INTEGRATION OF ANALYTICAL INVESTIGATIONS
ON THE FRACTURE BEHAVIOR
OF WELDED MOMENT RESISTING CONNECTIONS

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Integrative Analytical Investigations on the Fracture Behavior of Welded Moment Resisting Connections

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Final Project Report – SAC Task 5.3.3

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EXECUTIVE SUMMARY

The objective of this investigation is to utilize finite element analyses to investigate the fracture behavior of welded beam-column connections and, thereby, examine how fracture resistance is influenced by various design and detailing parameters. A related objective is to help integrate fracture-related data from other SAC investigations on materials, welding/joining and connection testing. The ultimate goal is to provide behavioral information to guide the development of guidelines and acceptance criteria for the design of fracture resistant welded beam-column connections.

This study is a follow-up to a preliminary investigation by the authors, conducted under SAC Subtask 5.3.1, to examine “pre-Northridge” style connections tested during Phase I of SAC. The present investigation extends the earlier study to address a broader range of design and detailing parameters and fracture effects. Elastic and inelastic finite element fracture analyses are used to evaluate fracture toughness demands in terms of mode I stress intensity factor ($K_I$) and Crack Tip Opening Displacement (CTOD). In addition, advanced analyses that employ a micro-mechanical fracture criterion (Stress Modified Critical Strain) are used to examine ductile crack initiation in locations without an initial flaw. Computed fracture demands are evaluated in light of test data from relevant material, weldment and connection tests. Parameters investigated include the following:

- weld flaw locations
- built-up welds with filleted reinforcement
- variations in beam and column sizes
- relative strength of beam to joint panel zone
- influence of continuity plates
- significance of welding-induced residual stresses
- influence of weld access hole geometry
- connections with Reduced Beam Sections (RBS)
- through-thickness fractures in column flanges

Data from the analyses substantiate observations from connection tests which indicate that improved weld details and higher toughness materials alone are not sufficient to reliably provide the inelastic deformation capacity required for seismic design. Standard, i.e., “pre-Northridge” style, connections made with notch toughness rated weld and base metals (CVN > 40 to 50 ft-lbs at 70°F) and small initial flaws ($a_0 < 0.1$ inch) are shown capable of reliably achieving their full plastic strength, but toughness demands required to sustain larger inelastic hinge rotations generally exceed the toughness of common notch tough weld metals. On the other hand, the
analyses do confirm the effectiveness of improved connections, such as the Reduced Beam Section (RBS) detail, to limit toughness demands within attainable limits. Analyses of RBS connections indicate that control of panel zone deformations is essential to limit fracture toughness demands in the critical beam flange weld. The sensitivity of fracture toughness demands to panel zone strength, and consequently panel zone deformations, is also apparent in standard (non-RBS) details. Other general observations and conclusions from the analyses include the following:

• Weld yield strength overmatching that is generally achieved with E70 weld metal and A572 Gr. 50 base metal (based on their average yield strengths of $F_{yw} = 65$ ksi and $F_{yb} = 55$ ksi, respectively) is beneficial for reducing toughness demands at weld root flaws at the weld-to-column interface. However, overmatching does not offer much if any such benefit for flaws at the weld-to-beam flange interface.

• Fracture toughness demands caused by the gap behind the backing bar are shown to be insensitive to the backing bar thickness or the fusion length between the backing bar and weld. Fillet welds used to seal the backing bar gap can reduce toughness demands at the built-in crack tip, however, their effectiveness depends on there being a sufficient fusion length between the backing bar and the groove weld to transfer stress into the seal weld.

• Inelastic toughness demands for flaws located at the weld-to-beam interface are generally about twice that of flaws at the weld-to-column interface (weld root). This suggests that more stringent acceptance criteria are appropriate for flaws at the weld-to-beam interface, particularly for weld toe cracks in the top-flange. Additionally, flaws on the inside faces of the beam flanges (the top of the bottom flange and bottom of the top flange) have much smaller toughness demands than flaws on the outside faces (the extreme fiber locations). This helps to explain the prevalence of bottom flange, versus top flange, fractures when weld backing bars are left in place.

