beam flange-to-column flange weld, the cover plate shall be placed and welded. The cover plate to column flange weld shall then be inspected by UT and MT, using the percentage testing required for Seismic Weld Demand Category BH/T joints.

Part I

5. FABRICATION

5.1 Reduced Beam Sections (RBSs)

5.1.1 Holes and Attachments

No holes may be drilled or punched in either flange of the beam within the length that has received the radius cut, or between the RBS cut and the column. Shear studs and mechanical deck fasteners to the beam flange within the length of the radius cut are prohibited. Spot welds for the attachment of metal decking are permitted.

5.1.2 RBS Cut Tolerances

The tolerance on the depth of each RBS cut is plus or minus ¼ inch, measured at the mid-thickness of the flange at the narrowest point of the cut flange. The length of the cut shall be within plus or minus 15% of the specified length. The depth of cut on each side shall be balanced, with no more than 3/8 inch total variation in the depth of cut from one side to the other. (Example: plus 1/8 inch on one side, minus 1/4 inch on other side.) The balance of remaining flange width about the web of the beam is not a consideration.

5.1.3 Cut Surface Roughness

After thermal cutting, the RBS surface shall have a surface roughness of no more than 500 microinches (AWS sample 3). Grinding of thermally cut edges shall be provided only as necessary to meet this criteria. Corners between the cut RBS surface and the top and bottom of the flanges shall be ground to remove sharp edges, but a minimum radius or chamfer is not required.

5.1.4 Gouges and Notches

Gouges and notches that occur in the thermal cut RBS surface may be repaired by grinding if not more than ¼ inch deep. The gouged or notched area shall be faired by grinding so that a smooth transition exists, and the total length of the area ground for the transition shall be no less than 10 times the depth of the removed gouge. If a sharp notch exists, the area shall be inspected by MT after grinding to ensure that the entire depth of gouge or notch has been removed. Grinding may not increase the depth of the RBS cut section more than ¼ inch beyond the specified depth of cut.

5.1.5 Welded Repair of Notches and Gouges

Gouges and notches that exceed ¼ inch in depth, but not to exceed ½ inch in depth, and those notches and gouges where repair by grinding would increase the effective depth of the RBS cut beyond tolerance, may be repaired by welding. Notches and gouges exceeding ½ inch in depth may be repaired only with the approval of the Engineer. The notch or gouge shall be removed and ground to provide a smooth radius of not less than 3/8 inch for welding. The repair area shall be preheated to a temperature between 400°F and 550°F, measured at the location of the weld

repair approximately one minute after removal of the heating source. Repair welding shall be done with notch-tough electrodes meeting the requirements in Part I, Section 2.4. A repair WPS is required. Following welding, the repair weld shall be ground to a smooth contour meeting the RBS requirements, with a surface roughness not to exceed 500 microinches. The welded repair area shall be inspected using magnetic particle testing.

5.2 Heavy Sections

5.2.1 General

See AISC *Specification* sections A3.1c for materials requirements, J2.8 for preheat requirements, J1.6 for access hole requirements, and J1.5 for weld tab and backing bar removal requirements. See AISC *LRFD* Figure C-J1.2 for dimensional and fabrication requirements for standard weld access holes. See Section 4.4 of this specification for further requirements for weld access holes.

5.2.2 Access Hole Requirements

Weld access holes must be preheated to a minimum of 150°F prior to thermal cutting, ground to 500 microinches (bright metal), and inspected for cracks using either penetrant testing (PT) or magnetic particle testing (MT). Optionally, weld access holes may be made by drilling and saw-cutting without grinding, but PT or MT of the cut surface is still required.

5.2.3 Welding

For joint welding, the minimum preheat and interpass temperature shall be 350°F. Weld tabs and backing bars shall be removed after completion of the joint, ground smooth to a maximum surface roughness of 500 microinches, with reinforcement not to exceed 1/8 inch, at a transition slope not to exceed 1:10.

5.3 End Plate Moment Connections

5.3.1 Flange Welds

Beam-flange-to-end-plate welds shall be detailed and welded as CJP groove welds, except that in the area of the flange directly above and below the beam web, backgouging and backwelding of the weld root is not required.

The groove weld detail shall be such that the root of the weld is located on the beam web side of the joint. Following backgouging and backwelding of the groove weld root, a 5/16 inch reinforcing fillet weld shall be placed on the inside portion of the flange weld, at the groove root. Alternatively, the 5/16-inch reinforcing fillet weld located on the beam web side of the beam flanges may serve as a backing weld. The root of the fillet must then be backgouged to solid weld metal, and the groove weld placed. No backing is provided in the vicinity of the beam web. Using either method, no MT of the backgouged area is required prior to placement of the backweld or the groove weld.

5.3.2 Web Welds

The beam web shall be welded full length to the end plate using fillet welds on each side of the beam web. Alternatively, groove welds may be substituted for the fillet welds.

The weld detail limitations of Section 5.6 of this specification regarding the k-area are not applicable to this connection, but MT following completion of all welds in this area is required.

5.3.3 Stiffener Welds

When Bolted Stiffened End Plate (BSEP) connections are used, the stiffener shall be welded to the end plate and the beam flange using CJP groove welds. A stiffener clip (snipe) shall be provided at the intersection of beam flange and end plate.

5.3.4 Weld Access Hole

No weld access holes are permitted at the intersection of beam web, beam flange and end plate.

5.3.5 NDT Requirements

The beam flange to end plate welds, and the welds between stiffener and beam flange and end plate, shall be inspected using MT and UT as QC/QA Category AH/T and between stiffener and beam flange BH/L. Discontinuities located at the root of the beam flange weld, at the intersection of the beam web and beam flange, shall not be cause for rejection.

Welds between the beam web and end plate shall be inspected using MT, for QC/QA Category BM/L, for their full length.

5.4 Bolted Connections

All bolts used in these connections shall be installed as in slip-critical joints with Class A faying surfaces. Bolts may be installed using any of the pretensioning methods prescribed in the RCSC Specification.

Bolts holes shall be standard diameter, unless otherwise noted on the drawings.

For all other bolted connections, the connection type shall be as designated on the drawings.

Commentary: The provisions of this section are applicable to the following types of bolted connections: Bolted Unstiffened End Plate (BUPE) connections, Bolted Stiffened End Plate (BSEP) connections, Bolted Flange Plate (BFP) connections, Double Split Tee (DST) connections.

5.5 Repair of Discontinuities in Main Members

In lieu of AISC Seismic Provisions Section 7.3c, the provisions of this section shall apply to beams in steel moment frames and only to that portion of the beam between the column flange

and the following locations:

- (a) for Welded Unreinforced Flange (WUF), Welded Free Flange (FF), and Improved Welded Unreinforced Flange (IWURF) connections a point away from the column face located a distance equal to one-half the depth of the beam
- (b) for Reduced Beam Section (RBS) connections a point away from the centerline of the cut radius located a distance equal to one-half the depth of the beam, but no closer than the far edge of the radius cut
- (c) for Welded Flange Plate (WFP) connections and Welded Cover Plated Flange (WCPF) connections from the end of the flange plate or cover plate, away from the column face, to a point located away from the end of the plate at a distance equal to one-half the depth of the beam
- (d) for Welded Bottom Haunch (WBH) connections and Welded Top and Bottom Haunch (WTBH) connections from the intersection of the haunch and beam flange, to a point located away from haunch intersection a distance equal to the one-half the depth of the beam
- (e) for Bolted Unstiffened End Plate (BUEP) connections to a point away from the column face located a distance equal to five-sixths the depth of the beam, plus the thickness of the end plate
- (f) for Bolted Stiffened End Plate (BSEP) connections from the end of the stiffener, away from the column face, to a point located away from the end of the stiffener a distance equal to one-half the depth of the beam
- (g) for Double Split Tee (DST) connections from the stem end of the tee to a point located a distance equal to one-half the depth of the beam

Commentary: It is recommended that the design drawings designate these locations and refer to this section of the specification for specific requirements.

5.5.1 Tack Welds

Tack welds are permitted if made prior to beginning welding of the joint. Tack welds for backing bars and weld tabs must be made within the groove weld joint. Tack welds for attachment of parts prior to fillet welding are acceptable, provided they are covered by the completed fillet weld.

Tack welds outside these locations must be removed by grinding or chipping. Air carbon arc gouging and thermal cutting to remove tack welds in these areas is not permitted. Following the removal of unacceptable tack welds, the weld area shall be ground to a depth of 1/16 inch, and faired to adjacent surfaces on a slope not to exceed 1:5.

5.5.2 Erection Aids

Unless requested by the Contractor and approved by the Engineer in advance, the use of welded attachments as erection aids within the designated areas is prohibited. If erection aids are placed within the designated area in error, or cannot be avoided, the Engineer's approval of the aid's placement, use, and the repair method is required.

Air carbon arc gouging is permitted for the removal of welds to within 1/8 inch of the base metal surface. Any remaining weld deposits shall be removed by grinding to a depth 1/16 inch below the surface, faired to adjacent surfaces on a slope not to exceed 1:5.

5.5.3 Air Carbon Arc Cutting and Thermal Cutting

Air carbon arc cutting (CAC-A) and thermal cutting is permitted within the above regions when required for the removal of backing bars and weld tabs, as specified in these documents. The use of these processes for repairs to or removal of base metal or welds in the above region is permitted only with the prior approval of the Engineer.

5.6 K-Area Welding Limitations

After welding of continuity plates and doubler plates, test column webs for cracking using liquid penetrant (PT) or magnetic particle testing (MT) over a zone 3 inches above and below the continuity plate or doubler plate welds. Testing may be performed after the weld has cooled to ambient temperature.

5.7 Surface Finish

5.7.1 Flush Surfaces

Welds in butt joints required to be flush shall be finished so as to not reduce the thicknesses of the thinner base metal or weld metal by more than 1/16 inch, or 5% of the material thickness, whichever is less. Remaining reinforcement shall not exceed 1/32 inch in height. However, all reinforcement shall be removed where the weld forms part of a faying or contact surface. All reinforcement shall blend smoothly into the plate surfaces with the transition areas free from undercut.

5.7.2 Finish Methods and Values

Chipping and gouging may be used, provided these methods are followed by grinding. Where surface finishing is required, surface roughness values shall not exceed 500 microinches, unless otherwise noted or specified in this document. Regardless of the surface finish required, the direction of grinding marks may be in any direction.

Measurement of surface finish values by visual appearance or tactile comparison is acceptable.

5.8 Weld Acceptance Criteria

5.8.1 Engineer's Authority

Welds or portions of welds that fail to meet the acceptance criteria of AWS D1.1 shall be repaired or replaced. The Contractor may request acceptance by the Engineer of a weld discontinuity, without repair or replacement, when it can be determined that the effect of the discontinuity will not be detrimental to the performance of the structure. The Engineer is the final authority for acceptance of such welds.

5.8.2 Magnetic Particle Testing

If a surface discontinuity or near-surface discontinuity, within 1/8 inch of the surface, is detected, the discontinuity shall be rejected and removed. If the separation from the surface cannot be determined, the discontinuity shall be categorized as a surface flaw, rejected and removed.

Regions of welds that cannot be inspected shall be identified and recorded, and the Engineer shall be notified.

5.8.3 Ultrasonic Testing - Flaw Detection

When ultrasonic testing is required, the joint shall be scanned for flaw detection purposes following the procedures prescribed in AWS D1.1, Annex K, with exceptions as noted below. Joints that fail the acceptance criteria described below may be inspected using the Ultrasonic Testing - Flaw Sizing methods as prescribed in Section 5.8.4 of this specification, or, at the Contractor's option, may be excavated for further investigation and repaired, then reinspected using these Flaw Detection procedures.

When ultrasonic testing is required, CJP and PJP groove welds in Seismic Weld Demand Categories A, B and C shall be scanned for flaw detection. Acceptance criteria shall be as for statically loaded welds in Annex K, Table K-1, of AWS D1.1.

Commentary: As an alternative, The engineer may elect to permit CJP and PJP groove welds in Seismic Weld Demand Categories A, B and C to be scanned for flaw detection purposes using AWS D1.1, Table 6.2 as the acceptance criteria. In this case, Note 3 regarding "tension welds" is not applicable.

Joints with backing bars remaining in place shall not be rejected on the basis of indication ratings (db values) from the interfaces between backing bar and base metal or backing bar and weld. The UT Procedure shall prescribe methods for distinguishing between backing bar indications and root discontinuities.

PJP groove weld joints shall not be rejected on the basis of indication ratings (db values) from the root area of the weld. Notches within the weld, located a distance more than 1/8 inch from the as-welded root, shall be scanned for acceptance using the criteria above.

Regions of welds that cannot be inspected shall be identified and recorded, and the Engineer shall be notified.

Regions of welds adjacent to cope holes may be inspected with multiple probe techniques.

5.8.4 Ultrasonic Testing – Flaw Sizing

Ultrasonic testing for flaw sizing shall be performed following written procedures as required by AWS D1.1, Annex K. When flaw-sizing techniques are implemented, the following acceptance criteria applies to groove welds:

1. If a surface flaw or near-surface flaw (within 1/8 inch of the surface) is detected, the flaw shall be rejected and removed. If the separation from the surface cannot be measured, the flaw shall be categorized as a surface flaw, rejected and removed.
2. When backing bars remain in place, the position of notch tips that extend into the weld metal shall be determined. The notch shall be rejected if it extends greater than 1/8-inch into the thickness of the weld. The weld present between the backing bar and column face shall not be considered a part of the weld thickness in determining the depth of notch or thickness of weld.
3. Embedded flaws, defined as those that do not come within 1/8 inch of the surface, shall be rejected if their height exceeds ¼ inch.
4. Embedded flaws shall be rejected if their area, as calculated by multiplying the maximum discontinuity height by the maximum discontinuity length, exceeds the thickness of the thinner parent metal multiplied by the thickness of the thicker parent metal.
5. Embedded flaws, either individually or as a group within a length of weld 12 inches or less, shall be rejected if they exceed a total area (the sum of the areas of individual discontinuities) equal to 10% of the thickness of the thinner parent metal multiplied by the weld length. The weld length used for this calculation shall not exceed 12 inches, with longer welds being evaluated in multiple parts.
6. Aligned discontinuities of lengths L1 and L2 separated by less than $(L1+L2)/2$ shall be evaluated as continuous.
7. Parallel discontinuities of heights H1 and H2 separated by less than $(H1+H2)/2$ shall be evaluated as continuous.

Part I

6. Quality Control and Quality Assurance

6.1 Specifications

The specifications and standards listed in this Section govern the execution of Quality Control and Quality Assurance, including nondestructive testing, except as noted otherwise in this document.

6.1.1 Codes

Applicable local building code, including standards referenced by the code

6.1.2 Quality Assurance Agency Practices

- ASTM E329 - Recommended Practice for Inspection and Testing Agencies for Concrete, Steel and Bituminous Materials as used in Construction*
- ASTM E543 - Standard Practice for Agencies Performing Nondestructive Testing*
- ASTM E1212 - Standard Practice for Establishment and Maintenance of Quality Control Systems for Nondestructive Testing Agencies*

6.1.3 Magnetic Particle Testing (MT)

- ASTM E709 - Standard Guide for Magnetic Particle Examination*
- ASTM E1444 - Standard Practice for Magnetic Particle Examination*

6.1.4 Ultrasonic Testing (UT)

- ASTM E587 - Standard Practice for Ultrasonic Angle-Beam Examination by the Contact Method*
- ASTM E114 - Standard Practice for Ultrasonic Pulse-Echo Straight-Beam Examination by the Contact Method*
- ASTM E164 - Standard Practice for Contact Examination of Weldments*
- ASTM A898 - Standard Specification for Straight Beam Ultrasonic Examination of Rolled Steel Structural Shapes*
- ASTM A435 - Standard Specification for Straight-Beam Ultrasonic Examination of Steel Plates*

6.1.5 Nondestructive Testing Personnel Qualification

- (a) ANSI/ASNT CP-189-1995, ASNT Standard for Qualification and Certification of Nondestructive Testing Personnel;

or

- (b) ASNT Recommended Practice No. SNT-TC-1A, *Personnel Qualification and Certification in Nondestructive Testing*, 1995

6.2 Submittals

The Quality Assurance Agency shall submit the following items:

1. Qualifications of QA Agency's management and QA personnel designated for the project,
2. QA Agency's Written Practice, as defined in Section 6.2.1,
3. Qualification records for Inspector and NDT technicians designated for the project,
4. QA Agency's NDT procedures, equipment calibration records, and personnel training records, and
5. QA Agency's Quality Control Plan for the monitoring and control of the Agency's operations.

6.2.1 Written Practice for Quality Assurance Agencies

The Quality Assurance Agency shall maintain a Written Practice for the selection and administration of inspection personnel, describing the training, experience and examination requirements for qualification and certification of inspection personnel. The Written Practice shall also describe the Agency's procedures for determining the acceptability of the structure in accordance with the applicable codes, standards, and specifications. The Written Practice shall also describe the Agency's inspection procedures, including general inspection, material controls, visual welding inspection, and bolting inspection.

Commentary: The Quality Assurance Written Practice submittals should be furnished to the fabricator and erector, to provide the parties performing the work with an understanding of the quality assurance measures that will be employed.

6.2.1.1 Bolting Inspection Procedures

Bolting inspection procedures shall meet the requirements of the RCSC *Specification* and the Quality Assurance Plan. Inspection procedures shall be written specifically for the installation and pretensioning methods to be used, i.e. turn-of-nut method, calibrated wrench method, twist-off bolt method, or direct tension indicator method. Written procedures shall also be prepared for the arbitration of disputes, to be used should a dispute arise regarding the pretension of previously installed fasteners.

6.2.1.2 Welding Inspection Procedures

Welding inspection procedures shall meet the requirements of the AWS *D1.1* and the Quality Assurance Plan.

6.2.1.3 Shear Connector (Stud) Inspection Procedures

Shear connector inspection procedures shall meet the requirements of the AWS *D1.1* and the Quality Assurance Plan.

6.2.2 Written Practice for Nondestructive Testing Agencies

The NDT Agency shall maintain a Written Practice for the control and administration of NDT personnel training, examination and certification, describing the training, experience and examination requirements for each level of certification. The Written Practice shall describe the responsibility of each level of certification for determining the acceptability of material and weldments in accordance with the applicable codes, standards, specifications and procedures.

6.3 Inspector Qualifications

6.3.1 Special Inspector Qualifications

Special Inspectors shall be trained and competent, to the satisfaction of the party responsible for the Quality Assurance Plan, to provide the assigned special inspection tasks.

6.3.2 Welding Inspector Qualifications

All Welding Inspectors shall have adequate visual acuity, documented by vision testing performed within the past three years, in accordance with AWS *D1.1*, Section 6.1.4.4.

All Welding Inspectors shall be trained and thoroughly experienced in inspecting welding operations, and qualified in accordance with AWS *D1.1*, Section 6.1.4.

For welds in Seismic Weld Demand Categories A and B, welding inspectors shall be AWS Certified Welding Inspectors (CWI), or Senior Certified Welding Inspectors (SCWI), as defined in AWS QC1, *Standard and Guide for Qualification and Certification of Welding Inspectors*, latest edition.

For welds in Seismic Weld Demand Category C, welding inspection personnel shall be AWS Certified Associate Welding Inspectors (CAWI) or higher, or otherwise qualified under the provisions of AWS *D1.1-98*, Section 6.1.4, to the satisfaction of the party responsible for the Quality Assurance Plan.

The use of assistants in inspection operations is permitted, provided the assistants are adequately trained in their responsibilities, and are under the close supervision of the inspector responsible for the joint or connection being inspected.

The qualification of an inspector previously certified as a CWI is acceptable, although the certification may have expired, provided the inspector has remained active in the inspection of welded steel fabrication.

6.3.3 NDT Personnel Qualifications

Nondestructive testing personnel shall be qualified under either of the American Society for Nondestructive Testing, Inc. (ASNT) documents referenced in Section 6.1.5.

NDT may be performed by NDT Level I personnel only under the close, direct supervision of an NDT Level II.

For welds in Seismic Weld Demand Categories A and B, UT may be performed only by UT technicians certified as Level II by their employer, or as ASNT Level III certified by examination by the ASNT, and have passed the Supplementary UT Inspector examination as described in Section 6.3.3.1.

6.3.3.1 Supplementary NDT Personnel Qualification Testing

Ultrasonic testing technicians who perform flaw detection or sizing shall be trained in applicable UT procedure and shall demonstrate their competence through testing as prescribed in Appendix E.

6.3.4 Bolting Inspector Qualifications

Bolting inspectors shall be trained and qualified to inspect bolting operations and high-strength bolted connections for compliance with the RCSC *Specification* and the Quality Assurance Plan. Competency shall be demonstrated through the administration of a written examination and through the hands-on demonstration by the Inspector of the methods to be used for bolt installation and inspection.

6.4 Contractor Tasks

The Contractor shall provide the management, personnel, equipment and services necessary to perform the Quality Control functions required in Sections 6.6, 6.7, 6.8, and 6.9.

6.5 Quality Assurance Agency Tasks

The Quality Assurance Agency shall provide the management, personnel, equipment and services necessary to perform the Quality Assurance functions required in Sections 6.6, 6.7, 6.8, and 6.9.

6.6 Welding Inspection

The welder, QC Welding Inspector, and QA Welding Inspector, as appropriate, shall perform the QC and QA tasks indicated in the following list, as amplified by Tables 6-1 and 6-2. Table 6-1 assigns a Process and Visual Inspection Welding Category for each welded joint, depending on the Seismic Weld Demand and Consequence Categories for the joint indicated on the design drawings. Table 6-2 indicates the specific actions required of the welder, QC Welding Inspector and QA Welding Inspector for joints of each Process and Visual Inspection Welding Category.

This list shall not be considered exclusive of any additional inspection tasks that may be necessary to meet the requirements of the codes or the Quality Assurance Plan.

1. Review and understand the applicable portions of the specifications, the Contract Documents and the shop drawings for the project.
2. Verify that all applicable welder qualifications, welding operator qualifications and tack welder qualifications are available, current and accurate.
3. Require requalification of any welder, welding operator or tack welder who has, for a period of six months, not used the process for which the person was qualified.
4. Verify welder identification and qualification. Verify that any required supplemental welder qualification testing, if required for the joint, has been executed and that the welder has passed.
5. Verify that each welder has a unique identification mark or die stamp to identify welds.
6. Verify that all applicable Welding Procedure Specifications (WPSs), with Procedure Qualification Records (PQRs) as needed, are available, current and accurate.
7. Verify that an approved Welding Procedure Specification (WPS) has been provided and that each welder performing the weld has reviewed the WPS. A copy of the appropriate WPSs shall be available for each joint, although need not be present at each joint location.
8. Review mill test reports for all main member and designated connection base material for compliance with the project requirements.
9. Verify base material identification with the approved shop drawings and specifications.
10. Verify the electrode, flux and shielding gas certifications for compliance with the Contract Documents.
11. Verify welding consumables with the approved shop drawings and approved WPSs.
12. Verify that electrodes are used only in the permitted positions and within the welding parameters specified in the WPS.
13. Verify that electrodes and fluxes are properly stored, and that exposure limits for the welding materials are satisfied.
14. At suitable intervals, observe joint preparation, assembly practice, preheat temperatures, interpass temperatures, welding techniques, welder performance and any postweld controlled cooling or heat treatment to ensure that the applicable requirements of the WPS and Code are satisfied.
15. At suitable intervals, verify proper current and voltage of the welding equipment in application of the WPS, if needed, by using a hand held calibrated amp and volt meter. Current and voltage shall be measured near the arc with this equipment.

16. Inspect the work to ensure compliance with AWS D1.1 or the specified weld acceptance criteria. Size and contour of welds shall be measured with suitable gauges. A strong light, magnifiers, or other devices as needed may be used to aid visual inspection.
17. Schedule NDT technicians in a timely manner, after the visual inspection is complete and the assembly has cooled. The final NDT on a specific weld shall not be performed sooner than 24 hours after the welding has been completed. See Section 6.7.3.
18. Mark the welds, parts, and joints that have been inspected, and accepted, with a distinguishing mark or die stamp, or maintain records indicating the specific welds inspected by each inspector.
19. Document the accepted and rejected items in a written report. Transmit the report to the designated recipients in a timely manner.

Commentary: The Engineer must indicate the appropriate Seismic Weld Demand Category and Seismic Weld Consequence Category for each welded joint, on the design drawings. FEMA 350 and FEMA 351 specify the appropriate categories for welded joints in prequalified moment resisting connections. Part II of this document provides guidelines for assigning Seismic Weld Demand and Seismic Weld Consequence categories.

Table 6-1 Process and Visual Welding Inspection Categories

Seismic Weld Consequence Category	Seismic Weld Demand Category		
	A	B	C
H	1	1	2
M	1	2	3
L	2	3	3

6.7 Nondestructive Testing of Welded Joints

6.7.1 Magnetic Particle Testing

For welds classified into Seismic Weld Demand Categories in the design documents, Magnetic particle testing (MT) shall be conducted by the Quality Assurance Agency at the frequency designated in Table 6-3. MT shall be performed in accordance with the requirements of AWS D1.1, the ASTM standards referenced in Section 6.1.3 of this specification, and the provisions of Appendix F.

6.7.2 Ultrasonic Testing

For welds classified into Seismic Weld Demand Categories in the design documents, Ultrasonic testing (UT) shall be conducted by the Quality Assurance Agency for the percentage of joints designated in Table 6-3. UT shall be performed in accordance with the requirements of AWS D1.1, and the ASTM standards referenced in Section 6.1 of this specification.

Table 6-2 Process and Visual Welding Inspection Tasks

Process and Visual Welding Inspection Category:		1				2				3			
Inspection Tasks	Welder	Inspector											
		QC		QA		QC		QA		QC		QA	
		H	O	H	O	H	O	H	O	H	O	H	O
Inspection Prior to Welding													
Proper WPS selected for joint detail	✓	✓		✓		✓			✓		✓		✓
Proper welding materials selected	✓	✓		✓		✓			✓		✓		✓
WPS settings (voltage, polarity, current, wire feed speed) on welding equipment verified	✓	✓			✓✓		✓		✓		✓		✓
Shielding gas type (if used) verified	✓	✓			✓		✓		✓		✓		✓
Shielding gas flow rate setting verified	✓	✓			✓		✓		✓		✓		✓
Fit of backing bar (if used) acceptable	✓	✓			✓		✓		✓		✓		✓
Measure root opening	✓		✓		✓		✓		✓		✓		✓
Measure groove angle	✓		✓		✓		✓		✓		✓		✓
Verify above dimensions within joint tolerance and WPS tolerance	✓	✓			✓		✓		✓		✓		✓
Mark root edge location on beam flange for UT inspection (if required)	✓	✓			✓		✓		✓		✓		✓
Condition of steel surface acceptable	✓	✓			✓		✓		✓		✓		✓
Existing tack welds clean and of adequate quality	✓	✓			✓		✓		✓		✓		✓
Wind velocity within limits	✓		✓		✓		✓		✓		✓		✓
Weld joint surfaces free of discontinuities	✓		✓		✓		✓		✓		✓		✓
Minimum preheat required applied and verified	✓		✓✓		✓✓		✓		✓		✓		✓
Maximum preheat verified	✓		✓✓		✓✓		✓		✓		✓		✓
Observation of welder's inspection	✓		✓✓		✓		✓		✓		✓		✓
Observation of QC inspection					✓✓		✓		✓				✓

See Legend and Notes at the end of the table.

Table 6-2 Process and Visual Welding Inspection Tasks (Continued)

Process and Visual Welding Inspection Category:		1				2				3			
Inspection Tasks	Welder	Inspector											
		QC		QA		QC		QA		QC		QA	
		H	O	H	O	H	O	H	O	H	O	H	O
Inspection During Welding													
WPS followed (voltage, current, wire feed speed, travel speed, stickout, gas flow rate, pass location)	✓		✓✓		✓✓		✓✓		✓		✓		✓
Welding materials exposure control maintained	✓		✓✓		✓✓		✓✓		✓		✓		✓
Minimum interpass temperature maintained and verified	✓		✓✓		✓✓		✓✓		✓		✓		✓
Maximum interpass temperature verified	✓		✓✓		✓✓		✓✓		✓		✓		✓
No detrimental change in environmental conditions	✓		✓		✓		✓		✓		✓		✓
Tack welds do not crack during welding	✓		✓		✓		✓		✓		✓		✓
Each pass cleaned	✓		✓		✓		✓		✓		✓		✓
Each pass within profile limitations	✓		✓		✓		✓		✓		✓		✓
Each pass meets quality requirements	✓		✓		✓		✓		✓		✓		✓
Proper technique used (electrode angle, stringer beads)	✓		✓		✓		✓		✓		✓		✓
NDT in process, when required	✓	✓		✓		✓		✓			✓		✓
Observation of welder’s inspection			✓		✓		✓		✓		✓		✓
Observation of QC process					✓				✓				✓

See Legend and Notes at the end of the table.

Table 6-2 Process and Visual Welding Inspection Tasks (Continued)

Process and Visual Welding Inspection Category:		1				2				3			
Inspection Tasks	Welder	Inspector											
		QC		QA		QC		QA		QC		QA	
		H	O	H	O	H	O	H	O	H	O	H	O
Inspection after Welding													
Welder identification legible	✓	✓		✓		✓		✓		✓			✓
Inspection delay period satisfied		✓		✓		✓		✓		✓		✓	
Weld cleaned	✓	✓		✓		✓		✓		✓		✓	
Weld size and length verified	✓	✓			✓	✓			✓	✓			✓
Weld within profile limitations	✓	✓			✓	✓			✓	✓			✓
Weld appearance indicates thorough fusion	✓	✓		✓		✓		✓		✓			✓
Weld craters acceptable	✓	✓			✓	✓			✓	✓			✓
Undercut within limitations	✓	✓		✓		✓		✓		✓			✓
Porosity within limitations	✓	✓			✓	✓			✓	✓			✓
Weld free of cracks	✓	✓		✓		✓		✓		✓			✓
Backing bars removed (if required)	✓	✓		✓		✓		✓		✓			✓
Weld tabs removed (if required)	✓	✓		✓		✓		✓		✓			✓
Surface finish (grind, contour) as required	✓	✓		✓		✓		✓		✓			✓
Observation of welder’s inspection			✓		✓	✓	✓		✓		✓		✓
Observation of QC process					✓				✓				✓
NDT completed		✓		✓		✓		✓		✓		✓	

See Legend and Notes at the end of the table.

Table 6-2 Process and Visual Welding Inspection Tasks (Continued)

Legend:	
✓	The welder or Inspector, as noted, should perform this function, with measurements as necessary.
✓✓	The Inspector should observe these operations more frequently.
H	Hold – The welder shall not proceed with welding until inspection of this item is performed by the designated Inspector. After this inspection, the welder may proceed with welding until completion, with observation of welding functions on a random, periodic basis by the Inspector. For inspection after welding, this inspection must be performed prior to final acceptance of the item.
O	Observe – The welder may proceed with welding after completing his / her own inspection. The Inspector shall perform random, daily, periodic inspection and observation of these welding functions.
Notes:	
1.	The precise level and frequency of observation by either QC or QA is not specified. Inspection frequency shall be adequate to provide reasonable confidence in the control of the welding process and the quality of the completed welds. Consideration shall be made of the consistency achieved in satisfying the required welding parameters, and the effect of the welding parameter upon weld quality and performance.
2.	As a minimum, the observation inspection tasks listed shall be performed on a daily basis.
3.	Hold point inspections for WPS selection and welding materials need be performed only when changed by the welder.
4.	This list shall not be considered exclusive of any additional inspection tasks that may be necessary to meet the requirements of the codes or the Quality Assurance Plan.

6.7.3 NDT Delay Periods

Final visual inspection may take place immediately upon cooling to ambient temperature. Final nondestructive testing, either MT or UT, may not begin until 24 hours after the completion of welding. If delayed cooling such as insulating blankets or PWHT has been used, the 24-hour delay period shall begin after the steel has reached ambient temperature.

At the Contractor's option, Contractor Quality Control NDT may be performed before the delay period has expired, but shall not be used for final acceptance. In-process MT, such as for verifying the removal of cracks and other discontinuities when backgouging or repairing thermal cut surfaces, may be performed immediately upon completion of the welding or backgouging. No cooling period is necessary. Final MT and UT of the joint or repair shall not be performed until the 24-hour delay period is met. Final MT of weld tabs may be performed immediately upon completion.

6.7.4 Nondestructive Testing Requirements

The NDT technician shall perform all NDT, other than visual, required by the Quality Assurance Plan, Contract Documents and Building Code. NDT shall be performed in a timely manner, so as not to hinder production, and to detect welding problems soon after occurrence so that corrective measures may be taken by the Contractor to rectify such problems.

The NDT technician shall mark the welds, parts, or joints that have been inspected and accepted with a distinguishing mark or die stamp, or maintain records indicating the specific welds inspected.

The accepted and rejected items shall be documented in a written report. The report shall be transmitted to the designated recipients in a timely manner.

6.7.4.1 Heavy Sections

Heavy Section column flanges located at welded moment-resisting connections shall be ultrasonically examined, prior to welding, for evidence of laminations, inclusions or other discontinuities in accordance with ASTM A435, *Straight Beam Ultrasonic Examination of Steel Plates*, or ASTM A898, *Straight Beam Ultrasonic Examination of Rolled Steel Structural Shapes*, as applicable. The area to be tested is a zone 3 inches above and below each beam flange connection. For plates, any discontinuity causing a total loss of back reflection that cannot be contained within a circle the diameter of which is 3 inches, or one-half the plate thickness, whichever is greater, shall be rejected. For shapes, ASTM 898 Level I criteria is applicable.

If moment connections are made to the weak axis of a wide flange column, the column web shall be similarly examined to the above criteria.

6.7.4.2 Lamellar Tearing

After joint completion, base metal thicker than 1-1/2 inches, in Seismic Force Resisting System joints where subject to through-thickness shrinkage strains from welding, and where UT is required by Table 6-3, shall be ultrasonically tested for discontinuities behind and adjacent to such welds. Any lamellar tearing discontinuities shall be accepted or rejected on the basis of criteria established by the Engineer or designated registered design professional in responsible charge of the work for that specific joint.

6.7.4.3 Column Splices

CJP and PJP groove welded column splices that are a part of the Seismic-Force-Resisting System, and are subject to applied tension under lateral loading, shall be ultrasonically tested as a QA Category BH/T weld. Those splices not subjected to applied tension from lateral loading, need not be ultrasonically tested.

Commentary: Since the Contractor can not determine which column splices may be subject to applied tension under lateral loading, the Engineer should indicate these splices on the design drawings. The determination as to whether a splice is under tension should be made considering load combinations with seismic loads amplified by the overstrength factor, W_o , in accordance with FEMA-302.

Table 6-3 Nondestructive Testing Requirements

		Seismic Weld Demand Category		
		A	B	C
Seismic Weld Consequence Category	H	CJP MT 100% of joints, full length UT 100% of joints, full length PJP, fillets MT 100% of joints, full length	CJP MT 100% of joints, full length if transversely loaded, partial length if longitudinally loaded UT 100% of joints, full length if transversely loaded, partial length if longitudinally loaded (Reduce UT to 25% of joints, of length as above, with low reject rate) PJP, fillets MT 25% of joints, full length if transversely loaded, partial length if longitudinally loaded	CJP UT 10% of joints, full length if transversely loaded, partial length if longitudinally loaded PJP, fillets MT 10% of joints, 6" spot at random
	M	CJP MT 100% of joints, full length UT 100% of joints, full length (Reduce UT to 25% of joints, full length, with high acceptance rate) PJP, fillets MT 100% of joints, full length	CJP MT 100% of joints, full length if transversely loaded, partial length if longitudinally loaded UT 100% of joints, full length if transversely loaded, partial length if longitudinally loaded (Reduce UT to 25% of joints, of length as above, with low reject rate) PJP, fillets MT 25% of joints, full length if transversely loaded, partial length if longitudinally loaded	No NDT required
	L	CJP MT 25% of joints, full length UT 25% of joints, full length PJP, fillets MT 10% of joints, 6" spot at random	CJP UT 10% of joints, full length PJP, fillets MT 10% of joints, 6" spot at random	No NDT required

Notes:

- UT is required only when the weld throat is 5/16" or greater.
- Reduce the rate of UT, where noted, if after 40 welds have been inspected, an individual welder's reject rate is less than 5%.
- Partial length testing is applicable for longitudinally loaded welds when over 24 inches in length. Inspect the beginning and end of each weld for a 6 inch length, plus any location along the length of the weld where a start and restart is visually noted for a distance of 6 inches on either side of the stop/start location, and a 6 inch length for every 10 feet for a given weld.
- For column splices, see Section 6.7.4.3.

6.7.4.4 Column Webs at Continuity Plates

After welding continuity plates, test column webs for cracking using liquid penetrant testing (PT) or magnetic particle testing methods over a 3 inch minimum zone above and below continuity plates.

6.7.4.5 Column Webs at Doubler Plates

Doubler plates that are welded to the column at the intersection of the column web and flange, either on the radius or in the column “k-area”, shall have the weld termination areas and adjacent column web inspected using magnetic particle testing or liquid penetrant testing (PT).

Column doubler plates that are welded only to the column flange need not have the column web inspected using nondestructive testing.

6.7.4.6 Weld Access Holes

Weld access holes shall be inspected using magnetic particle testing or liquid penetrant testing (PT) for base metal cracks and cracks from thermal cutting if the member is a part of the Seismic Force Resisting System framing, regardless of member size, or designated as a Heavy Section.

If a welded repair has been performed, magnetic particle testing shall be performed in the area immediately adjacent to the welded repair area.

6.7.4.7 Reduced Beam Section Repairs

If repairs to the Reduced Beam Section cut surface are performed by welding, the repair weld and immediately adjacent area shall be inspected using magnetic particle testing.

Commentary: This section only applies to moment-resisting connections employing prequalified Reduced Beam Section details. Refer to FEMA-350 for prequalification details.

6.8 Bolting Inspection

The Bolting Inspector, whether designated QC or QA, shall complete the following items:

1. Review and understand the applicable portions of the specifications, contract drawings, shop detail drawings and erection plans for the project.
2. Review all manufacturer certifications for material compliance with the project requirements.
3. Verify bolting material identification.
4. Verify suitable, controlled storage conditions.
5. Verify that all applicable bolt installation procedures are available, current and accurate.

6. Verify that the appropriate bolt installation procedure has been provided and reviewed by each bolting crew member performing the work.
7. Observe the pre-installation testing performed at the start of the work for each assembly lot.
8. For the calibrated wrench method of installation, observe calibration of the wrench(s) at the start of each work shift.
9. Verify the suitability of the bolted joints, such as bolt hole size and condition, prior to assembly. Check for unfair reaming or slotting of poorly aligned holes.
10. For slip-critical joints, verify that the required faying surface conditions are met.
11. Prior to pretensioning, verify that all bolts have been installed and the joint brought to the snug tight condition.
12. For pretensioned joints, routinely observe, at suitable intervals, the pretensioning operations to verify the proper application of the bolting procedures.
13. Arbitrate any disputes regarding achieved bolt pretension immediately upon installation of the bolts in dispute.
14. Complete a written report recording the joints observed, inspected, and accepted. The report shall be transmitted to the designated recipients in a timely manner.

6.9 Shear Stud Welding Inspection

Periodically, the stud welding shall be inspected and tested in accordance with *AWS D1.1* Section 7, including the following:

1. Witness or review stud base qualification testing, as needed.
2. Witness welding operator qualification testing.
3. Witness or review pre-production testing and qualification.
4. Periodically, inspect welding and perform verification inspection and testing.

6.10 Special Inspection

The Special Inspector shall perform the following inspection tasks, if so designated in the Quality Assurance Plan.

1. Ascertain that all material complies with the Contract Documents, either by material and product certifications and/or by testing.
2. Evaluate the Contractor's QC program and its effectiveness.
3. Verify that the fabricator is properly implementing the fabrication procedures and quality control procedures.
4. Verify the qualifications of the QC and QA Inspectors and the NDT technicians.

5. Inspect the steel frame to verify compliance with the details shown on the approved construction documents, such as bracing, stiffening, member locations and proper application of joint details at each connection.
6. Verify that all fabrication and erection, including welding and bolting, is performed in accordance with the Contract Documents.
7. Keep written records of the inspections performed.
8. Distribute copies of written records to the Owner, Engineer, Contractor and/or Building Official, as specified in the Quality Assurance Plan. Records documenting correction of nonconformance shall be distributed to the same organizations as those receiving reports of nonconformance.
9. Upon completion of the Inspector's tasks, issue a written report stating that all work inspected under the Inspector's charge meets the applicable codes and specifications, and that nonconformances have been corrected or resolved to the satisfaction of the Engineer or other persons in responsible charge.

Part I

APPENDIX A. WELD METAL / WELDING PROCEDURE SPECIFICATION TOUGHNESS VERIFICATION TEST

Scope

This appendix supplements Part I, Section 2.4.1, and provides a standard method for qualification testing of weld filler metals required to have specified notch toughness for service in steel moment frames for seismic applications. The WPS Toughness Verification Test may be performed by the filler metal manufacturer or by the Contractor.

Testing and certification of each lot to be used in production shall be performed on each filler metal manufacturer's production lot, as defined in AWS A5.01, *Filler Metal Procurement Guidelines*, as follows:

1. Class C3 for SMAW electrodes,
2. Class S2 for GMAW-S and SAW electrodes,
3. Class T2 for FCAW and GMAW-C, or
4. Class F2 for SAW fluxes.

Alternatively, filler metal manufacturers approved for production of products meeting the above requirements, under a program acceptable to the Engineer, need not conduct the mechanical A5 tests or the Weld Metal / WPS Toughness Verification Test for each lot, and may rely upon the Manufacturer's certifications that the product meets the specified performance requirements.

Test Conditions

Tests shall be conducted at the range of heat inputs for which the weld filler metal will be qualified under the WPS. It is recommended that tests be conducted at the Low Heat Input Level and High Heat Input Level indicated in Table A-1. Alternatively, the filler metal manufacturer or Contractor may elect to test a wider or narrower range of heat inputs and interpass temperatures. The range of heat inputs and interpass temperatures tested shall be clearly stated on the test reports and user data sheets. Regardless of the method of selecting test heat input, the WPS, as used by the Contractor, must fall within the range of heat inputs and interpass temperatures tested.

**Table A-1 WPS Toughness Verification Test
Welding and Preheat Conditions**

Cooling Rate	Heat Input	Preheat °F	Interpass °F
Low Heat Input Test	30 kJ/in.	70 +/- 25	200 +/- 50
High Heat Input Test	80 kJ/in.	300 +/- 25	500 +/- 50

Test Specimens

Two test plates shall be used for each heat input level and five CVN test specimens shall be made per test plate. Each plate shall be steel, of any AISC-listed structural grade. The test plate shall be 3/4-inch thick with a 1/2-inch root opening and 45° included groove angle. The test plate and specimens shall be as shown in Figure 2A in AWS A5.20-95, or as in Figure 5 in AWS A5.29-89R. A minimum of two passes per layer shall be used to fill the width.

All test specimens shall be taken from near the centerline of the weld at the mid-thickness location, in order to minimize dilution effects. CVN specimens shall be prepared in accordance with AWS B4.0-92, *Standard Methods for Mechanical Testing of Welds*, Section A3. The test assembly shall be welded in the flat position and shall be restrained during welding, or preset at approximately 5 degrees to prevent warpage in excess of 5 degrees. A welded test assembly that has warped more than 5 degrees shall be discarded. Welded test assemblies shall not be straightened.

The test assembly shall be tack welded and heated to the specified preheat temperature, measured by temperature indicating crayons or surface temperature thermometers one inch from the center of the groove at the location shown in the figures cited above. Welding shall continue until the assembly has reached the interpass temperature prescribed in Table A-1. The interpass temperature shall be maintained for the remainder of the weld. Should it be necessary to interrupt welding, the assembly shall be allowed to cool in air. The assembly shall then be heated to the prescribed interpass temperature before welding is resumed.

No thermal treatment of weldment or test specimens is permitted, except that machined tensile test specimens may be aged at 200°F to 220°F for up to 48 hours, then cooled to room temperature before testing.

Acceptance Criteria

All test samples shall meet the strength requirements for the electrodes as provided in Part I, Section 2.4.1.1. The lowest and highest values obtained from each of five specimens from a single test plate shall be disregarded. Two of the remaining three values shall equal, or exceed, the specified toughness of 40 ft-lbf energy level at the testing temperature. One of the three may be lower, but not lower than 30 ft-lbf, and the average of the three shall not be less than the required 40 ft-lbf energy level.

Part I

APPENDIX B. SUPPLEMENTAL WELDER QUALIFICATION TESTING PROCEDURE

Scope

This appendix supplements Part I, Section 3.3.1 and provides for qualification testing of welders required to perform critical welds in steel moment frames designed for seismic applications.

Test Plate

The test plate thickness shall be 1-1/2 inch. The web plate thickness shall be 1 inch. The access hole and groove angle shall be as indicated in Figure B-1. The groove angle shall be 30°, with a root opening of 3/8 inch. Steel backing shall be 3/8 inch thick and at least 1 inch wide. Weld tabs 3/8 inch thick and 1-1/2 inches long shall be attached at each end of the joint. The column plate, representing the column flange and tack welded to the test plate, shall be a minimum 3/8 inch thick, and shall be separated from the test plate by 1/4 inch. The length of test weld shall be 12 inches minimum, not including weld tabs. See Figure B-1.

Welding Operation

The welder shall fit and tack weld the test assembly from prefabricated parts, and shall perform all the groove welding. Backgouging, cutting, tack weld removal, and grinding operations listed in this paragraph may be performed by another welder. Following welding, the backing bar shall be removed by air carbon arc cutting, the joint root backgouged to solid weld metal, and the root backwelded until flush. The weld tabs shall be removed by thermal cutting or air carbon arc cutting, then ground smooth and flush with the test plate. The vertical restriction plate and web shall then be removed, tack welds ground to a depth 1/8 inch below the surface, and the weld surface ground smooth. Should another procedure be used on the project for the removal of the backing bar or weld tabs, then that procedure shall be used for this testing.

Testing

Either radiographic testing (RT) in accordance with AWS D1.1, Section 4.30.3 or side-bend tests in accordance with this section may be performed. When side-bend tests are to be performed, the flat test plate shall be cut to prepare three transverse side-bend tests, with each test specimen 3/4 inch thick. Two side-bend specimens shall be from near the middle of the joint at the web, and one side-bend specimen shall be from near either end of the joint. Testing shall be in accordance with AWS B4.0-92, *Standard Methods for Mechanical testing of Welds*, Section A1.

Alternatively, radiographic testing may be performed for the full length of the weld in accordance with *AWS D1.1*, Section 4.30.3.

Acceptance

All three test specimens must pass the transverse side-bend tests. If radiographic testing is used, the specimen must satisfy *AWS D1.1*, Section 4.30.3.1.

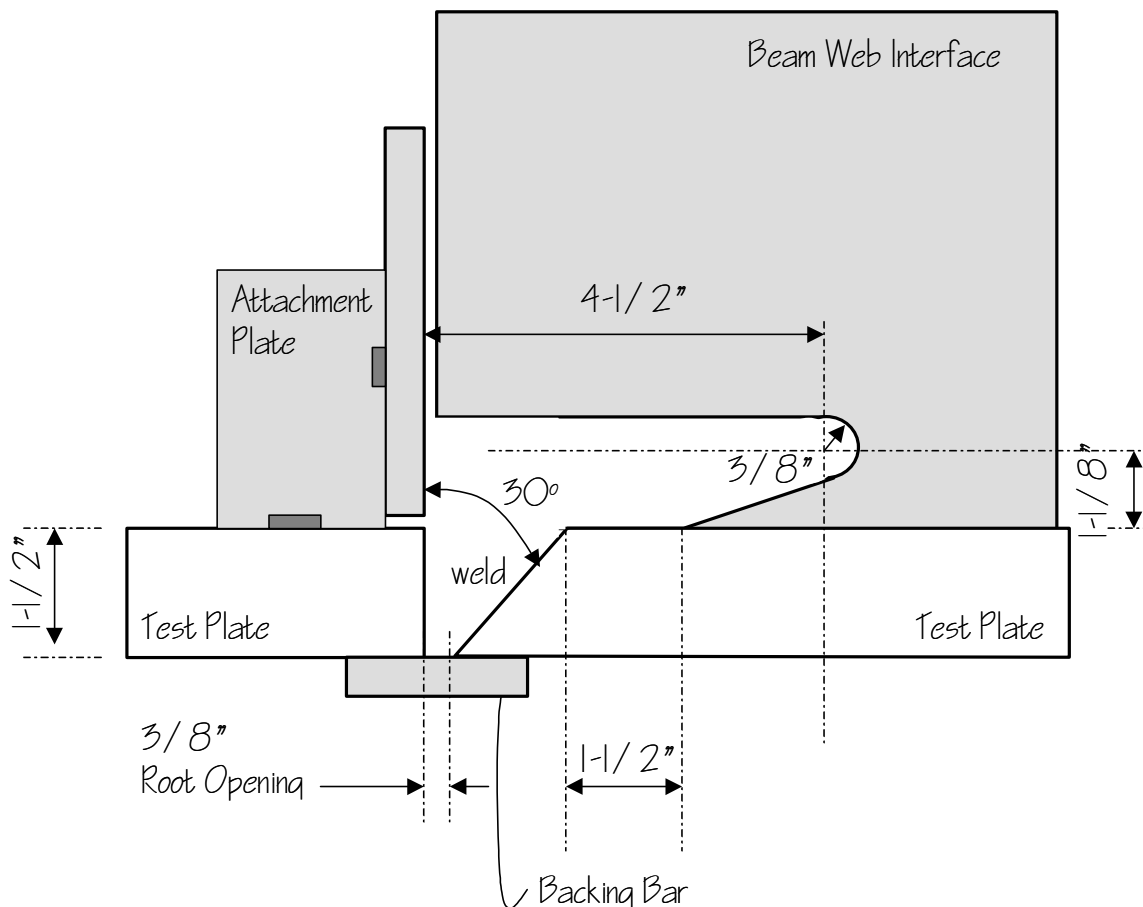


Figure B-1 Supplemental Welder Qualification Test Assembly

Part I

APPENDIX C. SUPPLEMENTAL CHARPY V-NOTCH TOUGHNESS TESTING FOR COMBINATIONS OF FILLER METALS

Scope

This appendix supplements Part I, Section 3.3.2 and specifies provisions for testing to confirm adequate Charpy V-Notch (CVN) toughness of weldments when the Flux-Cored Arc Welding – Self-Shielded welding process is combined with any other welding process, including Flux-Cored Arc Welding – Gas Shielded, and the weld is subject to CVN toughness requirements.

Test Specimens

One test plate as described below shall be used. Five CVN tests shall be made from the test plate. The sequence of weld layers from the different processes shall be the same as that to be employed in production. Approximately one-third the thickness of the test joint shall be welded with the welding material to be used first, or the existing welding material. The balance of the joint shall be welded with the welding material to be used following the first.

A single test plate, which may be of any AISC-listed structural grade, shall be used. The test plate shall be $\frac{3}{4}$ inches thick with a $\frac{5}{8}$ -inch root opening and 20° included groove angle. The test plate and specimens shall be as shown in Figure 5.1 in AWS D1.5-95, except that only an adequate length needed to provide the CVN test specimens, but not less than six inches, is necessary.

All test specimens shall be taken from near the centerline of the weld at the mid-thickness location, in order to minimize base metal dilution effects. CVN specimens shall be prepared in accordance with AWS B4.0-92, *Standard Methods for Mechanical Testing of Welds*, Section A3.

Acceptance Criteria

The lowest and highest values obtained for the five specimens shall be disregarded. Two of the remaining three values shall equal, or exceed, the specified toughness of 40 ft-lbf energy level at the testing temperature. One of the three may be lower, but not lower than 30 ft-lbf, and the average of the three shall not be less than the required 40 ft-lbf energy level.

Part I

APPENDIX D. SUPPLEMENTAL TESTING FOR EXTENDED EXPOSURE LIMITS FOR FLUX-CORED ARC WELDING FILLER METALS

Scope

This appendix supplements Part I, Section 3.3.3 and specifies provisions for test procedures for use in qualifying the extended exposure time for flux-cored arc welding filler metals.

Test Specimens

The electrode shall be exposed to an environment of 80°F (+ 5°F, - 0°F) and 80% (+5%, - 0%) relative humidity for the period of time desired to be approved for accumulated exposure. No conditioning of the electrode after exposure and prior to testing is permitted. The environmental chamber shall meet the requirements of AWS A5.5-96, Section 15.4, except for Section 15.4(3). Exposure time shall begin when the environmental chamber has reached the required temperature and humidity. Time, temperature, and humidity shall be continuously recorded for the period that the electrodes are in the chamber.

All testing for diffusible hydrogen levels shall be performed in accordance with AWS A5.20-95, Section 15, – Diffusible Hydrogen test, modified as follows:

1. The diameter of the electrode that will be used shall be tested.
2. Welds shall be made at a wire feed rate (or welding current) that is at least 75% of the manufacturer's maximum recommended wire feed rate (or welding current) for the electrode type and size being tested. The contact tip-to-work distance (TTW) shall be at the minimum recommended by the manufacturer.

Specification Limits

Based upon test results, the manufacturer of the electrode, or a consultant approved by the Engineer for such purposes, shall provide recommendations for storage and atmospheric exposure to ensure that the diffusible hydrogen level will not exceed the limits prescribed in Part I, Section 2.4.1.2. Such recommendations may take the form of fixed exposure periods, varying exposure limits using temperature-humidity charts similar to those provided in AWS D1.1-98, Annex VIII, and may include provisions for heated storage and baking of electrode wire. Rebaking of wire shall be included only when documented by testing the wire after baking to verify that the baking procedure returns the electrode to the diffusible hydrogen level prescribed in Part I, Section 2.4.1.2.

Part I

APPENDIX E. SUPPLEMENTARY ULTRASONIC TECHNICIAN TESTING

Scope

This appendix supplements Part I, Section 6.3.3 and provides procedures to be used to demonstrate proficiency in ultrasonic testing. The Quality Assurance (QA) Agency shall require ultrasonic testing (UT) personnel to demonstrate proficiency by satisfactory performance in a prequalification examination. The examination shall consist of practical tests that have been developed by the Agency's UT Level III Specialist, or an organization approved by the Agency, and shall incorporate the specific requirements of the Agency's procedures and the acceptance standards contained in Part I, Sections 5.8.3 and 5.8.4 as applicable. The practical examination shall also test the ability of UT personnel to complete correctly the relevant reports associated with the examination.

Test Specimens

The QA Agency shall have test coupons prepared that are of the type and number required to represent the details of the steel moment frame welded joints to be examined. Suitable test pieces should be full mock-ups of welded joints, with and without backing bars. Typical joint geometries of butt, corner and tee configurations shall be utilized. A minimum of 20 flaws shall be embedded in the test samples to provide an adequate test of ability.

Test coupons may contain natural discontinuities, artificial reflectors consisting of non-metallic inserts in the weld deposit, slots or holes machined in the weld, or thin steel inserts welded to bevel preparations to simulate incomplete fusion.

Test specimens to be employed in repetitive examinations shall be fabricated to produce intentional reflectors and to minimize natural flaws. These test pieces shall subsequently be examined by an ultrasonic specialist to confirm the detectability of the implant and the absence of unintentional reflectors. The UT Level III Specialist should characterize each reflector in the test plates into one of the categories defined herein. The characterization, size, and placement of the reflectors may be discussed with the candidate following completion of the examination. If the test pieces are to be used for later examinations, the examiner shall not reveal the exact details of the individual test pieces to avoid compromising the results of the subsequent examinations.

All materials to be employed for test specimen fabrication shall be examined by longitudinal wave techniques to ensure the absence of lamination and/or inclusions that might render the test pieces unacceptable for test purposes. Materials with laminations and inclusions may intentionally be incorporated into selected coupons to evaluate the candidate's performance on these imperfections. Plates for tee joints should be produced from steels with enhanced through-thickness properties to minimize lamellar tearing within the test coupon.

The size of discontinuities inserted or induced into the test coupons shall be consistent with the range of flaw size acceptance criteria set forth in Part I, Sections 5.8.3 and 5.8.4, as appropriate.

Candidate Scoring – Flaw Detection

Candidates shall submit a written report of all detected discontinuities found during the examination. The report shall include the characterization of each flaw type (spherical, cylindrical, or planar), the size (length and width, but not height unless flaw sizing is to be employed), the location along the weld, and the relative position within the weld cross-section.

For UT technicians who will perform flaw detection in accordance with Part I, Section 5.8.3, the following numerical system shall be employed in evaluating the UT technician candidate:

The rate of flaws detected, based upon reporting of the flaw and its location, irrespective of the indication rating recorded, is:

$$D = \text{detected flaws} / \text{total flaws}$$

The rate of false indications, based upon a recorded indication that also exceeds the indication rating defined in Part I, Section 5.8.3, is:

$$F = \text{false indications} / \text{total indications}$$

The UT technician rating R is:

$$R = \frac{n-2}{2(n-1)}(1+D-F) \times 100\% \quad (\text{E-1})$$

where n is the total number of flaws in the test specimens.

For technicians who will perform flaw detection in accordance with Part I, Section 5.8.3, the minimum UT technician rating is 80.