• Welding-induced residual stresses appear to be most significant at low stress levels where the behavior is elastic. In such cases, analyses indicate that the residual stresses impose an inherent toughness demand of about $K_I \approx 20$ ksi$\sqrt{\text{in}}$ at the weld root flaw. At larger inelastic deformations the change in toughness demand due to residual stresses becomes less significant, relative to other factors, due to large-scale yielding.

• Analyses of a connection with a W14 x 550 column indicate that when the column flanges are sufficiently thick (in this case, $t_{fc} = 3.82$ inch), the presence of continuity plates does not have a significant effect on fracture toughness demands at the beam flange weld. However, for columns with thinner flanges the presence of continuity plates can significantly reduce the toughness demand. For example, in a connection with a W21 x 131 column ($t_{fc} = 0.96$ inch), the addition of continuity plates decreases the maximum toughness demand by roughly 60% relative to the case without continuity plates.

• Analyses of connections with different size beams and columns indicate that toughness demand is most sensitive to the column flange thickness and the joint panel zone strength. Even when continuity plates are present, the fracture toughness demand at the weld root increases with decreasing flange thickness, implying that toughness demands are generally
larger for deep column members with thin flanges as compared to shallower heavier columns with thick flanges. Beyond this, the toughness demand also increases with increasing shear deformation of the joint panel zone.

- Comparative analyses between the SAC Task 5.12 through-thickness pull-plate test specimens and beam-column connections confirm that critical stress and strain conditions generated in the pull-plates exceed those in the beam-column connections. This indicates that, insofar as the materials in the pull-plate tests match those used in practice, through thickness column fractures are unlikely to occur in welded beam-column connections.

- Comparative analyses of SAC Task 7.05 T-stub weldment tests and beam-column connections show that the stress/strain states and fracture demands in the two can vary considerably. Thus, results from the T-stub tests are not directly transferable to beam-column connections. However, when interpreted through analytical fracture analyses, results from the T-stub tests can be used to establish the in situ fracture toughness of groove welds and the influence of welding materials and procedures on toughness.
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CONTENTS

1. Introduction
   1.1 General
   1.2 Related SAC Investigations
   1.3 Connection Geometry and Design Variables
   1.4 Scope and Objectives
   1.5 Organization of Report

2. Material Properties, Fracture Criteria, and Modeling Techniques
   2.1 Overview
   2.2 Stress-Strain Properties of A572 Gr. 50 Base Metal
   2.3 Stress-Strain Properties of Weld Metals
   2.4 CVN Toughness of Base Metal
   2.5 CVN Toughness of Weld Metal
   2.6 Fracture Toughness Indices
   2.7 Finite Element Modeling Techniques and Parameters
   2.8 Micro-Mechanical SMCS Ductile Crack Initiation Criterion

3. Elastic Behavior and $K_I$ Toughness Demands
   3.1 Overview
   3.2 General Behavior and Trends
      3.2.1 Basic Characteristics of Stress Intensity at Weld Root Defect
      3.2.2 Effect of Flaw Location
      3.2.3 Two- versus Three-Dimensional Behavior
   3.3 Parametric Analyses for $K_I$ at Column-to-Weld Interface
      3.3.1 Calibration Approach
      3.3.2 Parametric Study using 2D Analyses
      3.3.3 Calibration for Three-Dimensional Effects
      3.3.4 Summary of Predictive Equation for Edge Crack
      3.3.5 Calibration of Predictive Equation for Interior Crack
   3.4 Residual Stress Effects
   3.5 Summary

4. Inelastic Behavior and CTOD Toughness Demands
   4.1 Overview
   4.2 General Behavior and Trends
   4.3 Analyses of SAC/Michigan Connection Tests
   4.4 Parametric Study of Panel Zone, Flange Thickness and Continuity Plates
   4.5 Residual Stresses and Weld Matching Ratio
   4.6 Influence of Weld Access Hole and Residual Stresses using SMCS Model
   4.7 Comparative Study of RBS Connection Detail
4.8 Summary and Design Implications