Candidate Scoring – Flaw Sizing

For technicians being qualified to perform flaw sizing in accordance with Part I, Section 5.8.4, the UT technician rating is established by the following equations:

$$P = \frac{L_c}{L_a} \times 100\% \quad (\text{E-2})$$

$$R = \frac{L_c}{L_i} \left(1 - \frac{L_f}{L_i} \right) \times 100\% \quad (\text{E-3})$$

where:

- P = percentage of actual reflectors correctly detected and sized.
- R = overall rating including penalty for false alarms, 0 to 100.
- L_a = length of actual reflector contained in the test specimen.
- L_c = indicated length of actual discontinuities that have been correctly sized and located. (Credit is given for the lesser of the reported length or actual length of the reflector).
- L_i = total length of call by the candidate, right or wrong.
- L_f = length of call to where discontinuity exists.

For flaw sizing ratings, each linear inch of test specimen weld shall be considered independently in the compilation of the candidate's performance. Identification of the discontinuity is considered correct when the size and location of the reflector have been determined with sufficient accuracy to rate the discontinuity in accordance with the acceptance criteria. For ultrasonic examination, a dimension indicated within a factor of two of true dimensions (one-half to twice the actual dimension) is considered accurate within the limits of the examination technique.

Equation E-2 indicates the ability of the candidate to locate and size discontinuities that exist in the test pieces. A candidate must achieve a score of 70 or above on Equation E-2 as minimum performance.

Equation E-3 indicates the ability of the candidate to accept the areas of welds in the test pieces where no flaws exist. A low score indicates the candidate may call for a large number of unnecessary repairs during the course of the actual construction work. The Agency should consider, in evaluating the required performance, the consequences of unnecessary repairs, including the fact that weld repairs are made under less-favorable conditions than the original weld, thereby increasing the potential for a defective repair weld. Consequently, a score of 50 or above on Equation E-3 is minimum performance.

Reexamination

Previously qualified personnel shall be reexamined when they have not performed nondestructive UT examination of steel moment frame construction for a period not to exceed six months, when a specific cause to question performance arises, or more frequently when required by the QA Agency as a part of their Written Practice. Technicians shall be tested under this system, as a minimum, every three years.

Part I

APPENDIX F. SUPPLEMENTARY MAGNETIC PARTICLE TESTING PROCEDURES

Scope

This appendix supplements Part I, Section 6.7.1 and provides guidelines for preparation of procedures for magnetic particle testing (MT).

Written Procedures

Written MT examination procedures shall be prepared by the Quality Assurance Agency responsible for nondestructive testing, and approved by the designated party responsible for the Quality Assurance Plan. The procedures shall be developed considering the geometry of the work to be inspected, yoke application, light levels, accessibility, and safety.

The following items shall be detailed in the written procedure:

1. type of weld to be examined,
2. type of magnetizing equipment, including type of current employed,
3. surface preparation,
4. examination sequence,
5. magnetization plan,
6. magnetic field direction,
7. magnetic field strength at inspection location,
8. magnetic particle type and color,
9. interpretation of indications,
10. acceptance criteria, and
11. reporting forms and procedures.

Procedure Evaluation

An evaluation of the system performance and sensitivity shall be demonstrated prior to the beginning of any testing work. A magnetic particle test bar shall be used to verify sensitivity to a depth of 1/8 inch. This requirement shall be included within the written procedure.

Method

Prior to MT, the weld shall be inspected visually for detection of gross imperfections. The inspected surface shall be cleaned where necessary.

The continuous method, using an alternating current (AC) electromagnetic yoke (double leg or single leg configuration) with dry white-light-visible magnetic particles, may be used as a preliminary examination. Rectified AC (that is, DC) and residual induction methods may also be employed for preliminary examination.

Final examination shall use DC current of the strength required to achieve a depth of examination of 1/8 inch. The examination shall take place while the magnetizing current is on, including application of particles, removal of excess particles, and indication interpretation.

Yoke

The yoke shall be of the articulating-leg rectified AC (DC) type to allow for the inspection of various geometries. A single-leg yoke may be used in areas of tight access. Fixed-leg yokes are not permitted. Rectified AC (DC) yokes shall have a minimum lifting power of 40 pounds when the legs are spaced at the inspection distance. Yokes shall be routinely tested to ensure that the required magnetization performance and sensitivity levels are met during inspection.

Magnetization

A magnetizing current at 50 to 60 Hertz AC electromagnetization may be used for detection of surface-breaking discontinuities. System sensitivity shall be checked with a suitably located pie gauge. The current shall then be changed from AC to rectified AC (DC) for deeper penetration of the magnetizing field into the part.

Particles

The magnetic particles shall be dry, finely divided high-permeability ferromagnetic material with low retentivity and a suitable size range. The particles shall have 75 percent (by weight) particles finer than a 120 ASTM sieve mesh, a minimum of 15 percent (by weight) dry particles being finer than a 325 ASTM sieve mesh. Their color shall provide high contrast to the background on which applied. Particles shall be free from rust, fillers, or other material that could interfere with their use. Magnetic particle materials shall be used only once.

Particle application and removal equipment (powder bulbs, aerosol sprays) shall be such that fine indications are not removed by excessive force.

Lighting

Adequate light shall be present for observation of particle indications. A minimum of 100 foot-candles (1000 lux) shall be present at the inspection location, measured with a calibrated light meter.

Surface Condition

The surface shall be cleaned to bare metal prior to magnetic particle testing by grinding, wire brushing, hand scraping, or a combination of these. Welds shall be dry and free from foreign materials such as dirt, grease, rust, and excessive weld spatter. A thin, light coating of contrasting paint may be applied to assist in the detection of smaller imperfections.

Equipment Performance Checks

The performance and sensitivity of the magnetic particle inspection system and procedures shall be checked at regular intervals. The following parameters shall be checked each day, prior to performing any examination, and shall be documented:

1. yoke magnetizing strength,
2. sensitivity to imperfections at a depth of 1/8 inch, using a test bar,
3. yoke set-up,
4. magnetizing field direction,
5. magnetic particle performance, and
6. particle removal force, using standard samples with discontinuities.

Yoke Placement

Examination shall be conducted with sufficient overlap to ensure 100% coverage. However, if the geometry of the piece does not permit 100% evaluation of the piece, this shall be stated on the test report.

For inspection for discontinuities in the longitudinal direction, the yoke shall be placed astride and perpendicular to the weld. The yoke legs shall be positioned such that they are approximately ½ inch from the toe of the weld. The effective area of inspection is between the yoke legs, with a lateral area of approximately 1-1/2 inches on either side of the centerline between the yoke legs. The total length weld coverage for one position of the yoke is approximately 3 inches. Sufficient overlap should be used to ensure 100% coverage when moving along the weld length. Yoke movement in 2-inch intervals should ensure adequate overlap.

For inspection for discontinuities in the transverse direction, the yoke shall be oriented so that the yoke legs are approximately parallel to the weld, approximately $\frac{1}{2}$ inch from the toe of the weld. If the yoke has to be placed on top of the weld to gain access, the technician should ensure the best contact possible of the yoke legs to the weld. The effective area of inspection is between the yoke legs, with a 4-inch leg spacing recommended. Sufficient overlap may be achieved by moving the yoke to a position which overlaps the last position by at least 1 inch.

Particle Application and Removal

Dry particles shall be applied in such a manner that a light, uniform, dust-like coating of particles settles on the part while it is being magnetized. Particle indications shall be observed when being formed as the particles are being applied, and while the excess particles are being removed.

Sufficient air velocity for particle removal must be directed at the area of inspection to remove the excess particles entrapped in areas such as weld undercuts, while retaining the particles held by magnetic flux leakage from any discontinuities.

Interpretation and Evaluation of Indications

Relevant magnetic particle testing indications are those that result from magnetic flux leakage fields formed by discontinuities that attract and hold magnetic particles. False indications are those held primarily by nonmagnetic means such as mechanical entrapment in the toe of a weld. Nonrelevant indications are those particles held in place by magnetic attraction, but not originating from a structural discontinuity such as the boundary of two steels having significantly different magnetic properties or the boundary at the hardened heat-affected zone of a weld.

Testing personnel shall verify indications by performing the following steps:

1. Retest with the yoke field perpendicular to the discontinuity indication, if not already perpendicular.
2. Retest, confirming that excess particles are removed. If the suspect indication is removed during the retest, the indication is interpreted as nonrelevant or as a false indication.
3. If the indication has a light particle buildup and weak particle adhesion, and if doubt exists as to whether the indication is relevant or false, the area of the indication shall be lightly surface-ground and retested.

PART II
RECOMMENDED QUALITY ASSURANCE
GUIDELINES

Part II: RECOMMENDED QUALITY ASSURANCE GUIDELINES

1. GENERAL

1.1 Scope

Part II of these *Recommendations* is to provide information relevant to the quality control (QC) and quality assurance (QA) of the seismic-force-resisting systems of steel moment-frame buildings, in regions subject to potential damaging earthquake ground motion. It is intended to be used in coordination with, and in supplement to, locally applicable building codes and those national standards referenced by the building code. Included are discussions on:

- the various terms used regarding inspection functions (Part II, Chapter 1),
- basic quality control and quality assurance issues (Part II, Chapter 1),
- recommended Contractor (fabricator or erector) qualifications and Quality Control Program for the control of the quality of fabricated and erected structural steel (Part II, Chapter 2),
- recommended Quality Assurance (inspection and testing) Agency qualifications and the qualifications and duties of individual inspection and testing personnel (Part II, Chapter 3),
- recommended requirements and practices for structural steel material (quality, control, and inspection) (Part II, Chapter 4),
- recommended requirements and practices for welding material (quality, control, and inspection); and welding operations (quality and inspection) (Part II, Chapter 5),
- recommended requirements and practices for bolting material (quality, control, and inspection); and bolting operations (quality and inspection) (Part II, Chapter 6).

Commentary: These Recommended Quality Assurance Guidelines apply to the construction of steel moment frames used as the seismic-force-resisting elements of buildings and other structures. As used in these Recommendations, the terms “steel structure” or “steel framing” apply to the structural steel elements of the seismic-force-resisting system as defined in FEMA-302. The quality assurance and control guidelines contained in these Recommendations could be extended to the construction of other types of seismic-force-resisting systems and even to elements of structures that are not intended to be part of the seismic-force-resisting system. However, extension of these Recommendations to these other construction types and elements could result in a significant construction cost premium. These Recommendations are deemed appropriate to application to the construction of steel moment frames used in seismic-force-resisting systems due to the severe service conditions anticipated for those frames and the proven sensitivity of frame performance to construction quality. Specifically, investigations of damaged steel moment-frame connections following the 1994 Northridge earthquake revealed numerous instances of construction that did not conform to the applicable standards. Common deficiencies included the use of improper weld tabs, welded

joints with excessive bead size and, frequently, large zones at the roots of critical welded joints with lack of fusion and with slag inclusions. These defects contributed directly to the initiation of fractures in these joints. Further, investigations performed as part of this project confirm the following: the importance of weld quality and toughness to connection performance; the sensitivity of welded joint toughness to welder technique; and the general inability of inspection practices commonly employed in the past to provide adequate control to obtain reliable performance. The Quality Assurance Guidelines recommended herein are intended to address these issues directly. Although quality control of other construction systems is also important, it has not been demonstrated that the specific recommendations contained herein are either appropriate or necessary to obtaining reliable performance of those other construction systems.

1.2 Terms

This section describes some of the basic elements of a program necessary to ensure that construction meets the required standards. It also describes the recommended roles and responsibilities of key project participants in this process.

1.2.1 Quality Control

Quality Control (QC) includes those functions to be performed by the Contractor to ensure that the material and workmanship of construction meet the Quality Requirements. QC includes inspection and testing of fabrication and erection, as defined by AWS D1.1-2000, unless otherwise provided for in the Quality Assurance Plan or the Contract Documents. Fabrication and erection inspection includes routine welding inspection items such as personnel control, material control, dimensional control, preheat measurement, monitoring of welding procedures, visual inspection and nondestructive testing when specified.

Commentary: QC functions vary considerably between shop and field operations. For field work, less QC is typically provided, and there is a higher reliance upon Quality Assurance. Nonetheless, field QC is necessary to ensure quality in field operations, and must be provided by the Contractor performing field welding, field bolting, and related steel construction work.

1.2.2 Quality Assurance

Quality Assurance (QA) consists of those inspection services to be performed by an agency or firm other than the Contractor. QA is performed at the prerogative of the Owner, and may be mandated by the building code. QA includes monitoring the performance of the Contractor in implementing the Contractor's QC Program, ensuring that designated QC functions are performed properly by the Contractor on a routine basis. To ensure compliance, QA may also include performance of specific inspection tasks that may be similarly included in the Contractor's QC Plan, including the performance of nondestructive testing. Quality Assurance is termed "Verification Inspection" in AWS D1.1-2000.

1.2.3 Quality Control Plan

The Quality Control Plan is a written statement, prepared by the Contractor, of all measures that will be taken to ensure that steel moment-frame construction conforms to the applicable standards, project specifications, and the Quality Assurance Plan. Key parts of the QC Plan should include a description of the organizational structure, responsibilities, practices, procedures, processes, and resources for implementing and maintaining the project quality requirements, and ongoing efforts to monitor the effectiveness of the Plan. The Contractor's QC Plan should ensure that all parties understand what is to be constructed and the standards that apply.

1.2.4 Quality Assurance Plan

The Quality Assurance Plan is a written statement of the quality requirements for the project, prepared by the Engineer and typically containing two components: (1) the set of material and workmanship standards that the finished construction must meet, and (2) the set of procedures that are to be followed to confirm compliance with these requirements.

Elements in an acceptable Quality Assurance Plan must meet or exceed the requirements of the building code.

The Quality Assurance Plan should include:

- identification of the seismic force resisting system elements subject to the Plan,
- required Contractor Quality Control procedures,
- required services of the Engineer, including but not limited to the following:
 - review of required Contractor submittals,
 - monitoring of required inspection reports and test results,
 - construction consultation as required by the Contractor on the intent of the construction documents,
 - procedures for modification of the construction documents to reflect the demands of unforeseen field conditions discovered during construction,
- a listing of all Special Inspections and testing required by the building code, supplemental inspections and testing required by the Engineer, as well as the reference standards and the project standards,
- the type and frequency of Special Inspections, including a schedule for any periodic inspections,
- the type and frequency of testing,
- the frequency and distribution of special inspection and testing reports,
- the structural observations to be performed, and
- the frequency and distribution of structural observation reports.

The Quality Assurance Plan is typically prepared using reference to building code requirements and approved national standards, and supplemented by individual project specifications.

Commentary: A Quality Assurance Plan is required under the provisions of IBC Section 1705. The minimum requirements for the Plan are provided in IBC Section 1705.2.

A registered design professional must prepare the Quality Assurance Plan. See IBC Section 1705.2. Commonly, the Owner contracts with the engineer in responsible charge of the design for the preparation of the Quality Assurance Plan for the structural system. It is recommended in FEMA-302 that this engineer prepare the Quality Assurance Plan for the structural system, as this person is most familiar with the critical features of the designs that are important to its performance.

1.2.5 Contract Documents

The Contract Documents are all those drawings, specifications, notes and other documents prepared by the Engineer or other registered design professional, that define the structural system and quality requirements. The Contract Documents should include the specification of:

- the Quality Assurance Plan,
- the construction material,
- the requirements for Contractor material controls,
- if necessary, special material inspection requirements beyond standard practice or those designated in ASTM, AISC or AWS requirements,
- quality acceptance criteria,
- fabrication, erection, and installation tolerances, by reference to industry standards, and supplemented as needed for project-specific conditions, and
- the method of resolution of disputes regarding structural integrity.

The basic code requirements for structural welding of steel structures are contained in AWS *D1.1 - Structural Welding Code – Steel*. AWS *D1.1* uses the term “Fabrication/Erection Inspection” synonymously with the Quality Control function. The term “Verification Inspection” is used to describe the Quality Assurance function. AWS *D1.1* requires inspection for many items in Section 6, Part A, but does not specify whether the inspection is performed as either QC or QA. For this reason, distinction between QC and QA must be provided in the Quality Assurance Plan and Contract Documents.

Fabrication/Erection Inspection (or Quality Control), Verification Inspection (or Quality Assurance), and Nondestructive Testing should be performed at the same time whenever possible, to ensure that the Contractor's QC program is meeting the requirements of the Contract Documents.

1.2.6 Owner

Construction is performed on behalf of the owner. The Owner is responsible for providing Quality Assurance. The Owner must ensure that a suitable Quality Assurance Plan is established and implemented. The Owner should employ one or more Special Inspectors to observe the construction for compliance with the applicable standards and project specifications. The Owner may also direct the Engineer or other registered design professional to oversee the Quality Assurance function.

Commentary: Should the Owner have insufficient expertise or knowledge related to construction, the Owner should retain a qualified Engineer or other registered design professional to advise the Owner, prepare the Quality Assurance Plan, and oversee the QA program.

1.2.7 Engineer

The Engineer is the registered design professional in responsible charge of the design of the structural steel framing system. The Owner may select other registered design professionals for other related functions, including the preparation of the Quality Assurance Plan.

Commentary: The Engineer or other registered design professional may assist the Owner in:

- *establishing the Quality Requirements for the project, including the standards that the construction must meet and the QC and QA measures that are to be followed,*
- *establishing the Quality Assurance Plan for the project,*
- *establishing the necessary qualifications of the Contractor,*
- *establishing the necessary qualifications of the Quality Assurance Agency or firm,*
- *establishing the necessary qualifications of the individuals responsible for inspection and testing,*
- *evaluating the Contractor's Quality Control Plan, and*
- *evaluating the Quality Assurance Agency's Written Practice.*

The Engineer should additionally review Contractor submittals and prepare revisions to the structural drawings when necessitated by changes in the work.

1.2.8 Contractor

For the purposes of these *Recommendations*, the Contractor is the firm performing the fabrication or erection of the structural steel framework.

1.2.9 Building Official

As used in these *Recommendations*, the term Building Official is applied to the regulatory agency having jurisdiction of the building.

The Building Official should:

- require that construction documents, including the Quality Assurance Plan with at least the following items, be submitted prior to the issuance of a building permit:
 - a complete list of material and work requiring special inspections,
 - the inspections to be performed, and
 - a list of the individuals, approved agencies or firms intended to be retained for conducting such inspections;
- approve the final selection of Special Inspectors;
- maintain files of inspection reports for a defined length of time following completion;
- ascertain that all reported noncompliances have been rectified, or accepted by the Engineer as acceptable; and
- issue a certificate of occupancy.

Commentary: Section 1704.1.1 of the International Building Code requires the listed items to be submitted prior to issuance of a building permit. Section 1704.1 requires the approval of the Special Inspector by the Building Official.

1.2.10 Quality Assurance Agency

The Quality Assurance (QA) Agency is the independent testing organization selected by the Owner, or Owner's designee, to perform the Quality Assurance inspection and testing required by the Quality Assurance Plan, the specifications and the Contract Documents.

1.2.11 Special Inspector

The Special Inspector, as required by *International Building Code (IBC)* Section 1704.1, must be "a qualified person who shall demonstrate competence, to the satisfaction of the Building Official, for inspection of the particular type of construction or operation requiring special inspection." The Special Inspector is employed by the Owner, to provide the special inspections required. "Employed" in this case may be through a contractual relationship.

1.2.12 Special Inspection

Special Inspection is a Quality Assurance activity mandated by the Building Code. Special Inspection is defined by the *IBC* Section 1702.1 as "inspection as herein required of the materials, installation, fabrication, erection or placement of components and connections requiring special expertise to ensure compliance with approved construction documents and referenced standards."

Special Inspection is the detailed and specific inspection to ensure compliance with the Building Code. The duty of a Special Inspector is defined in *IBC* Section 1704.

1.2.12.1 Continuous Special Inspection

Continuous Special Inspection is the full-time observation and inspection of work requiring Special Inspection by an approved Special Inspector who is present in the area where the work is being performed.

1.2.12.2 Periodic Special Inspection

Periodic Special Inspection is the part-time or intermittent observation of work requiring Special Inspection by an approved Special Inspector who is present intermittently in the area where the work is being performed and who is present at the completion of the work.

1.2.13 Nondestructive Testing

Nondestructive testing (NDT) includes magnetic particle testing (MT), liquid dye penetrant testing (PT), radiographic testing (RT) and ultrasonic testing (UT). For the purpose of these *Recommendations*, the terms nondestructive examination (NDE) and nondestructive testing (NDT) are identical.

Commentary: The purpose of NDT is to serve as a backup to Visual Inspection (VI) and to detect flaws and defects in base metals and welds that are not otherwise visible. NDT is not a replacement for an adequate program of QC, QA and VI, and should not be used as such.

1.2.14 Structural Observation

Structural Observation is defined by the *IBC* in Section 1702.1 as “the visual observation of the structural system by a registered design professional for general conformance to the approved construction documents at significant construction stages and at completion of the structural system.”

Structural Observation should also include review of testing and inspection reports for general compliance with the construction documents. Observed deficiencies should be reported in writing to the Owner’s representative, Special Inspector, Contractor, and Building Official. A report to the same parties should be made at the completion of the structural work regarding the observations that have been made.

Structural Observation does not replace or waive Special Inspection, nor does it serve the same function. Requirements for Structural Observation are contained in *IBC* Section 1709. A recommended checklist for structural observation is included in Figure 1-1.

Structural Observation Checklist

(Steel Moment Frames for Seismic Application)

Perform Structural Observation upon completion of the first few moment connections and column splices, prior to placement of decking, covering by fireproofing, encasement in concrete or placement of other finishes. After verification of these moment connections, conduct Structural Observation on a periodic basis or when structural details or requirements change, and also upon completion of the structural framing.

Observation of welded joints is for general configuration of the joint only (for example, placement of components, weld type, backing presence, joint finishing), not for weld quality.

Measurement of members, connection material and welds is not a part of Structural Observation, as this is performed as a part of inspection.

Checklist

- ☐ Orientation and placement of connected components
- ☐ Removal of backing bars, as required
- ☐ Placement of reinforcing fillets, as required
- ☐ Removal and finishing of runoff tabs, as required
- ☐ Presence of continuity plates, as required
- ☐ Welding of continuity plates, as required
- ☐ Presence and type of doubler plates, as required
- ☐ Welding of doubler plates, as required
- ☐ Configuration and finish of access holes
- ☐ Placement of beam stiffeners, as required
- ☐ Contour and finish of RBS profile, if applicable
- ☐ Placement of welds for web connection, as required
- ☐ Type and placement of bolts
- ☐ Inaccessible conditions

Figure 1-1 Structural Observation Checklist

Commentary: Structural Observation, when required, must be performed by a registered design professional, typically the Engineer or the Engineer's designated representative.

1.2.15 Written Practice

The Written Practice is a document prepared by the Quality Assurance Agency describing its procedures for determining and ensuring the acceptability of the structure in accordance with the applicable codes, standards, specifications and procedures, including general inspection, material controls, visual welding inspection, and bolting inspection.

The Written Practice should describe the selection and administration of inspection personnel, and the training, experience and examination requirements for qualification and certification of inspection personnel.

1.2.16 Hold Point

A Hold Point is a stage of interim construction completion, beyond which the work should not proceed until the designated inspector has completed the inspection tasks specified for this stage.

1.2.17 Observation Point

An Observation Point is a stage of interim construction completion, beyond which work may proceed after the worker has completed the worker's own inspection, with further inspection provided by the designated inspector through observation of the work operations on a routine, occasional, random basis.

1.3 Pre-Job Meeting

It is recommended that a pre-job meeting or series of meetings be held with the Owner's representative responsible for the preparation and administration of the Quality Assurance Plan, the Fabricator's and Erector's Quality Control personnel, and the Quality Assurance Agency's personnel to plan and discuss the project, fabrication procedures, erection procedures, and inspection procedures. Topics of discussion at such meetings should include:

- overall scope of work for the project,
- any special construction required by the Contract Documents,
- inspection responsibilities, including the duties of welding, bolting and other fabrication and erection personnel, the Fabricator's and Erector's QC personnel, and the QA Agency's personnel,
- the use of hold and observation points,
- any specific nondestructive testing requirements that apply to the project, particularly those exceeding Code requirements or typical practice,
- method of notification and approval relative to determining correction of nonconforming work,
- locations and production schedules for off-site work,
- schedule and application of any approved periodic inspection,
- conditions requiring notification of the Engineer or Building Official, or both, and
- reporting procedures and frequency.

Either through a meeting or by direct dissemination of the information, inspection personnel should also be provided information regarding project-specific requirements.

1.4 Fabrication and Erection Drawings

The Contract Documents should require submittal of erection and shop drawings for the work. Shop drawings should provide sufficient information to allow fabrication to be performed. The erection drawings should identify piece marks for each fabricated element to be incorporated into the work, coordinated with the individual shop drawings. Any special erection sequence or erection instructions should be included on the erection drawings. The Engineer should review the shop and erection drawings to ensure that the overall structural configuration and detailing requirements conform to the design intent.

Part II

2. CONTRACTOR QUALIFICATIONS AND QUALITY TASKS

2.1 Scope

This chapter outlines the general qualifications and quality responsibilities of the Contractor. As used in this chapter, the “Contractor” is the steel fabricator, responsible for the shop fabrication of the structural steel, or the steel erector, responsible for all field erection and field welding. The general contractor or construction manager is not the “Contractor” as used in this section, unless performing some or all of the duties of fabricator or erector.

The Owner should ensure that the Contractor has established and put into practice an adequate Quality Control Plan (QCP). The Quality Assurance Plan should provide oversight to the Contractor's QCP. This may range from simple records and report reviews to a full testing and inspection program, depending upon the effectiveness of the Fabricator's or Erector's QCP, and the requirements of both the Quality Assurance Plan and the Building Code.

The Owner should be involved in the selection or approval of the Contractor. In order to establish the necessary level of Quality Control (QC) and Quality Assurance (QA) for a particular project, it is necessary to establish the level of experience and competence of the Contractor. Adjustments to the QA Plan may be appropriate after the Contractor has been selected.

Commentary: Although Section 1702.2 of the IBC exempts work performed by “Approved Fabricators” from some inspections, the use of “certified” or “qualified” fabrication shops, in lieu of requiring independent Quality Assurance provided by the Owner, is not recommended. However, a fabrication shop that is certified or qualified by a recognized program, such as the American Institute of Steel Construction (AISC) Quality Certification Program, does provide a level of assurance that the fabricator has the capability of providing good fabrication performance.

2.2 Contractor Quality Control Plan

The Contractor must establish, implement and maintain a suitable QCP for in-process quality control of the work.

The Contractor's QCP should, as a minimum, include the following:

- designation and qualifications of project QC personnel,
- designation and qualifications of personnel responsible for supervision of the work,
- Contractor's written QC and procedures manual,
- Contractor's written weld repair procedures, and
- Contractor's Nondestructive Testing (NDT) procedures and NDT personnel training records.

The Contractor's QCP must be in writing, enabling others to make an effective evaluation of its adequacy. The Contractor should also make periodic reviews, at least annually, of the QCP to verify its adequacy and to determine that it is being followed at all levels of the Contractor's operations.

The Contractor's QC function should be isolated from the production department, and the QC Manager should report directly to a high level company officer to avoid conflicts of interest with production.

2.3 Contractor's Statement of Responsibility

Prior to the commencement of work on the structural system, the Contractor should submit to the Owner, and the authority having jurisdiction, a written statement of responsibility containing the following:

- acknowledgment of awareness of the special requirements contained in the Quality Assurance Plan,
- acknowledgment that control will be exercised to obtain conformance with the Contract Documents approved by the authority having jurisdiction,
- procedures for exercising control within the Contractor's organization, the method and frequency of reporting, and the distribution of reports, and
- identification and qualifications of the persons exercising such control and their positions in the organization.

Commentary: In many cases, one firm performs fabrication, defined as work performed in a fabricating shop, and another firm performs erection, defined as work performed at the jobsite. Often, the fabrication and erection are under one contract, but the erection portion is sublet to an erection specialty subcontractor. When a single firm directly performs both functions, the management of each operation is often separate, and therefore the fabrication and erection operations, and the statement of responsibility, should be evaluated as though coming from separate organizations.

This Section 2.3 is similar to that recommended in FEMA-302, Section 3.2.2, and adopted as IBC Section 1705.3.

2.4 Certification Programs

To assist in evaluating the adequacy of a Contractor's qualifications, Contractor certification under one or more of the programs indicated in this section may be considered.

2.4.1 AISC Quality Certification

The American Institute of Steel Construction, Quality Certification Program, is a voluntary program under which owners, engineers and building officials can evaluate the quality of steel

fabricators. Steel fabricators are evaluated on the basis of their overall quality control program, judging the general management, engineering and drafting, purchasing, shop operations and quality control functions of the individual plant. Fabricating companies having more than one plant are certified on a plant-by-plant basis. The AISC Quality Certification Program is used to determine if the plant “has the personnel, organization, experience, procedures, knowledge, equipment, capability and commitment to produce fabricated steel of the required quality for a given category of structural steel work.”

There are two categories of certification for steel buildings, plus two categories for steel bridge structures. Conventional Steel Building Structures, previously known as Category I, include, but are not limited to:

- small public service and institutional buildings (for example, schools),
- shopping centers,
- low-rise truss and beam / column structures,
- light manufacturing plants,
- miscellaneous and ornamental iron fabricated to AISC Specifications,
- warehouses, and
- sign structures.