5. Transferability of Fracture Data Between Pull-Plate and Connection Tests

5.1 Overview of Pull-Plate Weldment Studies
5.2 SAC Task 7.05 T-Stub Weld Tests
   5.2.1 Basic Comparison of Elastic and Inelastic Behavior
   5.2.2 Sensitivity of T-stub to Weld Strength, Residual Stresses, and Fillet Reinforcement
   5.2.3 Modification of T-stub Specimen
   5.2.4 Interpretation and Design Implications of T-stub Tests
5.3 SAC Task 5.12 Through-Thickness Pull-Plate Tests
   5.3.1 Overview of Test Data
   5.3.2 SMCS Analyses of Pull-Plate
   5.3.3 SMCS Analyses of Beam-Column Connection
   5.3.4 Implications on Through-Thickness Failure Mode

6. Summary and Conclusions

   6.1 Summary
   6.2 Conclusions
   6.3 Implications for Design and Acceptance Criteria

REFERENCES

TABLES

FIGURES

APPENDICIES

Loading Rate Effects on Fracture
Table of SI Conversions
1. Introduction

1.1 General

Investigations during Phase I of the SAC Joint Venture established that premature cracking of welded moment connections during the 1994 Northridge earthquake resulted from a combination of factors – most notably, high stress/strain demands coupled with large inherent flaws and stress risers, deficient field welding, and over-reliance on low-toughness weld metal. Simply stated, these factors created conditions where imposed fracture toughness demands exceeded the available material toughness. Strategies to improve connection behavior can be broadly categorized between efforts to reduce fracture toughness demands imposed on the material and increase the inherent toughness capacity of the material. Modifications to reduce toughness demands involve, for example, removing weld backing bars and minimizing defects through better welding procedures and quality control. Given the large inelastic strains at the beam flange welds, these modifications alone are often not sufficient to reduce toughness demands to the degree necessary to ensure ductile connection behavior. Further improvements can be achieved by reconfiguring the connection to reduce overall loading demands on the weld, examples of which include the reduced beam section (“dogbone”) and cover-plate details that are designed to shift regions of high stresses and strains away from fracture critical locations. The other aspect of the solution, increasing the material toughness capacity, is achieved by using notch-toughness rated welding consumables and stricter adherence to welding procedures that control heat input, cooling rates, and other factors affecting the basic metallurgy of the weld and heat affected zone. Here again, however, there are limits to the fracture toughness that can be economically achieved using materials and procedures suitable for building construction.

In devising improved connection details and material toughness requirements, an accurate means is needed to determine fracture toughness demands in the connection. The objective of this investigation is to calculate fracture toughness demands in welded beam-column connections using finite element analyses and, thereby, examine how fracture resistance is influenced by various connection details. The ultimate goal is to provide support for the development of guidelines and acceptance criteria for the design of welded beam-column connections. In this study, fracture toughness demands are evaluated in terms of mode I stress intensity factor ($K_I$) and Crack Tip Opening Displacement (CTOD). A micro-mechanical ductile fracture criterion called the Stress Modified Critical Strain (SMCS) is also employed. In many respects, this study is a follow-up to previous fracture analyses by the authors (Chi et al. 1997, 2000) of “pre-Northridge” style connections tested during Phase I of SAC (Shuey et al. 1996, Popov et al. 1996). The present work seeks to further develop and apply the analytical techniques employed by the earlier study to address a broader range of design and detailing parameters.

1.2 Connection Geometry and Design Variables

The primary focus of this investigation is on fractures initiating in or adjacent to the beam flange weld of beam-column moment connections. In details where backing bars are left in place, fractures are most likely to initiate from initial flaws created by the gap behind the backing bar combined with weld root defects. As shown in Fig. 1.1, in bottom flange joints the weld root
defects are usually largest directly beneath the beam web where welding access is limited. Previous analyses (Chi et al. 1997, 2000), which are briefly summarized later in this report, have shown that fracture demands for weld root flaws can be categorized by the weld defect dimension $a_o$, measured from the bottom of the beam flange to the top of the flaw. Weld root flaws are practically unavoidable when backing bars are left in place, which is a strong incentive for removing weld backing bars. However, whether or not backing bars are present, analyses by Chi et al. show that it is the weld root defect length, $a_o$, and not the backing bar gap length, $w$, that is the governing parameter in determining the stress intensity factor at the crack tip. For this reason, the weld backing bar is not included in many of the models featured in this report, even though the intent is to calculate fracture toughness demands in connections with or without weld backing bars. Further, since the finite element fracture analyses in this report are concerned with toughness demands for fracture initiation (as opposed to fracture propagation), the calculated demands are not at all a function of the material toughness. Rather, the material toughness, which is determined largely by the welding electrode type, comes into play when interpreting whether the calculated demands will cause fractures. Thus, the analyses of connections with small weld root defects are representative of fracture toughness demands of pre-Northridge style details (backing bars left in place with low-toughness weld metal) as well as improved details (backing bars removed with higher toughness weld metal).

While this investigation is generally based on the connection configuration shown in Fig. 1.1, many variations of this connection detail are investigated to examine the following parameters and conditions:

- weld flaw location (edge and internal flaws at the weld root and edge flaws at the weld toe)
- built-up welds with filleted reinforcement
- variations in beam and column sizes
- relative strength of beam to joint panel strength
- influence of continuity plates
- significance of welding-induced residual stresses
- influence of weld access hole geometry
- connections with the Reduced Beam Section (RBS) detail

1.3 Scope and Objectives

Developed as a companion to other SAC projects involving testing/analyses of beam-column connections, components and materials, this project has the following inter-related objectives:

1. Integrate information and data from tests and analyses obtained through related SAC projects on connection performance, welding and joining, and material categorization. Primarily, this involves relating data on material properties and weldment pull-plate tests to fracture behavior observed in connections.

2. Extend finite element fracture initiation analyses previously conducted under Task 5.3.1A (Chi et al. 1997) to develop more complete quantitative data on fracture toughness demands at critical regions of various welded connection details.
3. Examine the limitations of finite element idealizations and traditional fracture indices (K_I, J_I, CTOD) for predicting inelastic behavior and fracture initiation under monotonic and cyclic loading as influenced by high stress/strain states and earthquake loading rates encountered in steel moment connections.

4. Provide guidance for the design of fracture resistant connections. One aspect of this involves the development of simple equations that provide approximate quantitative predictions of toughness demands as a function of material properties, connection and weld geometry, and initial flaw characteristics. A second aspect is to provide analysis data to support general guidelines on design parameters, such as weld metal yield strengths, joint panel zone deformations, etc., that significantly influence fracture behavior.

1.4 Organization of Report

Chapter 2 begins with a review of available material data (stress-strain and toughness properties) for base and weld metals that are used in the modeling and interpretation of the fracture analyses. Properties for base metals is based largely on data reported through SAC Task 5.1.1 Characterization of the Material Properties of Rolled Sections, SAC Task 5.1.2 Assess the Influence of the Through-Thickness Properties of Columns and Joint Geometry Upon the Behavior of Beam to Column Flange Welds and an earlier AISC survey of notch toughness data for rolled shapes. Properties for weld metals are based on data from SAC Task 5.2.3 Assess the Sensitivity of the Behavior of Welded Joints to Variations in Welding Procedures, Parameters and Conditions and Task 7.12 Supplemental Weld Tests in Support of Connection Test Program. Following this review of material properties, background information on the fracture analysis modeling techniques are presented.

Chapter 3 is devoted to modeling elastic behavior and determining K_I demands through a systematic study of geometric connection design parameters (e.g., beam and column sizes, flaw sizes and locations, etc.). Included are two- and three-dimensional elastic fracture analyses, and consideration of the extent to which welding-induced residual stresses increase fracture toughness demands. A major outcome of these analyses is the development and calibration of simplified equations to estimate elastic toughness demands (K_I) for edge and interior weld defects as a function of the applied stress, flaw size, flaw type (interior versus edge defect), and the beam and column proportions.