Complex Steel Building Structures, previously known as Category II, include, but are not limited to:

- large public service and institutional buildings,
- high rise buildings,
- stadiums,
- auditoriums,
- heavy manufacturing plants,
- power plants (fossil, non-nuclear),
- metal producing and metal rolling facilities,
- crane girders,
- bunkers and bins,
- chemical processing plants, and
- petroleum processing plants.

Fabricating plants certified for Complex Steel Building Structures are automatically certified for Conventional Steel Building Structures.

The AISC Quality Certification Program, Category MB - Metal Building Systems, refers to pre-engineered metal buildings. This category of certification includes a review of the design procedures used by the manufacturer of the building system in designing the structure, as well as the manufacturer's fabrication capabilities and systems.

2.4.2 AISC Erector Certification

The American Institute of Steel Construction, Erector Certification Program, is a voluntary program under which the construction industry can evaluate the quality of steel erectors. Steel erectors are evaluated on the basis of their overall quality control program, judging the management and operations of the erector.

The AISC Erector Certification Program is used to determine if the erector "has the personnel, organization, experience, procedures, knowledge, equipment, capability and commitment to erect fabricated steel of the required quality for a given category of structural steelwork."

There are two categories of certification for structural steel erectors. These are Certified Steel Erectors and Certified Advanced Steel Erectors.

A Certified Steel Erector may provide services on the same types of structures included under fabricator certification for Conventional Steel Building structures, as described in Part II, Section 2.4.1. In addition, a Certified Steel Erector may provide services for steel frame buildings up to ten stories in height.

A Certified Advanced Steel Erector may provide services on the same types of structures for fabricator certification for Complex Steel Building Structures, under Part II, Section 2.4.1. In addition, Certified Advanced Erectors may provide services for repair and upgrade of existing steel buildings.

2.4.3 Building Code Evaluation Services

The International Conference of Building Officials (ICBO) Evaluation Services, Inc. and Building Officials and Code Administrators (BOCA) Evaluation Services, Inc. provide evaluations of various manufacturers and products. Under ICBO Evaluation Services, steel fabricators are listed as Fabricators of Prefabricated Buildings and Components (064). BOCA Evaluation Services listings of steel fabricators and steel fabrications are contained in Division 5, but no specific category exists for structural steel fabricators.

2.4.4 Alternatives to Certification

2.4.4.1 International Standardization Organization (ISO) 9000

The use of *ISO 9000* or related credentials of the steel fabricator may be considered as an alternative to certification programs such as the AISC Quality Certification Program or ICBO Evaluation Services, as described above. A review of the fabricator's program and the audits conducted is advisable.

Commentary: Few fabricators have presently sought such ISO certification.

2.4.4.2 Jurisdiction Evaluation

Some local jurisdictions, such as the cities of Los Angeles and Houston, have programs in place to evaluate steel fabricators. Before relying on such evaluation programs, the Owner or Owner's designated representative should review the adequacy of such programs to qualify the Contractor for the type of work being undertaken.

Commentary: Although some jurisdictions exempt from inspection work performed in approved shops, this practice is not recommended.

2.4.4.3 Individual Evaluation

The Owner or a designated representative may evaluate the fabricator and erector individually to ascertain their levels of experience and skill, and to evaluate their Quality Control Plans. Such evaluations should be undertaken by personnel experienced in the field of structural fabrication or erection, including material control, welding, and bolting. Recommended checklists for evaluation of fabricators and erectors are presented in Figures 2-1 and 2-2, respectively.

When inspection or Nondestructive Testing (NDT) is performed by the Contractor (the fabricator or erector) as a part of the Quality Control function, the qualifications of the individuals performing such inspection or NDT should meet the same standards as the individuals performing Quality Assurance. It is not expected that the Contractor, as a firm, has the same qualifications or certifications as a Quality Assurance Agency performing such work, but it is important that the actual work be done in a satisfactory manner.

2.5 Contractor Obligations

The Contractor should cooperate fully with requests from inspection and testing personnel for access to the connections and joints to be inspected or tested. This includes beam and column turning in the shop, weld backing removal when NDT indicates a rejectable condition, and access to platforms or scaffolding as required to perform the work safely. When such services are required, the inspection must be performed in a timely manner so as not to hinder production or require duplication of effort.

The Contractor is responsible for all necessary corrections of deficiencies in material and workmanship. The Contractor should comply with requests of the Inspector to correct deficiencies, when such corrections are required by the Engineer or other designated responsible party. The NDT Technician should be apprised of any repairs made by the Contractor. Disputes regarding acceptance, rejection, repair and replacement of welds should be resolved by the Engineer or designated design professional.

Fabricator Evaluation Checklist

Note: This list is adapted from AISC Quality Certification for Complex Steel Building Structures. A more complete checklist is available from AISC, upon request.

Management

1. Does management review project quality requirements prior to production?
2. Does management assign or contract for project activities to suitably qualified personnel?
3. Does management select or create necessary quality procedures for the work?
4. Are special project quality requirements (for example, material controls, weld restrictions) communicated to plant departments?
5. Are fabrication and erection requirements (adjustment needs, erection aids, sequencing of NDT) and priorities reviewed prior to production?
6. Is shop supervision conversant with current provisions of AWS and AISC specifications?
7. Has the fabricator previously fabricated structures of similar complexity?
8. If fabrication is subcontracted to another fabricator, is the subcontractor evaluated and approved in a similar manner?

Page 1

Figure 2-1 Fabricator Evaluation Checklist

Fabricator Evaluation Checklist

Engineering and Detailing

1. Is there a qualified person supervising in-house detailing, and evaluating and coordinating outside detailing?
2. Does the company have capable in-house staff or consultants qualified by registration or experience?
3. Are requests for information documented?
4. Are detailed drawings checked by qualified personnel?
5. Are all detail drawings reviewed or approved by the Engineer?
6. Is there a current library of standards and specifications?
 - AISC
 - o *Manual of Steel Construction*, including Volume II Connections
 - o *Seismic Provisions for Structural Steel Buildings*
 - o *Code of Standard Practice*
 - o Quality Criteria and Inspection Standards
 - ANSI/AWS
 - o *D1.1 – Structural Welding Code – Steel*
 - o A5.xx – specifications for electrodes and shielding gases being used
 - ASTM
 - o *Volume 1.04 – Steel-Structural*, or separate specifications for the material used
 - o *Volume 1.08 or 15.08 – Fasteners*, or separate specifications for the material used
 - RCSC
 - o *Specification for Structural Joints Using ASTM A325 or A490 Bolts*
 - Applicable building codes

Page 2

Figure 2-1 Fabricator Evaluation Checklist (continued)

Fabricator Evaluation Checklist

Structural Steel

1. Are controls established to ensure adequate identification of incoming purchased items?
2. Are mill test reports kept on file?
3. Is material inspected for conformance to *ASTM A6*?
4. Is a written procedure used to provide identification of steel grade, and where required, heat numbers and material test reports for special requirements?
5. Is the grade of material and marking verified prior to fabrication, during fabrication, and restocking?

Welding

1. Are all electrodes, fluxes and shielding gases checked upon receipt for conformance to purchasing documents?
2. Are manufacturer's certifications of conformance kept on file?
3. Are welding electrodes, flux, and shielding gases stored properly and identified?
4. Are flux and rod ovens adequate and operating according to AWS specifications?
5. Are wire-feed welding machines (FCAW, GMAW, SAW) periodically checked to ensure correct current and voltage readings, and is a record kept?
6. Is SMAW welding equipment in use in acceptable operating condition?
7. Does the fabricator have mechanically-guided burning equipment?
8. Have welding personnel been qualified to AWS requirements?
9. Is a welder identification system in place and used?
10. Do welders inspect their welds to AWS workmanship provisions?
11. Are approved written WPSs in close proximity to, and used by, the welders?
12. Is a check made to ensure that approved WPSs are disseminated and followed in the shop?
13. Does the fabricator have a competent welding technician, supervisor or consultant available on call to resolve welding problems?

Page 3

Figure 2-1 Fabricator Evaluation Checklist (continued)

Fabricator Evaluation Checklist

Bolting

1. Are all bolts, nuts and washers checked upon receipt for conformance to purchasing documents?
2. Are manufacturer's certificates of conformance kept on file?
3. Are fasteners suitably stored and identified by production lot?
4. Does the fabricator have adequate and accurate punching equipment, drilling equipment, or both?
5. Is a bolt tension calibration device available for pre-installation testing and wrench calibration?
6. Are written bolt installation and tightening procedures available and used?

General Fabrication

1. Does the fabricator have adequate and accurate material cutting and finishing equipment?
2. Does the fabricator have material handling equipment, including cranes, to move material without damage?
3. Is fabrication in accordance with approved shop detail drawings?
4. Is fabrication in accordance with Contract Documents and specifications?

Page 4

Figure 2-1 Fabricator Evaluation Checklist (continued)

Fabricator Evaluation Checklist

Quality Control

1. Is there a written Quality Control Plan?
2. Is workmanship checked throughout the fabrication process?
3. Are Quality Control procedures followed?
4. Is inspection in accordance with the company inspection procedure?
5. Is a record kept of all inspections, such as by noted detail drawings?
6. Do all pieces receive a final inspection, and is a record kept of this inspection?
7. Are shop inspectors trained and qualified to perform inspection?
8. Do inspectors have the following equipment available: tapeline, welding gages, tag system?
9. Are there adequate procedures for coordination with outside inspectors and NDT technicians?
10. Is there a separation of responsibility between production and Quality Control?
11. Does Quality Control have the authority to stop the work, and the responsibility to inform the operating supervisor of non-conforming work?
12. Is there a functioning, written procedure for the disposition of non-conforming material or work?

Page 5

Figure 2-1 Fabricator Evaluation Checklist (continued)

Erector Evaluation Checklist

Note: This list is adapted from AISC Quality Certification for Complex Steel Building Structures and from the AISC Erector Certification Program.

Management

1. Does management review project quality requirements prior to erection?
2. Does management select or create necessary quality procedures for the work?
3. Are special project quality requirements (for example, material controls, weld restrictions) communicated to purchasing and field supervision?
4. Does management assign or contract for project inspection to suitably qualified personnel?
5. Are erection requirements (adjustment needs, erection aids, sequencing of NDT) and priorities reviewed prior to erection?
6. Are field superintendents and foremen conversant with the current provisions of AWS and AISC specifications?
7. Has the erector previously erected structures of similar complexity?
8. If erection or inspection is subcontracted to another firm, is the subcontractor evaluated and approved in a similar manner?

Page 1

Figure 2-2 Erector Evaluation Checklist

Erector Evaluation Checklist

Project Engineering

1. Is there a qualified person supervising in-house project engineering and coordinating outside engineering services?
2. Are requests for information, and responses, documented?
3. Is there a current library of standards and specifications?
 - AISC
 - o *Manual of Steel Construction*, including Volume II Connections
 - o *Seismic Provisions for Structural Steel Buildings*
 - o *Code of Standard Practice*
 - o Quality Criteria and Inspection Standards
 - ANSI/AWS
 - o *D1.1 – Structural Welding Code – Steel*
 - RCSC
 - o *Specification for Structural Joints Using ASTM A325 or A490 Bolts*
 - Applicable building codes

Fabricated Structural Steel

1. Are controls established to ensure adequate identification of delivered items?

Page 2

Figure 2-2 Erector Evaluation Checklist (continued)

Erector Evaluation Checklist

Welding

1. Are all electrodes, fluxes and shielding gases checked upon receipt for conformance to purchasing documents?
2. Are welding material manufacturer's certifications of conformance kept on file?
3. Are welding electrodes, flux, and shielding gases stored properly and identified?
4. Are electrode ovens adequate and operating according to AWS specifications?
5. Are electrode ovens placed in reasonable proximity to the welding operations?
6. Are wire-feed welding machines (FCAW, GMAW, SAW) periodically checked to ensure correct current and voltage readings, and is a record kept?
7. Is SMAW welding equipment in use in acceptable operating condition?
8. Does the erector have suitable thermal cutting, air arc gouging, grinding and hole-reaming equipment?
9. Have welding personnel been qualified to AWS requirements?
10. Is a welder identification system in place and used?
11. Do welders inspect their welds to the workmanship provisions of the applicable AWS specifications?
12. Are approved written weld procedures in close proximity to and used by the welders?
13. Is a check made to ensure that approved welding procedures are disseminated and followed?
14. Does the erector have a competent welding technician, supervisor or consultant available on call to resolve welding problems?

Page 3

Figure 2-2 Erector Evaluation Checklist (continued)

Erector Evaluation Checklist

Bolting

1. Are all bolts, nuts and washers checked upon receipt for conformance to purchasing or shipping documents?
2. If supplied by the erector, are manufacturer's certificates of conformance kept on file?
3. Are fasteners suitably stored and identified by production lot?
4. Is a bolt tension calibration device available for pre-installation testing and wrench calibration?
5. Are written bolt installation and tightening procedures available and used?

General Erection

1. Does the erector have erection equipment, including cranes, to move material without damage?
2. Is a complete set of erection drawings and shop detail drawings maintained at the jobsite?
3. Are project specifications or summaries of specifications and special instructions kept at the jobsite?
4. Is a complete set of erection procedures maintained at the jobsite?
5. Is erection in accordance with approved erection plans and procedures?
6. Is erection in accordance with Contract Documents and specifications?

Page 4

Figure 2-2 Erector Evaluation Checklist (continued)

Erector Evaluation Checklist

Quality Control

1. Is there a written Quality Control system?
2. Is workmanship checked throughout the erection process?
3. Are Quality Control procedures followed?
4. Is inspection in accordance with the company inspection procedure?
5. Is a record kept of all inspections, such as by noted detail drawings or erection drawings?
6. Do all connections receive a final inspection, and is a record kept of this inspection?
7. Are field inspectors trained and qualified to perform inspection?
8. Do inspectors have the following equipment available: tapeline, welding gages, tag system?
9. Are there adequate procedures for coordination with outside inspectors and NDT technicians?
10. Is there a separation of responsibility between production and Quality Control?
11. Does Quality Control have the authority to stop the work, and the responsibility to inform the field superintendent of non-conforming work?

Page 5

Figure 2-2 Erector Evaluation Checklist (continued)

Part II

3. Quality Assurance Agency Qualifications and Quality Assurance Tasks

3.1 Scope

This section defines the appropriate qualifications for organizations and individuals performing Quality Assurance (QA) of steel construction, the systems to be used, and the tasks to be performed by those Quality Assurance firms and individuals.

3.2 Evaluation of Quality Assurance Agencies

The firm or agency responsible for the Quality Assurance function, whether an independent testing laboratory or an individual, must be capable of performing the work, and should be evaluated and approved by the Owner, with the assistance of the Engineer or other qualified designated design professional, regarding the capability to perform such functions. The Building Official may also have requirements regarding the qualification of firms or individuals who will be performing the work. The level of expertise and experience necessary to perform such functions depends upon the specific role of the Inspector, namely as a Special Inspector, Welding Inspector, or NDT (nondestructive testing) technician.

3.3 Quality Assurance Agency Qualifications

The submittal of the Quality Assurance Agency's qualifications should be required. The submittal should include:

- qualifications of QA Agency's management and QA personnel designated for the project,
- QA Agency's written inspection procedures manual, and operations Quality Control manual (Written Practice),
- qualification records for Inspector and NDT technicians designated for the project, and
- QA Agency's NDT procedures, equipment calibration records, and personnel training records.

The review of QA Agency qualifications should be performed in a timely manner.

Each agency performing welding-related Quality Assurance should employ an individual qualified as an ASNT NDT Level III for the type of Nondestructive Testing to be performed. This individual should specify and oversee equipment calibration, inspection procedures, personnel certification, and training for the project.

3.3.1 Testing Laboratory Accreditation Programs

There are several accreditation programs available to independent testing laboratories to demonstrate, to third parties, the expertise of the QA Agencies in a particular field. Such programs include:

3.3.1.1 National Voluntary Laboratory Accreditation Program (NVLAP)

NVLAP is administered by the National Institute of Standards and Technology (NIST), and accredits public and private laboratories based upon evaluation of their technical qualifications and competence, following *ISO/IEC Guide 25, General Requirements for Accreditation of Laboratories*, and *ISO 9002*. Laboratories are reviewed in the following areas: accommodation and environment, calibration and test methods, certificates and reports, complaints, equipment and reference material, handling of calibration and test items, measurement traceability and calibration, organization and management, outside support services, personnel, quality system, audit and review, records, and subcontracting. Fields of accreditation include Construction Material Testing, and Fasteners and Metals.

Commentary: Fewer than 20 agencies are currently accredited for Construction Material under the NVLAP.

3.3.1.2 American Association for Laboratory Accreditation (A2LA)

A2LA is a membership organization that offers accreditation in a variety of fields, including Construction Material and Nondestructive Testing. A2LA follows *ISO/IEC Guide 25* in its accreditation process.

Commentary: Approximately 100 agencies are accredited for Construction Material under A2LA.

3.3.1.3 International Conference of Building Officials Evaluation Services, Inc. (ICBO ES)

ICBO ES conducts evaluations and issues reports on a variety of material and services, including Testing Laboratories (268). Specific areas of testing are listed in bi-monthly Evaluation Reports listings. However, published reports for Testing Laboratories are not made available.

3.3.1.4 Building Officials and Code Administrators Evaluation Services, Inc. (BOCA-ES)

BOCA-ES conducts evaluations and issues reports on a variety of material and services, including Testing Laboratories (Section 01410) and Inspection Services (Section 01420). Testing Laboratories are typically facilities performing product testing such as physical testing or fire testing. Inspection Services are typically firms providing product manufacturers with QC services, although some firms offer jobsite services.

3.3.1.5 Southern Building Code Congress International, Public Safety Testing, and Evaluation Services, Inc. (SBCCI PST & ESI)

SBCCI PST & ESI conducts evaluations and issues reports on a variety of materials and services, including “Testing Laboratories” and “Quality Assurance and/or Inspection Agencies.” Inspection Services are typically firms providing product manufacturers with QC services, although some firms offer jobsite services.

3.3.1.6 National Evaluation Service (NES)

NES, operated jointly by BOCA and SBCCI, conducts evaluations and issues reports on a variety of materials and services, including “Testing Laboratories” and “Compliance Assurance and/or Inspection Agencies”. National Evaluation Reports are identified as NER-TL for a “Testing Laboratory”, and NER-QA for a “Compliance Assurance and/or Inspection Agency”. Inspection Services are typically firms providing product manufacturers with QC services, although some firms offer jobsite services.

3.3.1.7 American Welding Society (AWS)

The AWS offers an accreditation program for testing laboratories that wish to conduct AWS Certified Welder Program tests. AWS does not evaluate inspection services beyond the scope of welding personnel testing.

3.3.1.8 American Council of Independent Laboratories (ACIL)

ACIL is a membership organization of approximately 350 firms in various disciplines, operating approximately 1500 facilities. No accreditation program is currently offered.

3.3.2 Alternatives to Certification

The Owner and Owner’s representative may refer to the following ASTM standards for guidelines and assistance in evaluating inspection and testing agencies.

- ASTM E329 - *Standard Specification for Agencies Engaged in the Testing and/or Inspection of Material Used in Construction*
- ASTM E543 - *Standard Practice for Agencies Performing Nondestructive Testing*
- ASTM E548 - *Standard Guide for General Criteria Used for Evaluating Laboratory Competence*
- ASTM E994 - *Standard Guide for Laboratory Accreditation Systems*
- ASTM E1212 - *Standard Practice for Establishment and Maintenance of Quality Control Systems for Nondestructive Testing Agencies*
- ASTM E1359 - *Standard Guide for Surveying Nondestructive Testing Agencies*

Figure 3-1 is a checklist that may be used in the evaluation of Quality Assurance agencies.

3.4 Written Practice

3.4.1 Written Practice for Quality Assurance Agencies

The Quality Assurance Agency should maintain a Written Practice for the selection and administration of inspection personnel. The Agency's Written Practice should describe the training, experience and examination requirements for qualification and certification of inspection personnel.

The Written Practice should describe the Agency's procedures for determining the acceptability of the structure in accordance with the applicable codes, standards, specifications and procedures, including general inspection, material controls, visual welding inspection, and bolting inspection.

3.4.2 Written Practice for Nondestructive Testing Agencies

The NDT Agency should maintain a Written Practice for the control and administration of NDT personnel training, examination and certification. The Agency's Written Practice should describe the training, experience and examination requirements for each level of certification. The Agency's written practice should describe the responsibility of each level of certification for determining the acceptability of material and weldments in accordance with the applicable codes, standards, specifications and procedures.

Commentary - The NDT Agency's Written Practice should be based upon either of two documents: ASNT Recommended Practice No. SNT-TC-1A (1996), Personnel Qualification and Certification in Nondestructive Testing; or ANSI/ASNT CP-189-95, ASNT Standard for Qualification and Certification of Nondestructive Testing Personnel.

3.5 Qualification of Inspection Personnel

Inspectors responsible for acceptance or rejection of material and workmanship shall be qualified to perform such inspection whether acting in a QC or QA role. The necessary level of expertise of the individual inspector depends upon the specific inspection role assigned and the complexity of the project. Inspectors are selected by the Agency responsible for the work, although the Engineer has the authority to verify the qualification of welding inspectors (AWS D1.1 Section 6.1.4.5), and the Building Official has the right of approval of inspectors (IBC Section 1704.1).

The Inspector may use assistants who are formally designated, made aware of their assigned responsibility and the acceptance criteria, and work under the direct supervision and monitoring of a qualified Inspector.

General Welding Inspector qualifications are defined by AWS D1.1 in Section 6.1.4. For nondestructive testing, ANSI/ASNT CP-189 defines the education, skills, training, knowledge, and experience required for personnel to perform properly to a specified NDT Level. ASNT SNT-TC-1A defines the demonstrated skill, demonstrated knowledge, documented training, and documented experience required for personnel to perform properly the duties of a specific job.

AWS defines certification as the act of determining, verifying, and attesting in writing to the qualification of personnel in accordance with specified requirements. ASNT defines certification in ANSI/ASNT CP-189 as written testimony that an individual has met the applicable requirements of the standard; ASNT defines certification in ASNT SNT-TC-1A as written testimony of qualification.

3.5.1 Special Inspector Qualification and Certification

The requirements for qualification of the Special Inspector must be established by the organization responsible for the Special Inspection, whether the Owner or the Owner's representative. The qualifications and selection of the Special Inspector must be approved by the Building Official, as a part of the issuance of the building permit (*IBC* Section 1704.1.1). The Special Inspector may be either an individual or a firm. If a firm, the individuals performing the Special Inspection function should meet the minimum qualifications to perform such work as established by the Quality Assurance Plan.

The Owner, the Owner's designated representative, or the Building Official may require the Special Inspector to maintain certification by an outside agency, such as International Conference of Building Officials/International Code Conference (ICBO/ICC), American Welding Society (AWS), or another independent accreditation agency.

3.5.1.1 ICBO Certified Special Inspector for Structural Steel and Welding

ICBO maintains a certification program for inspectors of steel-framed structures. This program consists of a test of an inspector's knowledge of the steel and special inspection related sections of the *Uniform Building Code*, *AISC Specification for Structural Steel Buildings*, *AISC Manual of Steel Construction*, *AWS D1.1, D1.3 and D1.4 Structural Welding Codes*, *AWS A2.4 Standard Symbols for Welding, Brazing and Nondestructive Testing*, and the inspector's ability to read and interpret structural steel erection drawings. No experience is required to take the test or to receive certification.

Commentary: Building Officials may elect to take certification exams relative to their respective model building code. Such exams cover the broad extent of the entire code, and may not deal with the specific provisions and inspection procedures or issues directly related to steel construction. Additional education and training relative to steel construction and inspection is generally needed.

3.5.2 Welding Inspector Qualification and Certification

Visual welding inspection personnel shall be qualified in accordance with *AWS D1.1*, Section 6.1.4. The basis of qualification should be specified in the Quality Assurance Plan. Acceptable qualification bases under *AWS D1.1* are:

- current or previous certification as an AWS Certified Welding Inspector (CWI) in accordance with the provisions of *AWS QC1, Standard for AWS Certification of Welding Inspectors*, or
- current or previous qualification by the Canadian Welding Bureau (CWB) to the requirements of the Canadian Standard Association (CSA) Standard W178.2, *Certification of Welding Inspectors*, or
- an engineer or technician who, by training or experience, or both, in metals fabrication, inspection and testing, is competent to perform inspection work.

The qualification of an inspector remains in effect indefinitely, provided the inspector remains active in the inspection of welded steel fabrication, or unless there is a specific reason to question the inspector's ability.

Although AWS D1.1 allows inspector qualification without the CWI certification, it is recommended that the welding inspection personnel for critical welding be AWS QC1 certified (or previously certified) by experience and written examination.

All Welding Inspectors must have adequate visual acuity, documented by vision testing performed within the past three years. See AWS D1.1, Section 6.1.4.4.

Commentary: The American Welding Society offers certification to welding inspectors in the form of Certified Welding Inspectors, Certified Associate Welding Inspectors, and Certified Senior Welding Inspectors. ANSI/AWS QC-1-96, Standard for AWS Certification of Welding Inspectors, governs the requirements and testing of such inspectors, including experience level. The CWI examination tests the inspector's knowledge of welding processes, welding procedures, welder qualification, destructive testing, nondestructive testing, terms, definitions, symbols, reports, records, safety and responsibilities. Although assumed to be competent to inspect welded construction, the AWS Certified Welding Inspector may not have the background or expertise in other areas of steel construction such as general fabrication and erection, bolted connections, steel bar joists, and metal decks, and additional education and training relative to these areas may be needed to perform inspections of those elements.

It should also be verified that the AWS Certified Welding Inspector is familiar with the AWS D1.1 Structural Welding Code. It is permitted to take the AWS examinations using AWS D1.1, American Petroleum Institute (API) 1104 Welding of Pipelines and Related Facilities, or American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code, and the required welding inspection experience may be in any area of welding.

AWS D1.1 does not recognize the AWS Certified Associate Welding Inspector (CAWI) as qualified to perform the work solely based upon this certification. Typically, a CAWI has passed the same accreditation examination as the CWI, but has less experience, with two years minimum experience rather than five years, in the field of welding inspection. A CAWI may also have the required experience, but may not have achieved the test score required for CWI status, but adequate for CAWI status. A CAWI could be considered qualified under the third condition listed above. The Senior Certified Welding Inspector is a relatively new program offered by the AWS, and this recent certification option has not been specifically included in the D1.1 code. A SCWI should at present be considered the equivalent of a CWI.

3.5.3 NDT Personnel Qualification and Certification

Certification of all levels of NDT personnel is the responsibility of the employer of the NDT technician. The certification of NDT personnel should be administered by an ASNT NDT Level III in the specific area of NDT.

Commentary: The term “ASNT NDT Level III” identifies individuals who have taken and passed a specific examination administered by ASNT. It is also possible to be “self-certified” as an NDT Level III under the SNT-TC-1A Recommended Practice. Such individuals are not permitted to use the acronym “ASNT” prior to “NDT Level III”.

Nondestructive testing personnel should be qualified under the American Society for Nondestructive Testing, *ANSI/ASNT CP-189, ASNT Standard for Qualification and Certification of Nondestructive Testing Personnel*, or *ASNT Recommended Practice No. SNT-TC-1A, Personnel Qualification and Certification in Nondestructive Testing*.

Certification of NDT personnel should be based on demonstration of satisfactory qualification in accordance with Sections 4, 5 and 6 of *ANSI/ASNT CP-189*, as modified by the employer's Written Practice, or in accordance with Sections 6, 7 and 8 of *ASNT SNT-TC-1A*. Personnel certifications must be maintained on file by the employer and a copy should be carried by the technician.

Employers may rely upon outside training and testing for NDT personnel for certification, however, the employer should supplement such certification testing with a review of the technician's experience and skill levels. As an example, the technician may have successfully passed the NDT Level II examination for that NDT process as administered by either the AWS or ASNT.

3.5.3.1 NDT Level Requirements

AWS D1.1, Section 6.14.6 requires that nondestructive testing be performed by NDT Level II technicians, or by NDT Level I technicians only when working under the direct supervision of an NDT Level II technician.

Commentary: Inspection by an NDT Level III technician is not recognized, as the NDT Level III may not perform actual testing regularly enough to maintain the special skills required to set up or to conduct the tests. AWS D1.5-96 requires similar qualification, except in the case of Fracture Critical Members. Under AWS D1.5 Section 12.16.1.2, testing of fracture critical members must be done by either a qualified NDT Level II under the supervision of a qualified NDT Level III, or by an ASNT NDT Level III (certified by ASNT examination), unless the Engineer accepts other forms of qualification.

3.5.3.2 Levels of NDT Qualification

ASNT identifies four qualification levels, as defined in *ANSI/ASNT CP-189*:

3.5.3.2.1 Trainee

A person who is not yet certified to any level. Trainees must work with a certified person, under the direction of an NDT Level II or NDT Level III, and should not independently conduct any tests or write a report of test results.

3.5.3.2.2 NDT Level I

An NDT Level I individual should have the skills to properly perform specific calibrations, specific NDT, and with prior written approval of the NDT Level III, perform specific interpretations and evaluations for acceptance or rejection and document the results. The NDT Level I should be able to follow approved nondestructive testing procedures and should receive the necessary guidance or supervision from a certified NDT Level II or NDT Level III individual.

3.5.3.2.3 NDT Level II

An NDT Level II individual should have the skills and knowledge to set up and calibrate equipment, to conduct tests, and to interpret, evaluate, and document results in accordance with procedures approved by an NDT Level III. The Level II should be thoroughly familiar with the scope and limitations of the method to which certified and should be capable of directing the work of trainees and NDT Level I personnel. The NDT Level II should be able to organize and report nondestructive test results.

Commentary: The NDT Level II technician may have taken written and practical tests as administered directly by either the ASNT or the AWS. Such independent testing of the NDT Level II technician is not required of a Level II.

3.5.3.2.4 NDT Level III

An NDT Level III individual should have the skills and knowledge: to establish techniques; to interpret codes, standards, and specifications; to designate the particular technique to be used; and to verify the accuracy of procedures. The individual should also have general familiarity with the other NDT methods. The NDT Level III should be capable of conducting or directing the training and examining of NDT personnel in the methods for which the NDT Level III is qualified.

Commentary: Additional provisions from SNT-TC-1A include: The NDT Level III should be responsible for the NDT operations for which qualified and assigned and should be capable of interpreting and evaluating results in terms of existing codes, standards, and specifications. The NDT Level III should have sufficient practical background in applicable material, fabrication, and product technology to establish techniques and to assist in establishing acceptance criteria when none are otherwise available.

The NDT Level III technician may have taken written and practical tests administered directly by either the ASNT or the AWS. Such independent testing of the NDT Level III technician is not required for an NDT Level III under SNT-TC-1A. Such independent testing of NDT Level III technicians is highly recommended. It is required under CP-189.

3.5.3.3 Recertification of NDT Personnel

Under *ANSI/ASNT CP-189*, NDT Level I and II personnel must be recertified by written and practical examination in accordance with Section 6. Individual certifications expire at the end of three years. NDT Level III personnel must be recertified by the employer every five years by verifying that the individual's ASNT NDT Level III certificate is current in each method for which recertification is sought. Should the ASNT NDT Level III certificate expire prior to the five years, the individual must retest for that certificate.

Under *ASNT SNT-TC-1A*, all levels of NDT personnel should be recertified periodically, with evidence of continuing satisfactory performance, or by re-examination in those portions of the examinations in Section 8 deemed necessary by the employer's NDT Level III. The maximum recommended recertification intervals are three years for Levels I and II, and five years for Level III.

The employer's Written Practice should include rules covering the duration of an interruption to service that triggers re-examination and recertification.

3.5.3.4 Suspension of Certification

The employer should suspend an individual's certification if:

- the vision examination interval exceeds one year, with certification reinstated concurrently with passing the vision examination, or
- the individual has not performed the duties in the methods for which certified during the previous twelve months, or
- the individual's performance is determined to be deficient in the method or technique for specific documented reasons, or
- for ASNT NDT Level III personnel, when the ASNT Level III certificate has not been renewed.

3.5.3.5 Revocation of Certification

The employer should revoke an individual's certification when:

- the individual has not performed the duties in the methods for which certified within the past 24 months, or
- for NDT Level III personnel, the ASNT NDT Level III certificate has been revoked, or

- an individual's conduct is deemed by the employer to be, or to have been, unethical or incompetent.

3.6 Inspection Tasks – General

The Quality Assurance Plan should clearly define the roles and responsibilities of Quality Control inspectors, Quality Assurance inspectors, and the other individuals involved in the QA function.

Commentary: The definitions of these roles can directly affect the project structure and associated budgets. To alleviate this dilemma, the Owner must tightly specify the requirements for both the QC and QA programs.

Depending upon the structure of the QC and QA functions for the particular project, the role of the visual inspector may vary considerably. Ideally, the QC Inspector is an employee of the Contractor and answers to a department head who is not connected with production. If this is not the case, an inherent conflict of interest may be present.

The level of involvement required of the QA Agency is highly dependent upon the requirements for the Contractor's QC program. If the Contractor's QC Plan is well organized, has competent inspection and testing personnel, and will be truly independent of production, the outside QA function can be designated to operate as an overseer where random spot inspection and testing will be sufficient. In the opposite case, where the QC department is permitted to be less competent or is managed by production, more thorough QA may be appropriate. In any case, QA must meet the minimum requirements of the building code.

3.7 Special Inspector Tasks

3.7.1 Quality Assurance Plan

The specific tasks of the Special Inspector should be defined in the Quality Assurance Plan and Contract Documents. The Special Inspector may be assigned all QA responsibilities, or may be assigned the task of supervising or monitoring QA performed by specific Welding Inspectors, NDT technicians or other Inspectors. Special Inspection may also include monitoring QC inspection performed by the Contractor.

Commentary: See IBC Section 1704 for specific tasks assigned to the Special Inspector.

3.7.2 Material

The Special Inspector should ascertain that all materials comply with the Contract Documents, by mill and product certifications, by testing, or by both.

3.7.3 Contractor's QC Program

The Special Inspector may be assigned the task of evaluating the Contractor's QC program and its effectiveness. If so assigned, the Special Inspector should verify that the fabricator is properly implementing the fabrication and Quality Control procedures outlined.

Requirements for Special Inspection of the fabricator's Quality Control Program are waived when the fabricator maintains an agreement with an approved independent inspection or quality control agency to conduct periodic in-plant inspections at the fabricator's plant, at a frequency that will ensure the fabricator's conformance to the requirements of the inspection agency's approved Quality Control program.

3.7.4 Inspector Qualifications

The Special Inspector should verify the qualifications of the QC and QA Inspectors and the NDT technicians.

3.7.5 Steel Frame

The Special Inspector should inspect the steel frame to verify compliance with the details shown on the approved construction documents, such as bracing, stiffening, and member locations, and the joint details at each connection.

3.7.6 Welding and Bolting

The Special Inspector should verify that all fabrication and erection, including welding and bolting, is performed in accordance with the Contract Documents.

3.7.7 Records

All individuals performing Special Inspection, regardless of classification, should keep written records of the inspections performed. Copies of these records should be distributed to the parties designated in the Quality Assurance Plan. Documents reporting nonconformance should be distributed to the Owner, Engineer, Contractor and Building Official as specified in the Plan. Records documenting correction of nonconformance should be distributed to the same parties as those receiving reports of nonconformance.

3.7.8 Reports

Upon completion of the Special Inspector's tasks, the Special Inspector shall issue a written report stating that all work inspected under the Inspector's charge is in accordance with the approved Construction Documents and meets the applicable codes and specifications, and that instances of nonconformance have been corrected to the satisfaction of the Engineer or other persons in responsible charge.

Quality Assurance Agency Evaluation Checklist

Note: This list is adapted from ISO/IEC Guide 25-1990, "General Requirements for the Competence of Calibration and Testing Laboratories," and ASTM E1359, "Standard Guide for Surveying Nondestructive Testing Agencies."

Organization and Management

1. Does the Agency have managerial staff with the authority and resources needed to discharge their duties?
2. Does the Agency have arrangements to ensure that its personnel are free from any commercial, financial and other pressures that might adversely affect the quality of their work?
3. Is the Agency organized in such a way that confidence in its independence of judgement and integrity is maintained at all times?
4. Does the Agency specify and document the responsibility, authority, and interrelation of all personnel who manage, perform or verify work affecting the quality of inspections and tests?
5. Does the agency provide supervision by persons familiar with the inspection disciplines in which it employs inspectors?
6. Does the Agency provide supervision by persons familiar with the test methods and procedures, the objectives of the testing and the assessment of the results?
7. Does the Agency have a technical manager who has overall responsibility for the technical operations?
8. Does the Agency have a quality manager who has responsibility for the quality system and its implementation?
9. Is the quality system reviewed at least once a year by the management to ensure its continuing suitability and effectiveness?
10. When the Agency procures outside services and supplies, does the Agency use only those outside support services and supplies that are of adequate quality?

Page 1

Figure 3-1 Quality Assurance Agency Evaluation Checklist

Quality Assurance Agency Evaluation Checklist

Quality Control Manual

1. Does the Agency maintain a Quality Control Manual containing, as a minimum, the following:
 - a. a quality policy statement, including objectives and commitments, by top management,
 - b. the organization and management structure of the Agency,
 - c. the relationship between management, technical operations, support services and the quality system,
 - d. procedures for control and maintenance of documentation,
 - e. job descriptions of key staff and reference to the job descriptions of other staff,
 - f. identification of the Agency's approved signatories,
 - g. the Agency scope of laboratory and field tests,
 - h. the Agency's scope of inspection services,
 - i. arrangements for ensuring that the Agency reviews all new work to ensure that the Agency has the appropriate facilities and resources before commencing such work,
 - j. reference to the test procedures used,
 - k. reference to procedures for calibration, verification and maintenance of equipment,
 - l. list of codes and standards maintained for reference by the Agency, and
 - m. procedures for audit and review?

Page 2

Figure 3-1 Quality Assurance Agency Evaluation Checklist (continued)

Quality Assurance Agency Evaluation Checklist

Personnel

1. Does the Agency have sufficient personnel, with the necessary education, training, technical knowledge and experience for their assigned functions?
2. Are records on the relevant qualifications, training, skills and experience of the technical personnel maintained by the laboratory?
3. Is there a written practice for personnel certification?
4. Is there a training program described in the Agency's written practice?
5. Are all personnel certified by examination for all levels of certification in each discipline for which they perform inspection or testing?
6. Are eye examinations required for all inspection and NDT personnel?
7. Are all personnel certifications available for review?
8. Are the minimum American Society for Nondestructive Testing recommendations of ASNT-TC-1A met for all NDT personnel for:
 - a. training hours,
 - b. experience, and
 - c. test questions?
9. Are outside agencies used to train and examine NDT personnel?

Facilities and Equipment

1. Are the facilities, equipment, instructions, and specifications adequate for the type of work to be performed?
2. Are adequate procedures in effect to control the maintenance, calibration, and use of equipment including applicable tools, gages, and other instrumentation?
3. Is equipment (tools, gages, and other instruments) identified to reflect:
 - a. date last calibrated,
 - b. date of next calibration, and
 - c. identification or serial number?

Figure 3-1 Quality Assurance Agency Evaluation Checklist (continued)

Quality Assurance Agency Evaluation Checklist

Project Administration

1. Are contract requirements reviewed to ensure NDT specification compliance with applicable codes, standards and NDT specifications?
2. Are NDT specification or contract requirements passed on to NDT Level I and II personnel by written procedures and instructions approved by an NDT Level III?
3. Are inspection and test results documented and reported as required?
4. Does the Agency have a documented system for uniquely identifying the work to be tested?
5. Are the results of each test, or series of tests, reported accurately, clearly, unambiguously and objectively?
6. Does the Agency have experience with work of similar size and complexity?
7. Does the Agency have sufficient staff with relevant experience and qualifications to meet the needs of the project?
8. Does the Agency have the ability to provide inspection services at offsite fabrication facilities?

Page 4

Figure 3-1 Quality Assurance Agency Evaluation Checklist (continued)

Part II

4. STRUCTURAL STEEL

4.1 Specification of Material

The Contract Documents should specify the American Society for Testing and Materials (ASTM) specification and grade of steel used for all structural steel elements of the building. Any Supplementary Requirements to the ASTM steel specifications, including identification of the members of the steel frame subjected to such requirements, should be clearly defined in the construction documents.

4.2 Submittals

The Contract Documents should contain requirements for the submittal of certified material test reports for all main members of the steel frame. These reports should identify the manufacturer, size, heat number, chemistry, yield strength, and ultimate tensile strength, plus any other properties required. These properties may include Charpy V-Notch toughness or other Supplementary Requirements listed in the Contract Documents and supplied by the mill. Main members include beams, girders, columns and bracing elements.

Mill test reports for material used in the connections of the seismic-force-resisting system, including web doubler plates, stiffeners (continuity plates), and beam web connection material should be submitted to, and should be reviewed by, the designated inspector. Test reports for other connection material such as clip angles, single plate framing connections (shear tabs), gusset plates, beam web stiffeners, need not be submitted. Material used by the fabricator for such connection material may be drawn from fabricator stock of known grade, provided the stock has had material test reports previously received and reviewed by the fabricator.

4.3 Material Control

The Contract Documents should specify requirements for fabrication material control, if controls beyond the specification of the grade of steel are required.

The Fabricator should establish written procedures for fabrication material control. The minimum acceptable level of control is for steel grade identification.

4.4 Quality Control - Structural Steel

The Fabricator's Quality Control Plan for material quality control should include, as a minimum:

- procedures for visual inspection of steel material upon receipt,
- procedures for verification of heat numbers and material test reports upon receipt,
- an adequate system of material control by grade, and also by heat number when required,

- review procedures for material test reports for project requirements, and
- a filing system for material test reports for all structural steel.

A sample checklist form is provided in Figure 4-1 to assist in the development of a suitable document for use in a fabricator's QC activities.

4.5 Extent of Steel Inspection

The Quality Assurance Plan and Contract Documents should clearly identify the extent of steel material inspection to be performed by the Contractor and by the Inspector or NDT Technician. QA mill inspection, prefabrication steel inspection, testing, or verification of chemistry or mechanical properties, if required, must be designated in the Contract Documents. Steel material inspection is generally limited to visual inspection of the surface condition of the steel, as well as visual inspection of thermal cut surfaces for indications of laminations. Requirements for additional physical sampling and testing are discouraged.

4.6 Structural Steel Inspection Tasks

Prior to fabrication, the Inspector should review submitted certified material test reports.

The QA Inspector, whether a Special Inspector or other Inspector, should verify that the Contractor follows the prescribed Quality Control Plan for structural steel. Verification of the system for maintaining steel grade and heat number identification, when required, should be performed. This includes random spot checks verifying material controls by grade identification, and specific verification, including traceability when required, of material grade and any other special requirements for those elements of the structure identified by the Quality Assurance Plan as needing QA verification.

Structural Steel Material Inspection Checklist

Project _____

Inspector Name _____ Date of Inspection ____/____/____

Producing Mill _____ Member Size _____

ASTM Grade Identification

- ☐ A36
- ☐ A992
- ☐ A572, grade _____
- ☐ A913, grade _____
- ☐ Other _____

Heat Number _____ (Required for specified main members only)

Piece Mark(s) _____

- ☐ ASTM A6 tolerances satisfied
 - ☐ Straightness (camber, sweep)
 - ☐ Flanges out-of-square
 - ☐ Member depth
 - ☐ Flange tilt
 - ☐ Web off-center

Description and location of any unacceptable conditions _____

- ☐ Surface quality adequate – seams, laps, tears, nicks

Description and location of any surface discontinuities _____

If UT of section is required for laminations check, prior to assembly or use, provide UT inspection report.

Number _____

Page 1

Figure 4-1 Structural Steel Material Inspection

Structural Steel Material Inspection Checklist

MATERIAL TEST REPORT (MTR) REVIEW

Inspector Name _____ Date of MTR Review ____/____/____

- ☐ Material test report on file
- ☐ Material test report satisfactory
- ☐ Yield Strength
- ☐ Tensile strength
- ☐ Elongation
- ☐ Chemical analysis
- ☐ Supplementary requirements (if required)
- ☐ Technical Bulletin #3 compliance (A572, grade 50 only)
- ☐ Maximum Tensile Strength (S18)
- ☐ Carbon equivalent (weldability)(S74)
- ☐ CVN toughness (S5)
- ☐ Ultrasonic Examination (S8)
- ☐ Maximum Yield Point and Yield-to-Tensile Ratio (S75)
- ☐ Charpy V-Notch Impact Test for Group 4 and 5 Structural Shapes – All Grades (S76)
- ☐ Fine austenitic grain size (S91)
- ☐ Other _____

(On the back of this form, or below, provide information on unacceptable ASTM A6 tolerances, and a description of unacceptable surface defects and acceptance levels.)

Page 2

Figure 4-1 Structural Steel Material Inspection (continued)

Part II

5. WELDING

5.1 Specifications

5.1.1 Welding Material

All electrodes, fluxes and shielding gases should conform to the latest American Welding Society (AWS) A5-series specifications.

The Contract Documents should specify the appropriate AWS specifications for welding material to be used. Any special requirements, including identification of the joints of the steel frame subjected to such requirements, must be clearly defined in the Contract Documents. Such requirements may include control of CVN toughness, diffusible hydrogen levels, and additional manufacturer or contractor testing. Where specified CVN toughness for filler metal is required for designated joints, the minimum CVN toughness value(s), testing temperature(s), and special testing parameters if required, shall be specified.

5.1.2 Weld Acceptance Criteria

The Contract Documents should establish the level of welding quality required for the project. Quality requirements for members of the seismic-force-resisting system may be different from the requirements for other members. Such criteria may be those established in *AWS D1.1*, or other suitable criteria as suggested in the technical literature. The use of acceptance criteria other than that prescribed in *AWS D1.1* is permitted by *AWS D1.1*, Section 6.8.

Welds executed by welding personnel who do not satisfy the personnel performance qualifications, and welds not executed in conformance with the approved Welding Procedure Specification (WPS), may be considered rejectable and require evaluation by a qualified Engineer or welding consultant.

5.2 Submittals

5.2.1 Welding Material

The Contract Documents should require the submittal of certifications for all welding material, including electrodes, fluxes and shielding gases. The manufacturer's certification should state that the welding electrodes, fluxes and shielding gases, as supplied, meet the applicable AWS A5 specifications. The manufacturer's data sheets for the welding materials should also be submitted. A copy should be provided to the appropriate Inspectors for review.

Commentary: Manufacturer's certifications are for compliance with the appropriate AWS A5 Specification, and are not a material test report taken from the specific lot of material being used. Such lot-specific tests are not required.

5.2.1.1 Supplemental CVN Toughness Verification Tests

The Contract Documents should require the submittal of results of specified tests conducted to verify the CVN toughness achieved using the filler metal selected for joints with special toughness requirements. The test protocol should follow that specified in Part I of these *Recommendations* for such purpose. Tests may be conducted by the filler metal manufacturer, contractor, or by a third party laboratory or agency. Testing by production lot is required unless a manufacturer testing and certification program acceptable to the Engineer is in use for the filler metal.

5.2.2 Welding Procedure Specifications

The Contract Documents should require the submittal of Welding Procedure Specifications (WPSs) for each weld to be made on the project. For shop welds, the Fabricator is responsible for developing and supplying the WPSs. For field welds, the Erector is responsible for developing and supplying the WPSs. The WPSs may also be developed and prepared by other competent sources, but remain the responsibility of the Fabricator or Erector.

Procedure Qualification Records (PQRs) shall be supplied for all non-prequalified WPSs.

The Contract Documents should require submittal of the WPSs to the Engineer for review. The WPSs should also be distributed to the Inspectors responsible for welding inspection.

The Engineer should review the WPSs for general suitability and adequacy, but need not take responsibility for the suitability or quality of the welds made using such procedures. Such responsibility remains with the Contractor responsible for the welding.

Commentary: WPSs, as with other Contractor-developed documents such as shop drawings, erection drawings and product data, should be reviewed by the Engineer to confirm conformance with the Contract Documents and design intent. Historically, Engineers and other design professionals routinely reviewed and approved Contractor submittals and recent editions of AISC and AWS specifications and codes have required that the Engineer approve such submittals. For example:

- *AISC Seismic Provisions for Structural Steel Buildings, Section 7.3a states:*
“Welding shall be performed in accordance with a Welding Procedure Specification (WPS) as required in AWS D1.1 and approved by the Engineer of Record.”
- *AWS D1.1, Section 6.3.1 (1998 edition) states:*
“The Inspector shall verify that all WPSs have been approved by the Engineer in accordance with 4.1.1.”

- AWS D1.1, Section 4.1.1 states:

“Except for prequalified WPSs, a WPS for use in production welding shall be qualified ... and approved by the Engineer.”

However, over the past 20 years, there has been an evolution in design practice with design professionals refusing to approve submittals, and instead, reviewing submittals for general conformance and either taking exception or not. The 2000 edition of AWS D1.1 acknowledges this changing practice and in Section 6.3.1, replacing the earlier Section 6.3.1, states:

“The Inspector shall review all WPSs to be used in the work and shall make certain that the procedures conform to the requirements of this code.”

This revision is not intended to preclude critical Contractor submittals, such as WPSs to the design professional, but rather is intended to acknowledge that the Engineer’s review of such submittals does not constitute an approval.

5.2.3 Other Submittals

The Contractor should maintain at the jobsite and make available to the Engineer, Inspector and NDT Technician all drawings, project specifications, material certifications, welder qualification records, WPSs and PQRs applicable to the project.

Commentary: As used in this section, jobsite applies both to the fabrication shop while fabrication work is being performed, and the construction site during erection.

5.3 Material Control

Identification of welding material should be maintained. Control should be exercised to verify that welding material with adequate notch toughness is used. Special welding material control requirements such as lot traceability are not required for standard welding material. Lot control may be advisable for welding material with special notch toughness requirements, should significant variations between lots, as established by tests, result in samples failing to achieve the required toughness.

Commentary: Manufacturer certifications document the overall quality of all welding material supplied, and variation from lot to lot is generally not significant.

5.3.1 Storage

The Contract Documents should specify special storage or material control requirements, if necessary. Welding material storage controls, such as requiring rod storage ovens and having limited atmospheric exposure times for SMAW low-hydrogen electrodes, must be as prescribed

in AWS D1.1, Section 5. Supplemental storage requirements for FCAW electrodes may be advisable for certain conditions and electrodes as well.

5.4 Quality Control and Quality Assurance

The Quality Assurance Plan should contain a list of all welding inspections required and designate the party responsible for each task. Inspections may be provided by the Contractor (the Fabricator or the Erector), by the Special Inspector, by the Welding Inspector, or by the Nondestructive Testing Technician.

The Contract Documents should specify the type and location of any joints requiring NDT other than visual. The extent of NDT, whether full length, partial length, or spot testing should be clearly stated. See AWS D1.1, Section 6.15.

In the series of design criteria and specification documents developed under this project, Quality Control and Quality Assurance tasks are recommended depending on the severity of loading (Seismic Weld Demand Categories) on a weld and the consequences of adverse welded joint performance (Seismic Weld Consequence Categories). Figure 5-1 provides a recommended checklist for prefabrication inspection for welding, applicable to all elements of the seismic force resisting system. Table 5-1 defines three Seismic Weld Demand Categories and provides guidance as to which welded joints should be assigned to each category. Table 5-2 defines three Seismic Weld Consequence Categories and provides guidance as to which consequence category various welded joints should be assigned. Table 5-3 assigns each type of welded joint to a Process and Visual Inspection class, depending on the Seismic Weld Demand Category and Seismic Weld Consequence Category for the joint. Table 5-4 provides recommendations for Nondestructive Testing for each class of welded joint. Table 5-5 is a checklist summarizing all recommendations for process and visual inspection for the various classes.

Commentary: The basic code requirements for structural welding of steel structures are contained in AWS D1.1 - Structural Welding Code - Steel. AWS D1.1 uses the term "Fabrication/Erection Inspection" synonymously with the "Quality Control" function. The term "Verification Inspection" is used to describe the "Quality Assurance" function. AWS D1.1 requires inspection for many items in Section 6, Part A, but does not specify whether the inspection is performed as either QC or QA. Such distinction must be provided in the Contract Documents.

In these Recommendations, the extent of inspection and other Quality Control and Quality Assurance measures recommended for a specific joint is dependent on the severity of joint loading or demand and the consequences of adverse joint performance. FEMA-350 – Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings, FEMA-351 – Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings and FEMA-352 – Postearthquake Evaluation and Repair Criteria for Steel Welded Moment-Frame Buildings, indicate the required quality control and quality assurance

classes for welded joints in steel moment frames, using prequalified connections. The criteria shown in the Tables 5-1 through 5-3 may be used to select appropriate inspection classes for joints that are not prequalified under these companion publications.

5.5 Quality Control Tasks

5.5.1 Welding Quality Control Program

The Contractor's QC program for welding should, as a minimum, include:

- Welders must be qualified for the work in accordance with AWS D1.1, Section 4, Parts A and C.
- Welding personnel should be knowledgeable regarding the use of Welding Procedure Specifications (WPSs), basic workmanship provisions, and in the use of weld gages and temperature measurement devices such as temperature indicating crayons.
- Suitable WPSs must be provided for all welding to be performed, and must be available to welding personnel at the place of work.
- Welding equipment must be adequate and functioning properly, and calibrated for accuracy within the past year.
- Welding material storage equipment such as rod ovens and flux ovens must be available and functional, if required by the process, the filler metal or the flux.
- Welding inspectors must be qualified in accordance with AWS D1.1, Section 6.1.4 to perform QC inspection.
- If not provided by the Contractor's QC personnel, outside NDT services must be available to the Contractor on an as-needed basis.

Table 5-5 provides recommendations for QC in-process inspections for a variety of welded joint classifications.

Commentary: The Contractor should require and ensure that welders meet established minimum requirements. Execution of critical welds requires skilled welders who will follow the project welding requirements.

5.6 Extent of Welding Inspection and Nondestructive Testing

The Quality Assurance Plan, the information furnished for bidding, and the Contract Documents should clearly identify the extent of inspection and Nondestructive Testing (NDT) to be performed by the Contractor and by the Quality Assurance Agency's Inspector or NDT Technician. Any approved periodic inspection should clearly be identified by type and frequency. Weld joints for which the Contract Documents require NDT should be tested for their full length, unless partial or spot testing is specified. When partial or spot testing is specified, the location and lengths of welds or categories of weld to be tested should be clearly designated in the

Contract Documents, or identified using NDT symbols in conjunction with the welding symbols on the design drawings. See *AWS D1.1*, Section 6.15.

Tables 5-1, 5-2, 5-3, 5-4, and 5-5 provide recommendations for the classification of welded joints, NDT requirements and in-process welding inspection requirements. Refer also to the Quality Control and Quality Assurance requirements specified in *FEMA-350*, *FEMA-351* and *FEMA-352* for prequalified moment-resisting connections.

On projects where a sliding sampling scale is used, the Inspector should keep records on each welder or welding operator. These records may be used as a basis for sampling rate reduction.

Commentary: See Notes to Table 5-4 for an illustration of a sampling scale method for UT inspection.

5.7 Welding Inspector Tasks

The duties of the Welding Inspector, whether designated as QC or QA, should include the following items.

- a. Review and understand the applicable portions of the specifications, the Contract Documents and the shop drawings for the project.
- b. Review material test reports for all main member and designated connection base material for compliance with the project requirements, prior to shop fabrication.
- c. Verify that all applicable welder, welding operator and tack welder qualifications are available, current and accurate.
- d. Require requalification of any welder, welding operator or tack welder who has, for a period of six months, not used the process for which the welder was qualified.
- e. Verify welder identification and qualification. Verify that any required supplemental qualification tests have been passed, and joint mock-ups, if required by the Contract Documents, have been executed.
- f. Verify that each welder has a unique identification mark or die stamp to identify his or her welds.
- g. Verify that all applicable Welding Procedure Specifications, with Procedure Qualification records as needed, are available and current.
- h. Verify that an approved Welding Procedure Specification (WPS) has been provided and that each welder performing the weld has reviewed the WPS. A copy of the appropriate WPSs should be available for each joint.
- i. Verify base material identification checks against approved shop drawings and specifications.
- j. Verify the electrode, flux and shielding gas specification sheets for compliance with the Contract Documents.
- k. Verify welding consumables comply with approved shop drawings and the approved WPSs.

- l. Verify that electrodes are used only in the permitted positions and within the welding parameters specified in the WPS.
- m. At suitable intervals, observe joint preparation, assembly practice, preheat temperatures, interpass temperatures, welding techniques, welder performance and any postweld heat treatment to ensure that the applicable requirements of the WPS and Code are satisfied.
- n. At suitable intervals, verify that the current and voltage of the welding equipment are in accordance with the WPS, if needed, by using a hand-held, calibrated, amp and volt meter. With this equipment, current and voltage should be measured near the arc.
- o. Inspect the work to ensure compliance with AWS D1.1 or the specified weld acceptance criteria. Size and contour of welds should be measured with suitable gauges. A strong light, magnifiers, or other devices, as needed, may be used to aid visual inspection.
- p. Schedule NDT technicians in a timely manner, after the visual inspection is complete and the assembly has cooled. A delay period may be required by the Contract Documents for final QA acceptance. For repair welding, the Nondestructive Testing should not be performed sooner than 24 hours after the repair welding has been completed and cooled to ambient temperature.
- q. Mark those welds, parts, and joints, that have been inspected and accepted, with a distinguishing mark or die stamp, or alternatively, maintain records indicating the specific welds inspected by each person.
- r. Document the accepted and rejected items in a written report. The report should be transmitted to the designated recipients in a timely manner.