Parameters affecting inelastic behavior and CTOD toughness demand are investigated through two- and three-dimensional inelastic analyses in Chapter 4. Among the connection design and detailing parameters considered are the following (1) beam and column sizes, (2) yield strengths of the beam, column and weld metal, (3) location and size of initial cracks or flaws, (4) welding-induced residual stresses, and (5) detailing questions such as the need for continuity plates, proportioning of Reduced Beam Section details, influence of weld profile and access hole geometry on fracture, etc. Included are analyses of connections modeled after those tested in Phase I of SAC and in SAC Task 7.02 Parametric Tests on Unreinforced Connections and Task 7.06 Parameter Tests on Reduced-Beam Section (RBS) Connections.
Analyses in chapter 5 are intended to relate data from weldment pull-plate tests to beam-column connection behavior. Included are analyses of T-stub tests conducted as part of SAC Task 7.05 Supplemental Testing of Unreinforced Connections with Integration of T-stub Weld Testing and Analysis and pull-plate tests conducted as part of SAC Task 5.12 Assess the Influence of the Through-Thickness Properties of Columns and Joint Geometry Upon the Behavior of Beam to Column Flange Welds. Analyses of the Task 7.05 T-stub tests employ conventional $K_I$ and CTOD fracture analyses, whereas analyses of the Task 5.12 through-thickness pull-plate tests are based on the micro-mechanical Stress Modified Critical Strain (SMCS) criterion.

Finally, chapter 6 summarizes the major conclusions of this investigation and resulting implications on design and acceptance criteria for welded beam-column connections.

1.5 Qualifying Remarks

The analysis results reported herein are one component of a multi-faceted and complex problem to design fracture resistant connections for seismic design, and as such, the results should be interpreted with due consideration of the simplifying assumptions inherent in the analysis models. The analyses imply many assumptions regarding the loading and material behavior. For example, the analyses are conducted under monotonically increasing load and do not take into account reverse cyclic loading, such as the complex loading histories that occur during earthquakes or the cyclic loading protocols applied in laboratory tests. The analyses also do not take into account loading-rate effects, although as described in the appendix, loading rate is expected to be of second-order importance relative to other factors. Finally, while the study addresses the sensitivity of response to select parameters, the analyses are deterministic and do not reflect the many uncertainties encountered in practice. For example, on the fracture demand side, uncertainties arise due to variations in flaw sizes, material yield strengths, strain hardening properties, welding-induced residual stresses, etc. Even more significant is the large variability in the fracture toughness of the materials, including the base metal, weld metal, and heat affected zone. Thus, while the fracture analyses presented herein provide detailed insight that can help guide connection design, final detailing and verification of performance must consider the many additional factors that are outside the scope of this investigation.
2. Material Properties and Fracture Models and Criteria

2.1 Overview

Summarized in this chapter are data and information concerning the material properties, fracture toughness indices, and finite element modeling techniques utilized for this investigation. Mechanical properties of base and weld metals, specifically stress-strain and notch toughness data, are summarized in Sections 2.2 to 2.5. In Sections 2.6 and 2.7, these data are then related to finite element modeling techniques and fracture toughness criteria necessary to conduct and interpret the finite element fracture analyses. Finally, the micro-mechanical Stress Modified Critical Strain (SMCS) criterion for ductile crack initiation is introduced in Section 2.8. For additional information beyond what is report in this chapter, the reader is referred to related reports by Chi et al. (1997) and Chi (1999).

2.2 Stress-Strain Properties of A572 Gr 50 Base Metal

As part of the SAC investigation of material properties, Jaquess et al. (1999) conducted extensive tension tests to characterize the stress-strain behavior of ASTM A572 Grade 50 steel. Shown in Fig. 2.1 is a schematic diagram of the engineering stress-strain properties they reported, and summarized in Fig. 2.2 and Table 2.1 are statistical values of the measured parameters. These data are from tension coupons extracted from the flanges of seventeen hot-rolled W-shapes of A572 Grade 50 material. Key aspects of the stress-strain data (Figs. 2.1 and 2.2) and how they are applied in the finite element fracture analyses are as follows.