Table 5-5 may be used as a guide for in-process welding inspection tasks. The sample checklists and forms in Figure 5-1 and Tables 5-1 through 5-5 are provided to assist in the development, understanding, and reporting of welding inspection tasks.

5.8 NDT Technician Tasks

The NDT technician should coordinate the NDT work with the Welding Inspector. The NDT technician must perform all NDT, other than visual, required by the Quality Assurance Plan, Contract Documents or Building Code. NDT should be performed in a timely manner, so as not to hinder production, and to detect welding problems soon after occurrence so that corrective measures may be taken by the Contractor to rectify such problems.

The NDT technician should mark those welds, parts, and joints, which have been inspected and accepted, with a distinguishing mark or die stamp, or alternatively, maintain records indicating the specific welds inspected.

The accepted and rejected items should be documented in a written report. The report should be transmitted to the designated recipients in a timely manner.

Checklist for Prefabrication Inspection for Welding

Project _____

Fabricator/Erector _____

Welding Systems Verification

- ☐ Welding personnel qualification records available
- ☐ Audit of welding personnel performed Date____/____/____ By _____
- ☐ List of needed WPSs prepared for project
- ☐ WPSs for project complete
- ☐ WPSs reviewed and approved by Engineer Date____/____/____ By _____
- ☐ WPSs available to welding personnel
- ☐ WPSs available to QC/QA personnel
(Note: prepare separate WPS log if needed)
- ☐ Welding equipment calibration records available
- ☐ Audit of welding equipment operation performed Date____/____/____ By _____
- ☐ Filler metal storage systems in place
- ☐ Filler metal storage systems followed
- ☐ SMAW electrode ovens
 - ☐ Temperature verified
 - ☐ Continuous operation verified
- ☐ SAW flux ovens
 - ☐ Temperature verified
 - ☐ Continuous operation verified
- ☐ FCAW wires suitably protected
- ☐ GMAW wires suitably protected
- ☐ SAW wires suitably protected

Page 1

Figure 5-1 Checklist for Prefabrication Inspection for Welding

Checklist for Prefabrication Inspection for Welding

Project Material Verification (Welding)

Welding Electrodes

Process (SMAW, FCAW, GMAW, SAW)				
Electrode class (A5 designation)				
Manufacturer				
Trade name				
Diameter				
Packaging type (for example, can, box, oven)				
Packaging condition (good, damaged, open)				
Certification papers available (yes, no)				
Certification papers reviewed (date)				

Shielding Gas

Supplier				
Type (for example, CO ₂)				
Certification papers available (yes, no)				
Certification papers reviewed (date)				

SAW Flux

Manufacturer				
Trade Name				
Type (active, neutral, alloy)				
Packaging type (for example, bag, oven)				
Packaging condition (good, damaged, open)				
Certification papers available (yes, no)				
Certification papers reviewed (date)				

Page 2

Figure 5-1 Checklist for Prefabrication Inspection for Welding (continued)

Table 5-1 Seismic Weld Demand Categories

Demand		Examples
A High	Welds in which service stresses are anticipated to be at or beyond the yield level, with some inelastic strain demand into the strain hardening region anticipated.	Beam-flange-to-column-flange CJP groove welds in: • unreinforced connections, • RBS connections, with “weak” panel zones, • free-flange connections.
B Medium	Welds in which service stresses are anticipated to be near or slightly exceed yield level, but for which negligible inelastic strain demand is anticipated.	Beam-flange-to-column-flange CJP groove welds in: • RBS connections with “strong” panel zones, • haunched connections, • cover-plated connections, • “Overlay” connections. In haunched connections, haunch-to-beam-flange and haunch-flange-to-column-flange CJP groove welds. Fillet welds in cover-plated and haunched connections. Web welds (shear tab or direct welded) in all moment connections. Doubler plate and continuity plate welds (both groove and fillet welds). CJP and PJP groove welded splices in butt joints with applied tension. In braced frames, all CJP and PJP groove welds between brace and beam or column (but not including gusset plates).
C Low	Welds in which service stresses are anticipated to remain below stresses permitted for design, or will remain in compression. Note: For LRFD, permitted design stress is defined as using (fF_w as $0.80(0.6F_{EXX})$) or lower as the design strength. For ASD, permitted design stress is defined as using an allowable stress of $0.3F_{EXX}$.	All other CJP and PJP groove welds in shear or compression, or a combination of shear and compression. All other fillet welds. All plug and slot welds.

Table 5-2 Seismic Weld Consequence Categories

H High	Steel moment frames with low redundancy (4 or fewer beams per floor resisting lateral forces in principal direction). Any shear joint supporting gravity loads from two or more floors. Splices resisting applied tension.
M Medium	Joints in steel moment frames, with redundancy. Joints in steel moment frames with a designed secondary system for lateral loading. Any shear joint supporting gravity loads from more than one member at a given floor level. Splices resisting only shear or compression or both.
L Low	Shear, compression and tension joints supporting single members only, and not part of the lateral-force-resisting system. Joints not required to carry gravity loads.

Table 5-3 Process and Visual Inspection Categories

Consequence	Demand		
	A	B	C
H	1	1	2
M	1	2	3
L	2	3	3

Note: Refer to Table 5-5 Welding Inspection Checklist

Table 5-4 Nondestructive Testing

Consequence	Demand		
	A	B	C
H	CJP MT 100% of joints, full length UT 100% of joints, full length PJP, fillets MT 100% of joints full length	CJP MT 100% of joints, full length if transversely loaded, partial length if longitudinally loaded UT 100% of joints, full length if transversely loaded, partial length if longitudinally loaded (Reduce UT to 25% of joints, of length as above, with high acceptance rate) PJP, fillets MT 25% of joints, full length if transversely loaded, partial length if longitudinally loaded	CJP UT 10% of joints, full length if transversely loaded, partial length if longitudinally loaded PJP, fillets MT 10% of joints, 6" spot at random
M	CJP MT 100% of joints, full length UT 100% of joints, full length (Reduce UT to 25% of joints, full length, with high acceptance rate) PJP, fillets MT 100% of joints full length	CJP MT 100% of joints, full length if transversely loaded, partial length if longitudinally loaded UT 100% of joints, full length if transversely loaded, partial length if longitudinally loaded (Reduce UT to 25% of joints, of length as above, with high acceptance rate) PJP, fillets MT 25% of joints, full length if transversely loaded, partial length if longitudinally loaded	No NDT required
L	CJP MT 25% of joints, full length. UT 25% of joints, full length PJP, fillets MT 100% of joints full length	CJP UT 10% of joints, full length PJP, fillets MT 10% of joints, 6" spot at random	No NDT required

- Notes:
1. CJP, complete joint penetration; PJP, partial joint penetration; MT, magnetic particle testing; UT, ultrasonic testing.
 2. UT only when weld throat is 5/16" or greater.
 3. Reduce rate of UT and MT when an individual welder's reject rate is less than 5%, after 40 welds have been inspected.
 4. Partial length testing for longitudinally loaded welds is applicable for welds over 24" in length, and includes the beginning and end of each weld for a 6" length, plus any location along the length of the weld where a start and restart is visually noted for a distance of 6" on either side of the stop/start location, and a 6" length every 10' for a given weld.
 5. CJP and PJP groove welded column splices that are a part of the seismic-force-resisting system and are subject to applied tension shall be ultrasonically tested as BH/T

Table 5-5 Welding Inspection Checklist

Process and Visual Welding Inspection Category		1				2				3			
Inspection Tasks	Welder	QC		QA		QC		QA		QC		QA	
		H	O	H	O	H	O	H	O	H	O	H	O
Inspection Prior To Welding													
Proper WPS selected for joint detail	✓	✓		✓		✓			✓		✓		✓
Proper welding materials selected	✓	✓		✓		✓			✓		✓		✓
WPS settings (voltage, polarity, current, wire feed speed) on welding equipment verified	✓	✓			✓✓		✓		✓		✓		✓
Shielding gas type (if used) verified	✓	✓			✓		✓		✓		✓		✓
Shielding gas flow rate setting verified	✓	✓			✓		✓		✓		✓		✓
Fit of backing bar (if used) acceptable	✓	✓			✓		✓		✓		✓		✓
Measure root opening	✓		✓		✓		✓		✓		✓		✓
Measure groove angle	✓		✓		✓		✓		✓		✓		✓
Verify above dimensions within joint tolerance and WPS tolerance	✓	✓			✓		✓		✓		✓		✓
Mark root edge location on beam flange for UT inspection (if required)	✓	✓			✓		✓		✓		✓		✓
Condition of steel surface acceptable	✓	✓			✓		✓		✓		✓		✓
Existing tack welds clean and of adequate quality	✓	✓			✓		✓		✓		✓		✓
Wind speed within limits	✓		✓		✓		✓		✓		✓		✓
Weld joint surfaces free of discontinuities	✓		✓		✓		✓		✓		✓		✓
Minimum preheat required applied and verified	✓		✓✓		✓✓		✓		✓		✓		✓
Maximum preheat verified (if specified)	✓		✓✓		✓✓		✓		✓		✓		✓
Observation of welder’s inspection	✓		✓✓		✓		✓		✓		✓		✓
Observation of QC inspection					✓✓		✓		✓				✓

See Legend and Notes at end of Table

Table 5-5 Welding Inspection Checklist (page 2)

Process and Visual Welding Inspection Category		1				2				3			
Inspection Tasks	Welder	QC		QA		QC		QA		QC		QA	
		H	O	H	O	H	O	H	O	H	O	H	O
Inspection During Welding													
WPS followed (voltage, current, wire feed speed, travel speed, stickout, gas flow rate, pass location)	✓		✓✓		✓✓		✓✓		✓		✓		✓
Welding materials exposure control maintained	✓		✓✓		✓✓		✓✓		✓		✓		✓
Minimum interpass temperature maintained and verified	✓		✓✓		✓✓		✓✓		✓		✓		✓
Maximum interpass temperature verified (if specified)	✓		✓✓		✓✓		✓✓		✓		✓		✓
No detrimental change to environmental conditions	✓		✓		✓		✓		✓		✓		✓
Tack welds do not crack during welding	✓		✓		✓		✓		✓		✓		✓
Each pass cleaned	✓		✓		✓		✓		✓		✓		✓
Each pass within profile limitations	✓		✓		✓		✓		✓		✓		✓
Each pass meets quality requirements	✓		✓		✓		✓		✓		✓		✓
Proper technique used (electrode angle, stringer beads)	✓		✓		✓		✓		✓		✓		✓
NDT in process when required	✓	✓		✓		✓		✓			✓		✓
Observation of welder’s inspection			✓		✓		✓		✓		✓		✓
Observation of QC process					✓				✓				✓

See Legend and Notes at end of Table

Table 5-5 Welding Inspection Checklist (page 3)

Process and Visual Welding Inspection Category			1				2				3			
Inspection Tasks	Welder	QC		QA		QC		QA		QC		QA		
		H	O	H	O	H	O	H	O	H	O	H	O	
Inspection After Welding														
Welder identification legible	✓	✓		✓		✓		✓		✓			✓	
Inspection delay period satisfied		✓		✓		✓		✓		✓		✓		
Weld cleaned	✓	✓		✓		✓		✓		✓		✓		
Weld size and length verified	✓	✓			✓	✓			✓	✓			✓	
Weld within profile limitations	✓	✓			✓	✓			✓	✓			✓	
Weld appearance indicates thorough fusion	✓	✓		✓		✓		✓		✓			✓	
Weld craters acceptable	✓	✓			✓	✓			✓	✓			✓	
Undercut within limitations	✓	✓		✓		✓		✓		✓			✓	
Porosity within limitations	✓	✓			✓	✓			✓	✓			✓	
Weld free of cracks	✓	✓		✓		✓		✓		✓			✓	
Backing bars removed (if required)	✓	✓		✓		✓		✓		✓			✓	
Weld tabs removed (if required)	✓	✓		✓		✓		✓		✓			✓	
Surface finish (grind, contour) as required	✓	✓		✓		✓		✓		✓			✓	
Observation of welder’s inspection			✓		✓	✓	✓		✓		✓		✓	
Observation of QC process					✓				✓				✓	
NDT completed				✓		✓		✓		✓		✓		

See Legend and Notes at end of Table

Table 5-5 Welding Inspection Checklist (page 4)

Legend:	
✓	The welder or Inspector, as noted, should perform this function, with measurements as necessary.
✓✓	The Inspector should observe these operations more frequently.
H	Hold – The welder shall not proceed with welding until inspection of this item is performed by the designated Inspector. After this inspection, the welder may proceed with welding until completion, with observation of welding functions on a random, periodic basis by the Inspector. For inspection after welding, this inspection must be performed prior to final acceptance of the item.
O	Observe – The welder may proceed with welding after completing his / her own inspection. The Inspector shall perform random, daily, periodic inspection and observation of these welding functions.
Notes:	
1.	The precise level and frequency of observation by either QC or QA is not specified. Inspection frequency shall be adequate to provide reasonable confidence in the control of the welding process and the quality of the completed welds. Consideration shall be made of the consistency achieved in satisfying the required welding parameters, and the effect of the welding parameter upon weld quality and performance.
2.	As a minimum, the observation inspection tasks listed shall be performed on a daily basis.
3.	Hold point inspections for WPS selection and welding materials need be performed only when changed by the welder.
4.	This list shall not be considered exclusive of any additional inspection tasks that may be necessary to meet the requirements of the codes or the Quality Assurance Plan.

6. BOLTING

6.1 Specifications

6.1.1 Bolting Material

The Contract Documents should specify the material for bolted connections. Production and inspection for all fastener material should be specified to conform to the latest editions of the American Society for Testing and Materials (ASTM) *Specifications*. High-strength bolts should conform to ASTM A325, A490, F1852 (twist-off bolt), or other bolt specifications as permitted for use in the American Institute of Steel Construction (AISC) and Research Council for Structural Connections (RCSC) *Specifications*. Nuts should meet either ASTM A563 or A194. Washers should meet ASTM F436. Direct tension indicators, if used, should meet ASTM F959.

6.1.2 Installation Requirements

All bolts and fastener assemblies should be installed in accordance with the requirements of the RCSC *Specifications*.

6.2 Submittals

6.2.1 Material Certifications

The Contract Documents should require the submittal of certifications for all bolting material, including bolts, nuts and washers. The manufacturer's certifications should state that the bolts, washers, nuts or other fastener components meet the applicable ASTM *Specification*. Certification of fastener material by the fastener component manufacturer is also a requirement of the ASTM *Specifications*. Certifications should be provided for each production lot of fastener component, with the exception that shipping lot certification is permitted for A490 bolts. The acceptance of shipping lot certifications for other fastener components is discouraged. Copies of certifications should be provided to the appropriate Inspector for review and use.

6.2.2 Bolting Installation Procedures

The Contract Documents should require the submittal of written procedures for the pre-installation testing, installation and pretensioning of high-strength bolts on the project. The procedures must meet the requirements of the RCSC *Specification*. Procedures need be submitted only for the methods of installation being used by the Fabricator and Erector, which may include the turn-of-nut, calibrated wrench, twist-off bolt, or direct tension indicator methods. Procedures should include both installation to the snug tight condition and pretensioned installation. Bolting procedures must also be distributed to the Inspector responsible for bolting inspection.

6.2.3 Bolting Inspection Procedures

The Contract Documents and Quality Assurance Plan should require the submittal of written inspection procedures for bolted installations. Such procedures must meet the requirements of the RCSC *Specification*. Inspection procedures should be written specifically for the installation and

pretensioning methods to be used, i.e. turn-of-nut, calibrated wrench, twist-off bolt, or direct tension indicator method. Written procedures should also be prepared for the arbitration of disputes, to be used should a dispute arise regarding the pretension of previously installed fasteners.

6.3 Material Controls

The Contract Documents should establish requirements for fastener material control, if control beyond type or lot is required. To verify the material suitability and installation procedures, the RCSC *Specification* requires pre-installation testing of each lot, or combinations of lots as used, prior to installation in the work. No records need be maintained regarding source or lot of individual fastener material once installed.

6.4 Quality Control and Quality Assurance

6.4.1 Extent of Bolting Inspection

The Quality Assurance Plan should contain a list of all required bolting inspections and should designate the responsible party. Such inspection may be provided by the Contractor, by the Special Inspector, or by a designated Bolting Inspector.

Bolted joints require verification of the quality of the holes, visual inspection by observation of the pre-installation tests and the tightening operations, and if a slip-critical joint is specified, the observation of the condition of the faying surface. The use of torque for random testing of installed bolts is not required and is discouraged.

6.4.2 Bolting Quality Control Program

The Contractor's Quality Control program for bolting should, as a minimum, include the following.

- Manufacturer's certifications are reviewed for adequacy and compliance with project requirements.
- Manufacturer's certifications are kept on file.
- A proper protected storage area is provided for all fasteners.
- A suitable material control system, including lot control, is in use.
- A bolt calibration device, calibrated within the past year, is available.
- Written bolting procedures are provided for the installation methods to be used.
- The bolting crew must know the selected bolting installation procedures.
- Bolting Inspectors must be trained and knowledgeable.

6.5 Bolt Installer Qualification

All personnel installing and pretensioning high-strength bolts should have the necessary knowledge and skills to perform the task. Verifying adequate knowledge of the installer provides a high level of confidence that the correct material will be selected and installed, that the snug condition will be achieved, and that the selected pretensioning method will be properly applied. The bolt installer qualification program should reduce the time and effort spent in bolt inspection, and reduce or eliminate the need for arbitration inspection.

Personnel responsible for installing bolts should possess, as a minimum, the following knowledge:

- bolt and nut grade identification,
- bolt length measurement,
- required lubrication conditions,
- the use of field bolt lists,
- the assessment of the snug tight condition, and
- the proper techniques for performing the selected pretensioning method.

For the turn-of-nut method, required knowledge includes determining the required rotation beyond the snug condition, proper match-marking techniques, and wrench observation techniques. For the calibrated wrench method, required knowledge includes the methods of wrench calibration, requirements for recalibration of wrenches, and the proper use of the calibrated wrench. For twist-off bolts, the required knowledge includes the use of the electric wrench for snugging, and the use of the wrench for pretensioning. For the direct tension indicator (DTI) method, required knowledge includes the placement of the DTI in the assembly, the use of feeler gages to check the snug condition, and the use of feeler gages to verify the pretensioned condition.

Bolt installer qualification testing should be performed through verbal or written questions on the above topics asked by the inspector, appropriate for the bolt pretensioning method selected for use. The installer should also perform three pre-installation verification tests for the method selected, witnessed by the inspector. Upon completion of this bolt installer qualification testing, the bolt installation personnel may proceed to install, snug, and pretension bolts on the project. Bolt installer qualification testing does not waive requirements for bolt installation inspection.

6.6 Bolting Inspection Tasks

The Bolting Inspector should coordinate the inspection work closely with the Contractor and other Inspectors. The Bolting Inspector(s) must perform all inspections required by the Contract Documents, the Quality Assurance Plan, the RCSC *Specification*, and the Building Code. Bolting inspection should be performed in a timely manner, so as not to hinder production, and to detect

bolting problems upon or soon after occurrence so that corrective measures may be taken by the Contractor to rectify such problems.

The duties of the Bolting Inspector, whether designated either QC or QA, should include the following items:

- a. Review and understand the applicable portions of the specifications, contract drawings, shop detail drawings and erection plans for the project.
- b. Review all manufacturer certifications for material compliance with the project requirements. (Refer to Figure 6-1)
- c. Verify bolting material identification. (Refer to Figure 6-1)
- d. Verify suitable, controlled storage conditions. (Refer to Figure 6-1)
- e. Verify that all applicable bolt installation procedures are available, current and accurate.
- f. Verify that all bolt installers are qualified (Refer to Figure 6-2)
- g. Verify that the appropriate bolt installation procedure has been provided and reviewed by each bolting crew member performing the work. (Refer to Figure 6-2)
- h. Observe the Contractor's pre-installation testing performed at the start of the work for each assembly lot. (Refer to Figure 6-3)
- i. For the calibrated wrench method of installation, observe calibration of the wrench(s) at the start of each work shift. (Refer to Figure 6-4)
- j. Verify the suitability of the bolted joints, such as bolt hole size and condition, prior to assembly. Check for unfair reaming or slotting of poorly aligned holes. (Refer to Figure 6-4)
- k. For slip-critical joints, verify that the required faying surface conditions are met. (Refer to Figure 6-4)
- l. Prior to pretensioning, verify that all bolts have been installed and the joint brought to the snug tight condition.
- m. For pretensioned joints, routinely observe, at suitable intervals, the pretensioning operations to verify the proper application of the bolting procedures. (Refer to Figure 6-4)
- n. Arbitrate any disputes regarding achieved bolt pretension immediately upon installation of the bolts in dispute. Any appreciable delay in arbitration will result in considerable inaccuracies in the arbitration procedures provided by the *RCSC Specification*. (Refer to Figure 6-4)
- o. Complete a written report recording the joints observed, inspected, and accepted. The report should be transmitted to the designated recipients in a timely manner. (Refer to Figure 6-4)

Bolting Material Inspection Record

(See Sections 6.6 b, c, d)

Project _____

Inspector _____

Date of Inspection ____/____/____

Complete this inspection log for each type of fastener product, diameter, length, grade and production lot for the fasteners to be used on the project.

Bolt

☐ A325 ☐ A490 ☐ A307 ☐ _____

Twist-Off Bolt

☐ F1852 ☐ Marked A325 ☐ Marked A490

Nut

☐ A563 grade _____ ☐ A194 grade _____

Washer

☐ F436

Direct Tension Indicator (DTI)

☐ F959 ☐ Type 325 ☐ Type 490

Manufacturer's mark and name _____

Diameter

1/2 5/8 3/4 7/8 1 1-1/8 1-1/4 1-3/8 1-1/2 _____

Length _____

Finish

- ☐ Black
- ☐ Hot dip galvanized
- ☐ Mechanically galvanized
- ☐ Epoxy coated
- ☐ Other

Lot Number _____

Figure 6-1 Bolting Material Inspection Record

Bolting Material Inspection Record

Container type

- ☐ Steel keg
- ☐ Wood keg
- ☐ Cardboard box
- ☐ Other
- ☐ Manufacturer's container
- ☐ Repackaged container

Storage conditions

- ☐ Protected
- ☐ Exposed

Lubrication condition

- ☐ Oily
- ☐ Dry
- ☐ Rusty
- ☐ Manufacturer coating
- ☐ Mixed
- ☐ Relubricated using _____

- ☐ **Certification reviewed** Date ____/____/____

Page 2

Figure 6-1 Bolting Material Inspection Record (continued)

Bolt Installer Qualification

(See Sections 6.5 and 6.6 f and g)

Bolt Installer Name _____ ID Number _____

	Bolt Installation Method			
	Turn-of-Nut	Calibrated Wrench	Twist-Off Bolt	Direct Tension Indicator
	Inspector _____ Date ____/____/____	Inspector _____ Date ____/____/____	Inspector _____ Date ____/____/____	Inspector _____ Date ____/____/____
Verification of Knowledge	Inspector _____ Date ____/____/____	Inspector _____ Date ____/____/____	Inspector _____ Date ____/____/____	Inspector _____ Date ____/____/____
Verification of skills using pre-installation test	Inspector _____ Date ____/____/____	Inspector _____ Date ____/____/____	Inspector _____ Date ____/____/____	Inspector _____ Date ____/____/____

Figure 6-2 Bolt Installer Qualification

Bolt Pre-Installation Assembly Tests

(See Section 6.6 h)

Project _____ Inspector _____

Date ____/____/____

Bolt Tension Calibrator Model _____ Number _____

Date of test ____/____/____

Test performed by _____ (Fabricator or Erector)

Bolt lot number _____

Nut lot number _____

Assembly lot number _____

Direct Tension Indicator lot number _____

Lubrication condition ☐ oily ☐ dry ☐ rusty ☐ relubricated using _____ ☐ mixed

For short bolt testing, with installation method other than turn-of-nut method:

DTI lot number _____

Measured average DTI gap at required pretension _____

Bolt Grade ☐ A325 ☐ A490

Bolt Diameter _____ Bolt Length _____

Minimum required pretension - (RCSC Specification, Table 4, following) _____ kips

Required Pre-Installation Test tension _____ kips

Installation method selected

- ☐ Turn-of-Nut
- ☐ Calibrated Wrench
- ☐ Twist-Off Type Tension Control Bolt
- ☐ Direct Tension Indicator

☐ Turn-of-Nut Method

(Performed on each lot of bolt / nut assembly at the start of the work)

- ☐ Required turns - ☐ 1/3 ☐ 1/2 ☐ 2/3
- ☐ Measured pretension _____ kips

☐ Calibrated Wrench Method

(Performed daily for each diameter, grade and lot)

- ☐ Achieved tension for wrench _____ kips
- ☐ Actual rotation _____ turn (not to exceed RCSC Specification, Table 5 rotation, following)

☐ Twist-Off Bolt Method

(Performed on each lot of bolt/nut/washer assembly at the start of the work)

- ☐ Achieved tension at twist-off _____ kips

☐ Direct Tension Indicator Method

(Performed on each lot of bolt/nut assembly with DTI at the start of the work)

Feeler gage thickness - .005 in .015 in

Pre-Installation Test Tension required _____ kips (See RCSC Specification, Table 4, following)

Actual pretension _____ kips (should equal or slightly exceed above value)

Number of feeler gage refusals _____

Number of DTI gaps	4	5	6	7	8	9
Number of refusals permitted in Pre-Installation Test	1	2	2	3	3	4
Number of refusals required for installation acceptance	2	3	3	4	4	5

Figure 6-3 Bolt Pre-Installation Assembly Tests

Bolt Pre-Installation Assembly Tests

RCSC Specification, Table 4. Fastener Tension Required for Pretensioned and Slip-Critical Connections

Bolt Diameter, d_b	Minimum Tension ^a in 1,000s of Pounds (kips)			
	A325 Bolts		A490 Bolts	
	Required Pretension	Pre-Installation Test Tension Required	Required Pretension	Pre-Installation Test Tension Required
1/2	12	13	15	16
5/8	19	20	24	25
3/4	28	29	35	37
7/8	39	41	49	51
1	51	54	64	67
1-1/8	56	59	80	84
1-1/4	71	75	102	107
1-3/8	85	89	121	127
1-1/2	103	108	148	155

RCSC Specification, Table 5. Nut Rotation from Snug Tight Condition^{a,b}

Bolt Length ^c (underside of head to end of bolt)	Disposition of Outer Face of Bolted Parts		
	Both faces normal to bolt axis	One face normal to bolt axis, other sloped not more than 1:20	Both faces sloped not more than 1:20 from normal to bolt axis
Not more than $4d_b$	1/3 turn	1/2 turn	2/3 turn
More than $4d_b$ but not more than $8d_b$	1/2 turn	2/3 turn	5/6 turn
More than $8d_b$ but not more than $2d_b$	2/3 turn	5/6 turn	1 turn

^a Nut rotation is relative to bolt regardless of the element (nut or bolt) being turned. For required nut rotations of 1/2 turn and less, the tolerance is plus or minus 30 degrees; for required nut rotations of 2/3 turn and more, the tolerance is plus or minus 45 degrees.

^b Applicable only to joints in which all material within the grip is steel.

^c When the bolt length exceeds $12d_b$, the required nut rotation shall be determined by actual testing in a suitable tension calibrator that simulates the conditions of solidly fitting steel.