- **Yield Plateau Stress ($F_y$)** is considered as characterizing the steel yield strength for the purposes of analytical calculations. As shown in Fig. 2.1, Jaquess et al. (1999) determined the yield stress by the 0.2% offset method. The mean measured yield stress is $F_y = 55$ ksi, which is about 10% larger than the minimum specified yield stress for A572 Grade 50.

- **Upper Yield Point ($F_{uy}$)** is a loading rate dependent characteristic of mild steel. The measured average was $F_{uy} = 57$ ksi, which is about 4% larger than the yield plateau stress.

- **Static Yield Stress ($F_{sy}$)** is a point along the yield plateau measured by holding a zero strain rate for three minutes. The reported average static yield strength, $F_{sy} = 52$ ksi, is about 5% less than $F_y$.

- **Strain at Strain-Hardening ($\varepsilon_{sh}$)** is defined as the point along the stress-strain curve where the strain-hardening region begins. As shown in Fig. 2.1, given that typical stress-strain curves do not show a sharp transition point, $\varepsilon_{sh}$ is determined by the intersection of the tangent to the strain hardening curve and the yield plateau. The mean value of $\varepsilon_{sh} = 0.015$ is about 8 times the initial yield strain.
• Strain-Hardening Modulus \( (E_{sh}) \) is an estimate of the initial slope in the strain-hardening region, determined by the intersection of points shown in Fig. 2.1. The average modulus is \( E_{sh} = 0.0131 \ E = 380 \text{ ksi} \), where the elastic modulus \( E = 29,000 \text{ ksi} \).

• Strain at Ultimate Stress \( (\varepsilon_u) \) is the strain at which the stress-strain curve reaches a maximum, after which the tensile coupon necks down and soon fractures. On average, \( \varepsilon_u = 0.015 \) or about 85 times the yield strain.

• Ultimate Strength \( (F_u) \) is the maximum stress reached in the tensile coupon. The average measured strength of \( F_u = 73 \text{ ksi} \) exceeds the minimum specified strength of 65 ksi for A572 Gr. 50. The resulting average yield ratio of \( F_y/F_u = 55/73 = 0.75 \) is less than the maximum value of 0.85 considered acceptable for seismic design by the AISC Seismic Provisions (1997).

Of these material properties, all except the upper yield point \( (F_{uy}) \) and the static yield stress \( (F_{sy}) \) are reflected in the stress-strain properties of the analytical models. Details on how the engineering stress-strain properties are adjusted to true stress-strain parameters (needed for the large strain finite element analyses) are summarized later.

**Property Variations Due to Coupon Location and Type:** The webs of rolled sections normally have larger yield strengths than the flanges due to greater mechanical working during the milling process. Jaquess et al. (1999) report measured differences of mean web-to-flange strengths of +3 to +5% from three different mills. These small differences are neglected in our finite element analyses.

Stress-strain behavior is usually measured using one of two standard coupon types permitted by ASTM, flat straps or round bars, though differences in the coupon types and sizes can cause variations in local strain rates (particularly in the necking region). As summarized in Table 2.2, data by Jaquess et al. (1999) show that the differences in the material properties for flat strap and bar coupons taken from the same material are less than 3% for \( F_y, F_{sy}, F_u \) and \( \varepsilon_u \), and range between 8% to 17% for \( F_{uy}, \varepsilon_{sh}, \) and \( E_{sh} \). These differences are not explicitly accounted for in the finite element analyses, where the assumed properties are based on the average values reported in Fig. 2.2 and Table 2.1.

**Yield and Ultimate Strengths Reported by Other Investigators:** The average values summarized above are compared to material test data reported by Dexter (1999), Ricles (1999), and Dong (1999) as part of their SAC investigations. Comparisons of yield and ultimate strengths are summarized in Fig. 2.3 and the resulting yield ratios in Fig. 2.4. All of the data designated by mill producers are from coupon tests of rolled shapes by Dexter et al. (1999) and the plate steel values are from Ricles et al. (1999) and Dong et al. (1999). The comparisons in Fig. 2.3 and 2.4 indicate that the average values characterize the steel properties fairly well, although there can be significant variations between producers which can exceed one sigma values (about 10%) compared to the mean. Data from coupons measuring through thickness properties indicate that these are not markedly different than the longitudinal properties, except that the through thickness data do exhibit larger variability.
2.3 Stress-Strain Properties of Weld Metals