Figure 6-3 Bolt Pre-Installation Assembly Tests (continued)

Bolted Joint Inspection Record <i>(See Section 6.6 i, j, k, l, m, n, o)</i>		
Project _____		
Inspector _____		
Date ____/____/____		
Joint: Elevation / floor _____ Grid _____		
Mark _____ to mark _____	Mark _____ to mark _____	Mark _____ to mark _____
Mark _____ to mark _____	Mark _____ to mark _____	Mark _____ to mark _____
Mark _____ to mark _____	Mark _____ to mark _____	Mark _____ to mark _____
Required installation: ☐ Snug Tight ☐ Pretensioned ☐ Slip-Critical		
Hole condition verification: ☐ No flame-cutting ☐ No unfair reaming or slotting (limited to 1/32") <i>Note: Flame-cutting and unfair reaming/slotting must be reported to the Engineer for review</i> Reported ? Date ____/____/____		
Shear-Bearing Joints - Threads Excluded Condition Only ☐ Proper length bolt used ☐ Proper orientation of bolt		
Snug Condition Verification (no subsequent pretensioning required)		
Date ____/____/____ Inspector _____		
☐ Steel in solid contact (no gaps in vicinity of bolt holes) ☐ Stick-out ☐ Bolt end flush with or beyond face of nut ☐ Stickout not excessive (nut has not run onto threadrunout)		
Snug Condition Verification (prior to pretensioning)		
Date ____/____/____ Inspector _____		
☐ Steel in firm contact (no gaps in vicinity of bolt holes) ☐ Turn-of-Nut Method: Match-marking properly prepared (if match mark system used) ☐ DTI Method: Number of DTI refusals below that required for completed installation ☐ Twist-Off Bolt Method: Spline intact		
Page 1		

Figure 6-4 Bolted Joint Inspection Record

Bolted Joint Inspection Record					
Pretension Verification					
Date ____/____/____ Inspector _____					
Stick-out					
☐ Bolt end flush with or beyond face of nut ☐ Stickout not excessive (nut has not run onto thread runout, use matching fasteners for comparison)					
Turn-of-Nut Method (match-marked)					
☐ Rotation required 1/3 1/2 2/3 _____ Turn ☐ Rotation applied 1/3 1/2 2/3 _____ Turn					
Turn-of-Nut Method (observed)					
☐ Rotation required 1/3 1/2 2/3 _____ Turn ☐ Rotation applied 1/3 1/2 2/3 _____ Turn					
Calibrated Wrench Method					
Wrench calibration log number _____					
Calibration date and time ____/____/____ ____:____ am/pm					
DTI Method					
Number of DTI gaps _____					
Number of refusals required (½ or more) _____					
☐ Number of refusals achieved					
Twist-Off Bolt method					
☐ Spline sheared off by wrench					
For slip-critical joints only:					
☐ No paint in faying surface ☐ Painted faying surface (if used) ☐ Approved coating type ☐ Coating cured					
Arbitration of Disputes					
Date ____/____/____ Inspector _____					
Reason for arbitration <i>(Note: No arbitration for Twist-Off Bolt method or DTI method)</i>					
☐ Pretensioning applied prior to verification of snug by inspector ☐ Turn-of-Nut Method (match-marked) - Turns not applied or verifiable ☐ Turn-of-Nut Method (observed wrench) - Wrench use not observed by installation crew ☐ Calibrated Wrench ☐ Wrench calibration not performed ☐ Wrench required recalibration ☐ Lubrication / condition of bolt / nut threads worse than calibrated					
Page 2					

Figure 6-4 Bolted Joint Inspection Record

REFERENCES, BIBLIOGRAPHY, AND ACRONYMS

This section contains references, additional bibliography and acronyms that are generally common to the set of reports, FEMA-350, FEMA-351, FEMA-352, and FEMA-353. Following the regular references are three sections containing ASTM Standards published by the American Society for Testing and Materials, West Conshohocken, Pennsylvania and listed numerically, AWS Specifications published by the American Welding Society, Miami, Florida, and listed numerically, FEMA Reports published by the Federal Emergency Management Agency, Washington, DC, and listed by report number, and SAC Reports published by the SAC Joint Venture, Sacramento, California, and listed by report number.

References and Additional Bibliography.

- AISC, 1985, *Specification for Structural Joints using ASTM A325 or A490 Bolts*, American Institute of Steel Construction, Chicago, Illinois.
- AISC, 1989, *Manual of Steel Construction*, ASD, Ninth Edition, American Institute of Steel Construction, Chicago, Illinois.
- AISC, 1993, 1997, *Load and Resistance Factor Design Specifications for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, Illinois.
- AISC, 1994a, *Proceedings of the AISC Special Task Committee on the Northridge Earthquake Meeting*, American Institute of Steel Construction, Chicago, Illinois.
- AISC, 1994b, *Northridge Steel Update 1*, American Institute of Steel Construction, Chicago, Illinois.
- AISC, 1997, *Seismic Provisions for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, Illinois.
- AISC, 1998a, *Load and Resistance Factor Design Specifications for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, Illinois.
- AISC, 1998b, *LRFD Manual of Steel Construction*, 2nd Edition, American Institute of Steel Construction, Chicago, Illinois.
- AISC, 1999, *Supplement No. 1 to the 1997 Seismic Provisions for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, Illinois.
- Allen, J., Partridge, J.E., Richard, R.M., and Radau, S., 1995, "Ductile Connection Designs for Welded Steel Moment Frames," *Proceedings, 64th Annual Convention*, Structural Engineers Association of California, Sacramento, California.
- Anderson, J, Duan, J., Xiao, Y., and Maranian, P., 2000, *Improvement of Welded Connections Using Fracture Tough Overlays*, Report No. SAC/BD-00/20, SAC Joint Venture, Sacramento, California.
- ASCE, 1998, ASCE-7 maps, American Society of Civil Engineers, Reston, Virginia.
- ASTM citations: see the list of ASTM Standards on page R-4.

- ATC, 1985, *Earthquake Damage Evaluation Data for California*, Report ATC-13 , Applied Technology Council, Redwood City, California.
- ATC, 1987, *Evaluating the Seismic Resistance of Existing Buildings*, Report ATC-14, Applied Technology Council, Redwood City, California.
- ATC, 1989, *Procedures for Postearthquake Safety Evaluations of Buildings*, Report ATC-20, Applied Technology Council, Redwood City, California.
- ATC, 1992, *Guidelines for Cyclic Seismic Testing of Components of Steel Structures*, Report ATC-24, Applied Technology Council, Redwood City, California.
- ATC, 1995, *Addendum to the ATC-20 Postearthquake Building Safety Evaluation Procedures*, Report ATC-20-2, Applied Technology Council, Redwood City, California.
- ATC, 1997a, *Seismic Evaluation and Retrofit of Concrete Buildings*, prepared by the Applied Technology Council (Report No. ATC-40), for the California Seismic Safety Commission (Report No. SSC 96-01), Sacramento, California.
- ATC, 1997b, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, Report No. FEMA-273, prepared by the Applied Technology Council for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, DC.
- ATC, 1997c, *Commentary to NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, Report No. FEMA-274, prepared by the Applied Technology Council for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, DC.
- Avent, R., 1992, “Designing Heat-Straightening Repairs,” *National Steel Construction Conference Proceedings*, Las Vegas, Nevada.
- AWS citations: see the list of AWS reports, specifications and codes on page R-5.
- Barsom, J.M., 1996, “Steel Properties — Effects of Constraint, Temperature, and Rate of Loading,” *Proceedings of the 2nd US Seminar, Seismic Design, Evaluation and Retrofit of Steel Bridges*, San Francisco, Report No. UCB/CEE STEEL-96/09, Dept. of Civil and Environmental Engineering, UC Berkeley, pp.115-143.
- Boore, D.M., and Joyner, W.B., 1994, *Proceedings of Seminar on New Developments in Earthquake Ground-Motion Estimation and Implications for Engineering Design Practice*, Report ATC-35-1, Applied Technology Council, Redwood City, California, pp 6-1 to 6-41.
- BSSC, 1992, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings*, developed by the Building Seismic Safety Council for the Federal Emergency Management Agency, Report FEMA-178, Washington, D.C.
- BSSC, 1997a, *1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 1 – Provisions*, prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency (Report No. FEMA-302), Washington, DC.
- BSSC, 1997b, *1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 2 – Commentary*, prepared by the Building Seismic

- Safety Council for the Federal Emergency Management Agency (Report No. FEMA-303), Washington, DC.
- Campbell, K.W., and Bozorgnia, Y., 1994, "Near-Source Attenuation of Peak Horizontal Acceleration from Worldwide Accelerograms Recorded from 1957 to 1993," *Fifth U.S. National Conference on Earthquake Engineering, Proceedings*, Vol. III, pp 283-292, Earthquake Engineering Research Institute, Oakland, California.
- Chi, W.M., Deierlein, G., and Ingraffea, A., 1997, "Finite Element Fracture Mechanics Investigation of Welded Beam-Column Connections," SAC Joint Venture, Report No. SAC/BD-97/05.
- FEMA citations: see the list of FEMA reports on page R-6.
- Goel, R.K., and Chopra, A.K., 1997, "Period Formulas for Moment-Resisting Frame Buildings," *Journal of Structural Engineering*, Vol. 123, No. 11, pp. 1454-1461.
- Gross, J.L., Engelhardt, M.D., Uang, C.M., Kasai, K. and Iwankiw, N.R., 1999, *Modification of Existing Welded Steel Moment Frame Connections for Seismic Resistance, AISC Design Guide Series 12*, American Institute of Steel Construction, Chicago, Illinois.
- Grubbs, K., 1997, "The Effect of the Dogbone Connection on the Elastic Stiffness of Steel Moment Frames" Masters Thesis, Department of Civil Engineering, University of Texas at Austin.
- ICBO, 1988, 1991, and 1997, *Uniform Building Code, indicated edition*, International Conference of Building Officials, Whittier, California.
- ICC, 2000, *International Building Code*, International Code Council, Falls Church, Virginia.
- Kircher, C.A., Nassar, A.A., Kustu, O. and Holmes, W.T., 1997, "Development of Building Damage Functions for Earthquake Loss Estimation," *Earthquake Spectra*, Vol. 13, No. 4, Earthquake Engineering Research Institute, Oakland, California, pp. 663-682.
- Kircher, C.A., Reitherman, R.K., Whitman, R.V., and Arnold, C., 1997, "Estimation of Earthquake Losses to Buildings," *Earthquake Spectra*, Vol. 13, No. 4, Earthquake Engineering Research Institute, Oakland, California, pp. 703-720.
- Kircher, C.A., 1999, *Procedures for Development of HAZUS-Compatible Building-Specific Damage and Loss Functions*, National Institute of Building Sciences, Washington, D.C.
- Krawinkler, H., Gupta, A., Medina, R. and Luco, N., 2000, *Loading Histories for Seismic Performance Testing of SMRF Components and Assemblies*, Report No. SAC/BD-00/10, SAC Joint Venture, Sacramento, California.
- NIBS, 1997a, *HAZUS Earthquake Loss Estimation Methodology, Users Manual*, National Institute of Building Sciences, Washington, DC.
- NIBS, 1997b, *HAZUS Earthquake Loss Estimation Methodology, Technical Manual*, 3 Volumes. National Institute of Building Sciences, Washington, DC.
- RCSC, 1996, *Load and Resistance Factor Design: Specification for Structural Joints Using ASTM A325 or A490 Bolts*, Research Council on Structural Connections.

Richard, R., Partridge, J.E., Allen, J., and Radau, S., 1995, "Finite Element Analysis and Tests of Beam-to-Column Connections," *Modern Steel Construction*, Vol. 35, No. 10, pp. 44-47, American Institute of Steel Construction, Chicago, Illinois.

SAC citations: see the list of SAC Joint Venture reports on page R-7.

Shonafelt, G.O., and Horn, W.B, 1984, *Guidelines for Evaluation and Repair of Damaged Steel Bridge Members*, NCHRP Report 271, prepared by the National Cooperative Highway Research Program, for the Transportation Research Board, Washington, DC.

Wald, D.J., Quitoriano, T.H., Kanamori, H. and Scrivner, C.W., 1998, "Trinet Shakemaps – Rapid Generation of Peak Ground Motion and Intensity Maps for Earthquakes in Southern California", *SMIP98 Proceedings*, California Division of Mines and Geology, Sacramento, California.

Whitman, R., Anagnos, T., Kircher, C., Lagorio, H.J., Lawson, R.S., and Schneider, P., 1997, "Development of a National Earthquake Loss-Estimation Methodology," *Earthquake Spectra*, Vol. 13, No. 4, Earthquake Engineering Research Institute, Oakland, California, pp. 643-661.

Youssef, N.F.G, Bonowitz, D., and Gross, J.L., 1995, *A Survey of Steel Moment-Resisting Frame Buildings Affected by the 1994 Northridge Earthquake*, Report No. NISTR 56254, National Institute for Science and Technology, Gaithersburg, Maryland.

ASTM Standards.

ASTM Standards are published by the American Society for Testing and Materials, West Conshohocken, Pennsylvania, and are listed alphanumerically.

ASTM, 1997, *Standard Test Methods and Definitions for Mechanical Testing of Steel Products A6, Supplementary Requirement S5*

A36, *Specification for Carbon Structural Steel*

A325, *Specification for Structural Bolts, Steel, Heat-Treated, 120/105 ksi Minimum Tensile Strength*

A435, *Straight Beam Ultrasonic Examination of Steel Plates*

A490, *Specification for Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength*

A563, *Specification for Carbon and Alloy Steel Nuts*

A572, *Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*

A898, *Straight Beam Ultrasonic Examination of Rolled Steel Structural Shapes*

A913, *Specification for High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self-Tempering Process*

A992, *Standard Specification for Steel for Structural Shapes for Use in Building Framing*

E329, *Standard Specification for Agencies Engaged in the Testing and/or Inspection of Material Used in Construction*

E543, *Standard Practice for Agencies Performing Nondestructive Testing*
E548, *Standard Guide for General Criteria Used for Evaluating Laboratory Competence*
E994, *Standard Guide for Laboratory Accreditation Systems*
E1212, *Standard Practice for Establishment and Maintenance of Quality Control Systems for Nondestructive Testing Agencies*
E1359, *Standard Guide for Surveying Nondestructive Testing Agencies*
F436, *Specification for Hardened Steel Washers*
F959, *Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners*
F1554, *Specification for Anchor Bolts, Steel, 36, 55, and 105 ksi Yield Strength*
F1852, *Specification for “Twist-Off” Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*

AWS Reports, Specifications, and Codes.

AWS reports are published by the American Welding Society, Miami, Florida, and are listed alphanumerically.

AWS A2.4, *Standard Symbols for Welding, Brazing, and Nondestructive Testing*
AWS A4.3, *Standard Methods for Determination of the Diffusible Hydrogen Content of Martensitic, Bainitic, and Ferritic Steel Weld Metal Produced by Arc Welding*
ANSI/AWS A5.1-91, *Specification for Carbon Steel Electrodes for Shielded Metal Arc Welding*
ANSI/AWS A5.18-93, *Specification for Carbon Steel Electrodes and Rods for Gas Shielded Arc Welding*
ANSI/AWS A5.20-95, *Specification for Carbon Steel Electrodes for Flux-Cored Arc Welding*
AWS, 1995, *Presidential Task Group Report*
ANSI/AWS A5.5-96, *Specification for Low-Alloy Steel Electrodes for Shielded Metal Arc Welding*
ANSI/AWS A5.28-96, *Specification for Low-Alloy Steel Electrodes and Rods for Gas Shielded Arc Welding*
ANSI/AWS A5.23/A5.23M-97, *Specification for Low-Alloy Steel Electrodes and Fluxes for Submerged Arc Welding*
ANSI/AWS A5.25/A5.25M-97, *Specification for Carbon and Low-Alloy Steel Electrodes and Fluxes for Electroslag Welding*
ANSI/AWS A5.26/A5.26M-97, *Specification for Carbon and Low-Alloy Steel Electrodes for Electrogas Welding*
ANSI/AWS A5.32/A5.32M-97, *Specification for Welding Shielding Gases*

ANSI/AWS A5.17/A5.17M-97, *Specification for Carbon Steel Electrodes and Fluxes for Submerged Arc Welding*

ANSI/AWS A5.29-98, *Specification for Low-Alloy Steel Electrodes for Flux-Cored Arc Welding*

AWS D1.1-1998, 2000, *Structural Welding Code – Steel*

AWS D1.3, *Structural Welding Code*

AWS D1.4, *Structural Welding Code*

AWS QC1, *Standard for AWS Certification of Welding Inspectors*

FEMA Reports.

FEMA reports are listed by report number.

FEMA-178, 1992, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings*, developed by the Building Seismic Safety Council for the Federal Emergency Management Agency, Washington, DC.

FEMA-267, 1995, *Interim Guidelines, Inspection, Evaluation, Repair, Upgrade and Design of Welded Moment Resisting Steel Structures*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-267A, 1996, *Interim Guidelines Advisory No. 1*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-267B, 1999, *Interim Guidelines Advisory No. 2*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-273, 1997, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, prepared by the Applied Technology Council for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, DC.

FEMA-274, 1997, *NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings*, prepared by the Applied Technology Council for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, DC.

FEMA-302, 1997, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 1 – Provisions*, prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency, Washington, DC.

FEMA-303, 1997, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 2 – Commentary*, prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency, Washington, DC.

FEMA-310, 1998, *Handbook for the Seismic Evaluation of Buildings – A Prestandard*, prepared by the American Society of Civil Engineers for the Federal Emergency Management Agency, Washington, DC.

FEMA-350, 2000, *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-351, 2000, *Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-352, 2000, *Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-353, 2000, *Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-354, 2000, *A Policy Guide to Steel Moment-Frame Construction*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-355A, 2000, *State of the Art Report on Base Metals and Fracture*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-355B, 2000, *State of the Art Report on Welding and Inspection*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-355C, 2000, *State of the Art Report on Systems Performance of Steel Moment Frames Subject to Earthquake Ground Shaking*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-355D, 2000, *State of the Art Report on Connection Performance*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-355E, 2000, *State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-355F, 2000, *State of the Art Report on Performance Prediction and Evaluation of Steel Moment-Frame Buildings*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC Joint Venture Reports.

SAC Joint Venture reports are listed by report number, except for SAC 2000a through 2000k; those entries that do not include a FEMA report number are published by the SAC Joint Venture.

SAC 94-01, 1994, *Proceedings of the Invitational Workshop on Steel Seismic Issues, Los Angeles*, September 1994, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 94-01, 1994b, *Proceedings of the International Workshop on Steel Moment Frames, Sacramento*, December, 1994, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 95-01, 1995, *Steel Moment Frame Connection Advisory No. 3*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

- SAC 95-02, 1995, *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-267, Washington, DC.
- SAC 95-03, 1995, *Characterization of Ground Motions During the Northridge Earthquake of January 17, 1994*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.
- SAC 95-04, 1995, *Analytical and Field Investigations of Buildings Affected by the Northridge Earthquake of January 17, 1994*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.
- SAC 95-05, 1995, *Parametric Analytic Investigations of Ground Motion and Structural Response, Northridge Earthquake of January 17, 1994*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.
- SAC 95-06, 1995, *Technical Report: Surveys and Assessment of Damage to Buildings Affected by the Northridge Earthquake of January 17, 1994*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.
- SAC 95-07, 1995, *Technical Report: Case Studies of Steel Moment-Frame Building Performance in the Northridge Earthquake of January 17, 1994*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.
- SAC 95-08, 1995, *Experimental Investigations of Materials, Weldments and Nondestructive Examination Techniques*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.
- SAC 95-09, 1995, *Background Reports: Metallurgy, Fracture Mechanics, Welding, Moment Connections and Frame Systems Behavior*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-288, Washington, DC.
- SAC 96-01, 1996, *Experimental Investigations of Beam-Column Subassemblages, Part 1 and 2*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.
- SAC 96-02, 1996, *Connection Test Summaries*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-289, Washington, DC.
- SAC 96-03, 1997, *Interim Guidelines Advisory No. 1 Supplement to FEMA-267 Interim Guidelines*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-267A, Washington, DC.
- SAC 98-PG, *Update on the Seismic Safety of Steel Buildings – A Guide for Policy Makers*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.
- SAC 99-01, 1999, *Interim Guidelines Advisory No. 2 Supplement to FEMA-267 Interim Guidelines*, prepared by the SAC Joint Venture, for the Federal Emergency Management Agency, Report No. FEMA-267B, Washington, DC.

- SAC, 2000a, *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-350, Washington, D.C.
- SAC, 2000b, *Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-351, Washington, D.C.
- SAC, 2000c, *Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-352, Washington, D.C.
- SAC, 2000d, *Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-353, Washington, D.C.
- SAC, 2000e, *A Policy Guide to Steel Moment-Frame Construction*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-354, Washington, D.C.
- SAC, 2000f, *State of the Art Report on Base Metals and Fracture*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-355A, Washington, D.C.
- SAC, 2000g, *State of the Art Report on Welding and Inspection*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-355B, Washington, D.C.
- SAC, 2000h, *State of the Art Report on Systems Performance*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-355C, Washington, D.C.
- SAC, 2000i, *State of the Art Report on Connection Performance*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-355D, Washington, D.C.
- SAC, 2000j, *State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-355E, Washington, D.C.
- SAC, 2000k, *State of the Art Report on Performance Prediction and Evaluation*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-355F, Washington, D.C.
- SAC/BD-96/01, *Selected Results from the SAC Phase 1 Beam-Column Connection Pre-Test Analyses*, submissions from B. Maison, K. Kasai, and R. Dexter; and A. Ingrassia and G. Deierlein.
- SAC/BD-96/02, *Summary Report on SAC Phase 1 - Task 7 Experimental Studies*, by C. Roeder (a revised version of this document is published in Report No. SAC 96-01; the original is no longer available).
- SAC/BD-96/03, *Selected Documents from the U.S.-Japan Workshop on Steel Fracture Issues*.

- SAC/BD-96/04, *Survey of Computer Programs for the Nonlinear Analysis of Steel Moment Frame Structures*.
- SAC/BD-97/01, *Through-Thickness Properties of Structural Steels*, by J. Barsom and S. Korvink.
- SAC/BD-97/02, *Protocol for Fabrication, Inspection, Testing, and Documentation of Beam-Column Connection Tests and Other Experimental Specimens*, by P. Clark, K. Frank, H. Krawinkler, and R. Shaw.
- SAC/BD-97/03, *Proposed Statistical and Reliability Framework for Comparing and Evaluating Predictive Models for Evaluation and Design*, by Y.-K. Wen.
- SAC/BD-97/04, *Development of Ground Motion Time Histories for Phase 2 of the FEMA/SAC Steel Project*, by P. Somerville, N. Smith, S. Punyamurthula, and J. Sun.
- SAC/BD-97/05, *Finite Element Fracture Mechanics Investigation of Welded Beam-Column Connections*, by W.-M. Chi, G. Deierlein, and A. Ingrassia.
- SAC/BD-98/01, *Strength and Ductility of FR Welded-Bolted Connections*, by S. El-Tawil, T. Mikesell, E. Vidarsson, and S. K. Kunnath.
- SAC/BD-98/02, *Effects of Strain Hardening and Strain Aging on the K-Region of Structural Shapes*, by J. Barsom and S. Korvink.
- SAC/BD-98/03, *Implementation Issues for Improved Seismic Design Criteria: Report on the Social, Economic, Policy and Political Issues Workshop* by L.T. Tobin.
- SAC/BD-99/01, *Parametric Study on the Effect of Ground Motion Intensity and Dynamic Characteristics on Seismic Demands in Steel Moment Resisting Frames* by G. A. MacRae
- SAC/BD-99/01A, *Appendix to: Parametric Study on the Effect of Ground Motion Intensity and Dynamic Characteristics on Seismic Demands in Steel Moment Resisting Frames* by G. A. MacRae
- SAC/BD-99/02, *Through-Thickness Strength and Ductility of Column Flange in Moment Connections* by R. Dexter and M. Melendrez.
- SAC/BD-99/03, *The Effects of Connection Fractures on Steel Moment Resisting Frame Seismic Demands and Safety* by C. A. Cornell and N. Luco
- SAC/BD-99/04, *Effects of Strength/Toughness Mismatch on Structural and Fracture Behaviors in Weldments* by P. Dong, T. Kilinski, J. Zhang and F.W. Brust
- SAC/BD-99/05, *Assessment of the Reliability of Available NDE Methods for Welded Joint and the Development of Improved UT Procedures* by G. Gruber and G. Light
- SAC/BD-99/06, *Prediction of Seismic Demands for SMRFs with Ductile Connections and Elements* by A. Gupta and H. Krawinkler
- SAC/BD-99/07, *Characterization of the Material Properties of Rolled Sections* by T. K. Jaquess and K. Frank
- SAC/BD-99/08, *Study of the Material Properties of the Web-Flange Intersection of Rolled Shapes* by K. R. Miller and K. Frank

- SAC/BD-99/09, *Investigation of Damage to WSMF Earthquakes other than Northridge* by M. Phipps
- SAC/BD-99/10, *Clarifying the Extent of Northridge Induced Weld Fracturing and Examining the Related Issue of UT Reliability* by T. Paret
- SAC/BD-99/11, *The Impact of Earthquakes on Welded steel Moment Frame Buildings: Experience in Past Earthquakes* by P. Weinburg and J. Goltz
- SAC/BD-99/12, *Assessment of the Benefits of Implementing the New Seismic Design Criteria and Inspection Procedures* by H. A. Seligson and R. Eguchi
- SAC/BD-99/13, *Earthquake Loss Estimation for WSMF Buildings*, by C. A. Kircher
- SAC/BD-99/14, *Simplified Loss Estimation for Pre-Northridge WSMF Buildings*, by B. F. Maison and D. Bonowitz
- SAC/BD-99/15, *Integrative Analytical Investigations on the Fracture Behavior of Welded Moment Resisting Connections*, by G. G. Deierlein and W.-M. Chi
- SAC/BD-99/16, *Seismic Performance of 3 and 9 Story Partially Restrained Moment Frame Buildings*, by B. F. Maison and K. Kasai
- SAC/BD-99/17, *Effects of Partially-Restrained Connection Stiffness and Strength on Frame Seismic Performance*, by K. Kasai, B. F. Maison, and A. Mayangarum
- SAC/BD-99/18, *Effects of Hysteretic Deterioration Characteristics on Seismic Response of Moment Resisting Steel Structures*, by F. Naeim, K. Skliros, A. M. Reinhorn and M.V. Sivaselvan
- SAC/BD-99/19, *Cyclic Instability of Steel Moment Connections with Reduced Beam Section*, by C.-M. Uang and C.-C. Fan
- SAC/BD-99/20, *Local and Lateral-Torsion Buckling of Wide Flange Beams*, by L. Kwasniewski, B. Stojadinovic, and S. C. Goel
- SAC/BD-99/21, *Elastic Models for Predicting Building Performance*, by X. Duan and J. C. Anderson
- SAC/BD-99/22, *Reliability-Based Seismic Performance Evaluation of Steel Frame Buildings Using Nonlinear Static Analysis Methods*, by G. C. Hart and M. J. Skokan
- SAC/BD-99/23, *Failure Analysis of Welded Beam to Column Connections*, by J. M. Barsom
- SAC/BD-99/24, *Weld Acceptance Criteria for Seismically-Loaded Welded Connections*, by W. Mohr
- SAC/BD-00/01, *Parametric Tests on Unreinforced Connections*, by K.-H. Lee, B. Stojadinovic, S. C. Goel, A. G. Margarian, J. Choi, A. Wongkaew, B. P. Reyher, and D.-Y. Lee
- SAC/BD-00/02, *Parametric Tests on the Free Flange Connections*, by J. Choi, B. Stojadinovic, and S. C. Goel
- SAC/BD-00/03, *Cyclic Tests on Simple Connections Including Effects of the Slab*, by J. Liu and A. Astaneh-Asl

- SAC/BD-00/04, *Tests on Bolted Connections*, by J. Swanson, R. Leon and J. Smallridge
- SAC/BD-00/05, *Bolted Flange Plate Connections*, by S. P. Schneider and I. Teeraparbong
- SAC/BD-00/06, *Round Robin Testing of Ultrasonic Testing Technicians*, by R. E. Shaw, Jr.
- SAC/BD-00/07, *Dynamic Tension Tests of Simulated Welded Beam Flange Connections*, by J. M. Ricles, C. Mao, E. J. Kaufmann, L.-W. Lu, and J. W. Fisher
- SAC/BD-00/08, *Design of Steel Moment Frame Model Buildings in Los Angeles, Seattle and Boston*, by P. Clark
- SAC/BD-00/09, *Benchmarking of Analysis Programs for SMRF System Performance Studies*, by A. G. and H. Krawinkler
- SAC/BD-00/10, *Loading Histories for Seismic Performance Testing of SMRF Components and Assemblies*, by H. Krawinkler, A. Gupta, R. Medina and N. Luco
- SAC/BD-00/11, *Development of Improved Post-Earthquake Inspection Procedures for Steel Moment Frame Buildings*, by P. Clark
- SAC/BD-00/12, *Evaluation of the Effect of Welding Procedure on the Mechanical Properties of FCAW-S and SMAW Weld Metal Used in the Construction of Seismic Moment Frames*, by M. Q. Johnson
- SAC/BD-00/13, *Preliminary Evaluation of Heat Affected Zone Toughness in Structural Shapes Used in the Construction of Seismic Moment Frames*, by M. Q. Johnson
- SAC/BD-00/14, *Evaluation of Mechanical Properties in Full-Scale Connections and Recommended Minimum Weld Toughness for Moment Resisting Frames*, by M. Q. Johnson, W. Mohr, and J. Barsom
- SAC/BD-00/15, *Simplified Design Models for Predicting the Seismic Performance of Steel Moment Frame Connections*, by C. Roeder, R.G. Coons, and M. Hoit
- SAC/BD-00/16, *SAC Phase 2 Test Plan*, by C. Roeder
- SAC/BD-00/17, *Behavior and Design of Radius-Cut, Reduced Beam Section Connections*, by M. Engelhardt, G. Fry, S. Johns, M. Venti, and S. Holliday
- SAC/BD-00/18, *Test of a Free Flange Connection with a Composite Floor Slab*, by M. Venti and M. Engelhardt
- SAC/BD-00/19, *Cyclic Testing of a Free Flange Moment Connection* by C. Gilton, B. Chi, and C. M. Uang
- SAC/BD-00/20, *Improvement of Welded Connections Using Fracture Tough Overlays*, by James Anderson, J. Duan, P. Maranian, and Y. Xiao
- SAC/BD-00/21, *Cyclic Testing of Bolted Moment End-Plate Connections*, by T. Murray and E. Sumner
- SAC/BD-00/22, *Cyclic Response of RBS Moment Connections: Loading Sequence and Lateral Bracing Effects*, by Q.S. Yu, C. Gilton, and C. M. Uang

SAC/BD-00/23, *Cyclic Response of RBS Moment Connections: Weak Axis Configuration and Deep Column Effects*, by C. Gilton, B. Chi, and C. M. Uang

SAC/BD-00/24, *Development and Evaluation of Improved Details for Ductile Welded Unreinforced Flange Connections*, by J.M. Ricles, C. Mao, L.W. Lu, and J. Fisher

SAC/BD-00/25, *Performance Prediction and Evaluation of Steel Special Moment Frames for Seismic Loads*, by K. Lee and D. A. Foutch

SAC/BD-00/26, *Performance Prediction and Evaluation of Low Ductility Steel Moment Frames for Seismic Loads*, by S. Yun and D. A. Foutch

SAC/BD-00/27, *Steel Moment Resisting Connections Reinforced with Cover and Flange Plates*, by T. Kim, A.S. Whittaker, V.V. Bertero, and A.S.J. Gilani

SAC/BD-00/28, *Failure of a Column K-Area Fracture*, by J.M. Barsom and J.V. Pellegrino

SAC/BD-00/29, *Inspection Technology Workshop*, by R. E. Shaw, Jr.

Acronyms.

A, acceleration response
ACAG, air carbon arc gouging
ACIL, American Council of Independent Laboratories
AISC, American Institute for Steel Construction
ANSI, American National Standards Institute
API, American Petroleum Institute
ASD, Allowable Stress Design
ASNT, American Society for Nondestructive Testing
ASTM, American Society for Testing and Materials
ATC, Applied Technology Council
A2LA, American Association for Laboratory Accreditation
AWS, American Welding Society
BB, Bolted Bracket (connection)
BFP, Bolted Flange Plates (connection)
BOCA, Building Officials and Code Administrators
BSEP, Bolted Stiffened End Plate (connection)
BUEP, Bolted Unstiffened End Plate (connection)
CAC-A, air carbon arc cutting
CAWI, Certified Associate Welding Inspector
CJP, complete joint penetration (weld)

CP, Collapse Prevention (performance level)
CUREe, California Universities for Research in Earthquake Engineering
CVN, Charpy V-notch
CWI, Certified Welding Inspector
D, displacement response
DST, Double Split Tee (connection)
DTI, Direct Tension Indicator
EGW, electrogas welding
ELF, equivalent lateral force
ESW, electroslog welding
FCAW-S, flux-cored arc welding – self-shielded
FCAW-G, flux-cored arc welding – gas-shielded
FEMA, Federal Emergency Management Agency
FF, Free Flange (connection)
FR, fully restrained (connection)
GMAW, gas metal arc welding
GTAW, gas tungsten arc welding
HAZ, heat-affected zone
IBC, *International Building Code*
ICBO, International Conference of Building Officials
ICC, International Code Council
IMF, Intermediate Moment Frame

IO, Immediate Occupancy (performance level)	RCSC, Research Council for Structural Connections
ISO, International Standardization Organization	RT, radiographic testing
IWURF, Improved Welded Unreinforced Flange (connection)	SAC, the SAC Joint Venture; a partnership of the Structural Engineers Association of California, the Applied Technology Council, and California Universities for Research in Earthquake Engineering
L, longitudinal	SAW, submerged arc welding
LDP, Linear Dynamic Procedure	SBC, <i>Standard Building Code</i>
LRFD, load and resistance-factor design	SBCCI, Southern Building Code Congress International
LS, Life Safety (performance level)	SCWI, Senior Certified Welding Inspector
LSP, Linear Static Procedure	SEAOC, Structural Engineers Association of California
MCE, Maximum Considered Earthquake	SFRS, seismic-force-resisting system
MMI, Modified Mercalli Intensity	SMAW, shielded metal arc welding
MRS, modal response spectrum	SMF, Special Moment Frame
MRSF, steel moment frame	SP, Side Plate (connection)
MT, magnetic particle testing	SUG, Seismic Use Group
NBC, <i>National Building Code</i>	SW, Slotted Web (connection)
NDE, nondestructive examination	T, transverse
NDP, Nonlinear Dynamic Procedure	TIGW, tungsten inert gas welding
NDT, nondestructive testing	UBC, <i>Uniform Building Code</i>
NEHRP, National Earthquake Hazard Reduction Program	UT, ultrasonic testing
NES, National Evaluation Services	VI, visual inspection
NSP, Nonlinear Static Procedure	WBH, Welded Bottom Haunch (connection)
NVLAP, National Volunteer Laboratory Accreditation Program	WCPF, Welded Cover Plate Flange (connection)
OMF, Ordinary Moment Frame	WFP, Welded Flange Plate (connection)
PGA, peak ground acceleration	WPQR, Welding Performance Qualification Record
PJP, partial joint penetration (weld)	WPS, Welding Procedure Specification
PIDR, pseudo interstory drift ratio	WSMF, welded steel moment frame
PQR, Performance Qualification Record	WT, Welded Top Haunch (connection)
PR, partially restrained (connection)	WTBH, Welded Top and Bottom Haunch (connection)
PT, liquid dye penetrant testing	WUF-B, Welded Unreinforced Flanges – Bolted Web (connection)
PWHT, postweld heat treatment	WUF-W, Welded Unreinforced Flanges – Welded Web (connection)
PZ, panel zone	
QA, quality assurance	
QC, quality control	
QCP, Quality Control Plan, Quality Certification Program	
RBS, Reduced Beam Section (connection)	

SAC Phase II Project Participants

FEMA Project Officer

Michael Mahoney
Federal Emergency Management Agency
500 C St. SW, Room 404
Washington, DC 20472

FEMA Technical Advisor

Robert D. Hanson
Federal Emergency Management Agency
DFO Room 353
P.O. Box 6020
Pasadena, CA 91102-6020

Joint Venture Management Committee (JVMC)

William T. Holmes, Chair
Rutherford and Chekene
303 Second St., Suite 800 North
San Francisco, CA 94107

Christopher Rojahn
Applied Technology Council
555 Twin Dolphin Dr., Suite 550
Redwood City, CA 94065

Edwin T. Huston
Smith & Huston, Inc.
8618 Roosevelt Way NE
Seattle, WA 98115

Arthur E. Ross
Cole/Yee/Shubert & Associates
2500 Venture Oaks Way, Suite 100
Sacramento, CA 95833

Robert Reitherman
California Universities for Research in
Earthquake Engineering
1301 South 46th St.
Richmond, CA 94804

Robin Shepherd
Earthquake Damage Analysis Corporation
40585 Lakeview Drive, Suite 1B
P.O. Box 1967
Big Bear Lake, CA 92315

Project Management Committee (PMC)

Stephen A. Mahin, Project Manager
Pacific Earthquake Engr. Research Center
University of California
Berkeley, CA 94720

William T. Holmes, JVMC
Rutherford and Chekene
303 Second St., Suite 800 North
San Francisco, CA 94107

Ronald O. Hamburger, Project Director for
Project Development
EQE International
1111 Broadway, 10th Floor
Oakland, CA 94607-5500

Christopher Rojahn, JVMC
Applied Technology Council
555 Twin Dolphin Dr., Suite 550
Redwood City, CA 94065

James O. Malley, Project Director for
Topical Investigations
Degenkolb Engineers
225 Bush St., Suite 1000
San Francisco, CA 94104-1737

Robin Shepherd, JVMC
Earthquake Damage Analysis Corporation
40585 Lakeview Drive, Suite 1B
P.O. Box 1967
Big Bear Lake, CA 92315

Peter W. Clark, Technical Assistant to PMC
SAC Steel Project Technical Office
1301 South 46th St.
Richmond, CA 94804

Project Administration

Allen Paul Goldstein, Project Administrator
A.P. Goldstein Associates
1621B 13th Street
Sacramento, CA 95814

Lori Campbell, Assistant to the Project
Administrator
1621 B 13th Street
Sacramento, CA 95628

Lee Adler
Structural Engineers Association of
California
1730 I Street, Ste. 240
Sacramento, CA 95814

Project Oversight Committee (POC)

William J. Hall, Chair
3105 Valley Brook Dr.
Champaign, IL 61821

John L. Gross
National Institute of Stds. & Technology
Building and Fire Research Lab,
Building 226, Room B158
Gaithersburg, MD 20899

Shirin Ader
International Conference of Building
Officials
5360 Workman Mill Rd.
Whittier, CA 90601-2298

James R. Harris
J.R. Harris and Co.
1580 Lincoln St., Suite 550
Denver, CO 80203-1509

John M. Barsom
Barsom Consulting, Ltd.
1316 Murray Ave, Suite 300
Pittsburgh, PA 15217

Richard Holguin
520 Kathryn Ct.
Nipomo, CA 93444

Roger Ferch
Herrick Corporation
7021 Koll Center Parkway
P.O Box 9125
Pleasanton, CA 94566-9125

Nestor Iwankiw
American Institute of Steel Construction
One East Wacker Dr., Suite 3100
Chicago, IL 60601-2001

Theodore V. Galambos
University of Minnesota
122 CE Building, 500 Pillsbury Dr. SE
Minneapolis, MN 55455

Roy Johnston
Brandow & Johnston Associates
1600 West 3rd St.
Los Angeles, CA 90017

Leonard Joseph
Thornton-Tomasetti Engineers
641 6th Ave., 7th Floor
New York, NY 10011

John Theiss
EQE/Theiss Engineers
1848 Lackland Hills Parkway
St. Louis, MO 63146-3572

Duane K. Miller
The Lincoln Electric Company
22801 St. Clair Ave.
Cleveland, OH 44117-1194

John H. Wiggins
J.H. Wiggins Company
1650 South Pacific Coast Hwy, Suite 311
Redondo Beach, CA 90277

Team Leaders for Topical Investigations

Douglas A. Foutch
University of Illinois
MC-250, 205 N. Mathews Ave.
3129 Newmark Civil Engineering Lab
Urbana, IL 61801

Helmut Krawinkler
Department of Civil Engineering
Stanford University
Stanford, CA 94305

Karl H. Frank
University of Texas at Austin
10100 Burnet Rd.
Ferguson Lab, P.R.C. #177
Austin, TX 78758

Charles W. Roeder
University of Washington
233-B More Hall FX-10
Dept. of Building and Safety
Seattle, WA 98195-2700

Matthew Johnson
Edison Welding Institute
1250 Arthur E. Adams Drive
Columbus, OH 43221

L. Thomas Tobin
Tobin and Associates
134 California Ave.
Mill Valley, CA 94941

Lead Guideline Writers

John D. Hooper
Skilling Ward Magnusson Barkshire, Inc.
1301 Fifth Avenue, Suite 3200
Seattle, WA 98101-2699

C. Mark Saunders
Rutherford & Chekene
303 Second St., Suite 800 North
San Francisco, CA 94107

Lawrence D. Reaveley
University of Utah
Civil Engineering Dept.
3220 Merrill Engineering Building
Salt Lake City, UT 84112

Robert E. Shaw
Steel Structures Technology Center, Inc.
42400 W Nine Mile Road
Novi, MI 48375-4132

Thomas A. Sabol
Englekirk & Sabol Consulting Engineers
P.O. Box 77-D
Los Angeles, CA 90007

Raymond H. R. Tide
Wiss, Janney, Elstner Associates, Inc.
330 Pfingsten Road
Northbrook, IL 60062-2095

C. Allin Cornell, Associate Guideline Writer
Stanford University
Terman Engineering Center
Stanford, CA 94305-4020

Technical Advisory Panel (TAP) for Materials and Fracture

John M. Barsom, POC
Barsom Consulting, Ltd.
1316 Murray Ave, Suite 300
Pittsburgh, PA 15217

Serge Bouchard*
TradeARBED
825 Third Avenue, 35th Floor
New York, NY 10022

Michael F. Engestrom*
Nucor-Yamato Steel
P.O. Box 678
Frederick, MD 21705-0678

Karl H. Frank, Team Leader
University of Texas at Austin
10100 Burnet Rd.
Ferguson Lab, P.R.C. #177
Austin, TX 78758

Nestor Iwankiw*
American Institute of Steel Construction
One East Wacker Dr., Suite 3100
Chicago, IL 60601-2001

Dean C. Krouse*
705 Pine Top Drive
Bethlehem, PA 18017

Frederick V. Lawrence
University of Illinois at Urbana-Champaign
205 N. Mathews Ave.
Room 2129 Newmark Lab
Urbana, IL 61801

Robert F. Preece
Preece, Goudie & Associates
100 Bush St., Suite 410
San Francisco, CA 94104

Raymond H. R. Tide, Guideline Writer
Wiss, Janney, Elstner Associates, Inc.
330 Pfingsten Road
Northbrook, IL 60062-2095

TAP for Welding and Inspection

John M. Barsom
Barsom Consulting, Ltd.
1316 Murray Ave, Suite 300
Pittsburgh, PA 15217

John W. Fisher
Lehigh University
117 ATLSS Drive
Bethlehem, PA 18015-4729

J. Ernesto Indacochea
University of Illinois at Chicago
Civil and Materials Engineering (mc 246)
842 West Taylor Street
Chicago, IL 60607

Matthew Johnson, Team Leader
Edison Welding Institute
1250 Arthur E. Adams Drive
Columbus, OH 43221

David Long
PDM Strockal, Inc.
2324 Navy Drive
Stockton, CA 95206

Duane K. Miller, POC
The Lincoln Electric Company
22801 St. Clair Ave.
Cleveland, OH 44117-1194

Robert Pyle*
AISC Marketing
10101 South State Street
Sandy, Utah 84070

Douglas Rees-Evans*
Steel Dynamics, Inc.
Structural Mill Division
2601 County Road 700 East
Columbia City, IN 46725

Richard I. Seals
P.O. Box 11327
Berkeley, CA 94712-2327

Robert E. Shaw, Guideline Writer
Steel Structures Technology Center, Inc.
42400 W Nine Mile Road
Novi, MI 48375-4132

TAP for Connection Performance

Charlie Carter*
American Institute of Steel Construction
One East Wacker Drive, Suite 3100
Chicago, IL 60601-2001

Robert H. Dodds
University of Illinois at Urbana-Champaign
205 N. Mathews Ave.
2129 Newmark Lab
Urbana, IL 61801

Roger Ferch, POC
Herrick Corporation
7021 Koll Center Parkway
P.O. Box 9125
Pleasanton, CA 94566-9125

John D. Hooper, Guideline Writer
Skilling Ward Magnusson Barkshire, Inc.
1301 Fifth Avenue, Suite 3200
Seattle, WA 98101-2699

Egor Popov
University of California at Berkeley
Department of Civil and Environmental
Engineering, Davis Hall
Berkeley, CA 94720

Steve Powell*
SME Steel Contractors
5955 W. Wells Park Rd.
West Jordan, UT 84088

Charles W. Roeder, Team Leader
University of Washington
233-B More Hall FX-10
Dept. of Building and Safety
Seattle, WA 98195-2700

Stanley T. Rolfe
University of Kansas
Civil Engineering Department
2006 Learned Hall
Lawrence, KS 66045-2225

Rick Wilkinson*
Gayle Manufacturing Company
1455 East Kentucky
Woodland, CA 95695

TAP for System Performance

Jacques Cattan*
American Institute of Steel Construction
One East Wacker Drive, Suite 3100
Chicago, IL 60601-2001

Gary C. Hart
Hart Consultant Group
The Water Garden, Ste. 670E
2425 Olympic Blvd.
Santa Monica, CA 90404-4030

Y. Henry Huang*
Los Angeles County Dept. of Public Works
900 S. Fremont Avenue, 8th Floor
Alhambra, CA 91803

Helmut Krawinkler, Team Leader
Department of Civil Engineering
Stanford University
Stanford, CA 94305

Dennis Randall*
SME Steel Contractors
5955 West Wells Park Road
West Jordan, UT 84088

Andrei M. Reinhorn
State University of New York at Buffalo
Civil Engineering Department
231 Ketter Hall
Buffalo, NY 14260

Arthur E. Ross, JVMC
Cole/Yee/Shubert & Associates
2500 Venture Oaks Way, Suite 100
Sacramento, CA 95833

C. Mark Saunders, Guideline Writer
Rutherford & Chekene
303 Second St., Suite 800 North
San Francisco, CA 94107

W. Lee Shoemaker*
Metal Building Manufacturers Association
1300 Summer Avenue
Cleveland, OH 44115

John Theiss, POC
EQE/Theiss Engineers
1848 Lackland Hills Parkway
St. Louis, MO 63146-3572

TAP for Performance Prediction and Evaluation

Vitelmo V. Bertero
University of California at Berkeley
Pacific Earthquake Engr. Research Center
1301 S. 46th St.
Richmond, CA 94804

Bruce R. Ellingwood
Johns Hopkins University
Department of Civil Engineering
3400 N. Charles St.
Baltimore, MD 21218

Douglas A. Foutch, Team Leader
University of Illinois
MC-250, 205 N. Mathews Ave.
3129 Newmark Civil Engineering Lab
Urbana, IL 61801

Theodore V. Galambos, POC
University of Minnesota
122 CE Building, 500 Pillsbury Dr. SE
Minneapolis, MN 55455

Lawrence G. Griffis
Walter P. Moore & Associates
3131 Eastside, Second Floor
Houston, TX 77098

Edwin T. Huston, JVMC
Smith & Huston, Inc.
8618 Roosevelt Way NE
Seattle, WA 98115

Harry Martin*
American Iron and Steel Institute
11899 Edgewood Road, Suite G
Auburn, CA 95603

Tom Schlafly*
American Institute of Steel Construction
One East Wacker Drive, Suite 3100
Chicago, IL 60601-2001

Thomas A. Sabol, Guideline Writer
Englekirk & Sabol Consulting Engineers
P.O. Box 77-D
Los Angeles, CA 90007

Technical Advisors

Norm Abrahamson
Pacific Gas & Electric
P.O. Box 770000, MC N4C
San Francisco, CA 94177

Robert Kennedy
RPK Structural Mechanics Consultants
18971 Villa Terr
Yorba Linda, CA 92886

C.B. Crouse
URS – Dames and Moore
2025 First Avenue, Suite 500
Seattle, WA 98121

Social Economic and Policy Panel

Martha Cox-Nitikman
Building and Owners and Managers
Association, Los Angeles
700 South Flower, Suite 2325
Los Angeles, CA 90017

Alan Merson
Morley Builders
2901 28th Street, Suite 100
Santa Monica, CA 90405

Karl Deppe
27502 Fawnskin Dr.
Rancho Palos Verdes, CA 90275

Joanne Nigg
University of Delaware
Disaster Research Center
Newark, DE 19716

Eugene Lecomte
Institute for Business and Home Safety
6 Sheffield Drive
Billerica, MA 01821

William Petak
University of Southern California
Lewis Hall, Room 201
650 Childs Way
Los Angeles, CA 90089

James Madison
Attorney at Law, Mediator and Arbitrator
750 Menlo Avenue, Suite 250
Menlo Park, CA 94025

Francine Rabinovitz
Hamilton, Rabinovitz and Alschuler
1990 South Bundy Drive, Suite 777
Los Angeles, CA 90025

Dennis Randall
SME Steel Contractors
5955 West Wells Park Road
West Jordan, UT 84088

Stephen Toth
TIAA-CREF
730 Third Avenue
New York, NY 10017-3206

David Ratterman
Stites and Harbison
400 West Market St., Suite 1800
Louisville, KY 40202-3352

John H. Wiggins, POC
J.H. Wiggins Company
1650 South Pacific Coast Hwy, Suite 311
Redondo Beach, CA 90277

L. Thomas Tobin, Panel Coordinator
134 California Ave.
Mill Valley, CA 94941

Performance of Steel Buildings in Past Earthquakes Subcontractors

David Bonowitz
887 Bush, No. 610
San Francisco, CA 94108

Peter Maranian
Brandow & Johnston Associates
1660 West Third Street
Los Angeles, CA 90017

Peter Clark
SAC Steel Project Technical Office
1301 South 46th St.
Richmond, CA 94804

Terrence Paret
Wiss Janney Elstner Associates, Inc.
2200 Powell St. Suite 925
Emeryville, CA 94602

Michael Durkin
Michael Durkin & Associates
22955 Leanora Dr.
Woodland Hills, CA 91367

Maryann Phipps
Degenkolb Engineers
225 Bush Street, Suite 1000
San Francisco, CA 94104

James Goltz
California Institute of Technology
Office of Earthquake Programs
Mail Code 252-21
Pasadena, CA 91125

Allan Porush
Dames & Moore
911 Wilshire Blvd., Suite 700
Los Angeles, CA 90017

Bruce Maison
7309 Lynn Ave
Elcerrito, CA 94530

Access Current Knowledge Subcontractors

David Bonowitz
887 Bush, No. 610
San Francisco, CA 94108

Stephen Liu
Colorado School of Mines
Mathematics and Computer Science
Department
Golden, CO 80401

Materials and Fracture Subcontractors

Robert Dexter
University of Minnesota
122 Civil Engineering Building
500 Pillsbury Drive SE
Minneapolis, MN 55455-0116

Karl H. Frank
University of Texas at Austin
10100 Burnet Rd.
Ferguson Lab, P.R.C. #177
Austin, TX 78758

Welding and Inspection Subcontractors

Pingsha Dong / Tom Kilinski
Center for Welded Structures Research
Battelle Memorial Institute
501 King Avenue
Columbus, OH 43201-2693

Glenn M. Light / George Gruber
Southwest Research Institute
6220 Culebra Road, P. O. Drawer 28510
San Antonio, TX 78228-0510

Matthew Johnson
Edison Welding Institute
1250 Arthur E. Adams Drive
Columbus, OH 43221

William C. Mohr
Edison Welding Institute
1250 Arthur E. Adams Drive
Columbus, OH 43221

Connection Performance Subcontractors

Gregory Deierlein
Stanford University
Terman Engineering Center
Department of Civil and Environmental Engr.
Stanford, CA 94305-4020

Sherif El-Tawil / Sashi Kunnath
University of Central Florida
Civil and Environmental Engr. Department
Orlando, FL. 32816-2450

Charles W. Roeder
University of Washington
233-B More Hall FX-10
Seattle, WA 98195-2700

Anthony Ingrassia
Cornell University
School of Civil Engineering
363 Hollister Hall
Ithaca, NY 14853

System Performance Subcontractors

Paul Somerville
Woodward-Clyde Federal Services
566 El Dorado St., Suite 100
Pasadena, CA 91101-2560

Andrei M. Reinhorn
State University of New York at Buffalo
Civil Engineering Department
231 Ketter Hall
Buffalo, NY 14260

Farzad Naeim
John A. Martin & Associates
1212 S. Flower Ave.
Los Angeles, CA 90015

C. Allin Cornell
Stanford University
Terman Engineering Center
Stanford, CA 94305-4020

Helmut Krawinkler
Dept. of Civil Engineering
Stanford University
Stanford, CA 94305

Kazuhiko Kasai
Tokyo Institute of Technology
Structural Engineering Research Center
Nagatsuta, Midori-Ku
Yokohama 226-8503, JAPAN

Gregory MacRae
University of Washington
Civil Engineering Department
Seattle, WA 98195-2700

Bruce F. Maison
7309 Lynn Avenue
El Cerrito, CA 94530

Performance Prediction and Evaluation Subcontractors

James Anderson
University of Southern California
Civil Engineering Department
Los Angeles, CA 90089-2531

Gary C. Hart
Department of Civil and Environmental
Engineering
University of California
Los Angeles, CA 90095

Douglas A. Foutch
University of Illinois
MC-250, 205 N. Mathews Ave.
3129 Newmark Civil Engineering Lab
Urbana, IL 61801

Y.K. Wen
University of Illinois
3129 Newmark Civil Engineering Lab
205 N. Mathews Ave.
Urbana, IL 61801

Testing Subcontractors

Subhash Goel / Bozidar Stojadinovic
University of Michigan
Civil Engineering Department
Ann Arbor, MI 48109

Thomas Murray
Virginia Tech, Dept. of Civil Engineering
200 Patton Hall
Blacksburg, VA 24061

Roberto Leon
Georgia Institute of Technology
School of Civil & Environmental Engr.
790 Atlantic Ave.
Atlanta, GA 30332-0355

James M. Ricles / Le-Wu Lu
Lehigh University
c/o ATLSS Center
117 ATLSS Drive, H Building
Bethlehem, PA 18015-4729

Vitelmo V. Bertero / Andrew Whittaker
UC Berkeley
Pacific Earthquake Engr. Research Center
1301 S. 46th St.
Richmond, CA 94804

John M. Barsom
Barsom Consulting, Ltd.
1316 Murray Ave, Suite 300
Pittsburgh, PA 15217

Hassan Astaneh
University of California at Berkeley
Dept. of Civil and Environmental Engr.
781 Davis Hall
Berkeley, CA 94720

Stephen Schneider
University of Illinois at Urbana-Champaign
3106 Newmark Civil Engr. Lab, MC-250
205 N. Mathews Avenue
Urbana, IL 61801

Michael Engelhardt
University of Texas at Austin
Ferguson Laboratory
10100 Burnet Road, Building 177
Austin, TX 78712-1076

Matthew Johnson
Edison Welding Institute
1250 Arthur E. Adams Drive
Columbus, OH 43221

Gary T. Fry
Texas A&M University
Department of Civil Engineering
Constructed Facilities Division, CE/TTI
Building, Room 710D
College Station, TX 77843-3136

James Anderson
University of Southern California
Civil Engineering Department
Los Angeles, CA 90089-2531

Chia-Ming Uang
University of California at San Diego
Dept. of AMES, Division of Structural Engr.
409 University Center
La Jolla, California 92093-0085

Bozidar Stojadinovic
Dept. of Civil & Environmental Engr.
University of California
Berkeley, CA 94720

Inspection Procedure Consultants

Thomas Albert
Digiray Corporation
2235 Omega Road, No. 3
San Ramon, CA 94583

Andrey Mishin
AS & E High Energy Systems
330 Keller Street, Building 101
Santa Clara, CA 95054

Randal Fong
Automated Inspection Systems, Inc.
4861 Sunrise Drive, Suite 101
Martinez, CA 94553

Robert Shaw
Steel Structures Technology Center, Inc.
42400 W. Nine Mile Road
Novi, MI 48375-4132

Andre Lamarre
R.D Tech, Inc.
1200 St. Jean Baptiste, Suite 120
Quebec City, Quebec, Canada G2ZE 5E8

Carlos Ventura
Dept of Civil Engineering
University of British Columbia
2324 Main Hall
Vancouver, BC, Canada V6T 1Z4

Glenn Light
Southwest Research Institute
6220 Culebra Road
San Antonio, TX 78228

Guideline Trial Applications Subcontractors

John Hopper
Skilling Ward Magnusson Barkshire, Inc.
1301 Fifth Avenue, Suite 320
Seattle WA 98101-2699

Lawrence Novak
Skidmore, Owings, and Merrill
224 S. Michigan Ave, Suite 1000
Chicago, IL 60604

Leonard Joseph
Thornton-Tomassetti Engineers
641 6th Avenue, 7th Floor
New York, NY 10011

Maryann Phipps
Degenkolb Engineers
225 Bush Street, Suite 1000
San Francisco, CA 94104

Economic and Social Impact Study Subcontractors

Ronald Eguchi
EQE Engineering and Design
300 Commerce Dr., Ste. 200
Irvine, CA 92602

Charles Kircher
Charles Kircher & Associates
1121 San Antonio Road, Suite D-202
Palo Alto, CA 94303

Martin Gordon / Peter Morris
Adamson Associates
170 Columbus Avenue
San Francisco, CA 94133

Lizandro Mercado
Brandow & Johnston Associates
1600 West 3rd St.
Los Angeles, CA 90017

Richard Henige
Lemessurier Consultants Inc.
675 Massachusetts Ave.
Cambridge, MA 02139-3309

Greg Schindler
KPFF Consulting Engineers
1201 3rd Ave.
Seattle, WA 98101-3000

Report Production and Administrative Services

A. Gerald Brady, Technical Editor
Patricia A. Mork, Administrative Asst.
Peter N. Mork, Computer Specialist
Bernadette A. Mosby, Operations Admin.
Michelle S. Schwartzbach, Pub. Specialist
Applied Technology Council
555 Twin Dolphin Drive, Suite 550
Redwood City, CA 94065

Carol Cameron, Publications Coordinator
Ericka Holmon, Admin. Assistant
California Universities for Research in
Earthquake Engineering
1301 S. 46th Street
Richmond, CA 94804

*indicates industrial or organizational contact representative