Summarized in Fig. 2.5 and 2.6 are yield and ultimate strengths of several weld metals from tests reported by Johnson (1999). E70T-4 is a Flux Core Arc Weld (FCAW) that was commonly used in welded steel moment connections before the 1994 Northridge earthquake, and E71T-8, E70T-6, and E70TG-K2 are all notch-tough rated FCAW metals that have been replacing E70T-4 since the earthquake. E7018 is a Shielded Metal Arc Weld (SMAW) that is not common in modern building construction but is included for reference to give a historical perspective and because it is known to have very high toughness. Data designated EWI # are from tests by the Edison Welding Institute of tension coupons extracted from either beam-flange weldments fabricated in conjunction with SAC connection tests or from specific weld metal studies. The data points designated “Lincoln” are from tests reported by the Lincoln Electric Company.

The average yield strengths of all welds are in range of about \( F_y = 70 \) to \( 75 \) ksi, with the one notable exception being the strength of \( F_y = 55 \) ksi for E70T-4. Ultimate strengths of all welds are in the range of \( F_u = 80 \) to \( 90 \) ksi, thus meeting the minimum requirements for E70 materials. Compared to base metals, there is much higher variability in the weld metal yield strengths, presumably due to variations in welding procedures affecting heat input, cooling rates, etc. Given the large variability and small sample sizes, it is not clear whether the lower average strength for E70T-4 is a general trend or a function of the small sample size. Nevertheless, this outlying data indicates the large variability that can exist in weld properties. Differences of this magnitude (\( F_y = 55 \) versus \( 70 \) ksi) can be significant in cases where connection behavior is sensitive to weld yield strength – or more specifically the relative yield strength of the weld-to-base metal. This highlights the need for accurate measurement and reporting of weld properties in tests and analyses.

2.4 CVN Toughness of Base Metal

In addition to the tension coupon tests to determine stress-strain properties, Jaquess et al. (1999) conducted Charpy-V Notch (CVN) tests to statistically characterize the fracture toughness of A572 Gr. 50 steel in rolled shapes. They reported the results in terms of average coefficients to define the temperature-transition curve shown in Fig. 2.7 and given by the following equation:

\[
CVN = \beta_1 + \beta_2 \tanh[\beta_3 (T - \beta_4)]
\]

(2.1)

where CVN is in ft-lbs. and T is in degrees F. The reported mean coefficients for the base metal material are \( \beta_1 = 99 \), \( \beta_2 = 97 \), \( \beta_3 = 0.083 \), and \( \beta_4 = 29 \).

Referring to Fig. 2.7, the brittle-to-ductile transition region begins around \( 25^\circ F \) and fully ductile upper-shelf behavior begins around \( 50^\circ F \). The average upper shelf toughness is quite high, CVN=196 ft-lb (265 Joules) with a range of CVN = 67 to 264 ft-lb (90 to 356 Joules) and standard deviation of 59 ft-lb (80 Joules). These statistics are combined results for samples taken from the webs, flanges, and core regions of rolled shapes, and even the lowest core region values satisfy the AISC Seismic Provisions (1997) minimum specified core toughness of CVN = 20 ft-lbs. (27 Joules) at \( 70^\circ F \) for material thicker than 1-1/2 inches. Assuming that this survey is
indicative of steel shapes used to fabricate connections tested in the SAC program, then these
data suggest that the tested connections were fabricated from steels with exceptional toughness
that well exceed the minimum specified toughness. The extent to which the connection tests are
indicative of the performance of steels with the minimum specified toughness is not clear.

Shown in Figure 2.8 is a comparison of the average CVN curve by Jaquess et al. (1999) and data
reported in an earlier mill survey by AISC (1995). All AISC data are from core regions of hot-
rolled A572 Gr. 50 shapes. While these data tend to bracket the average curve, they exhibit
significantly more scatter than the data in Jaquess et al.’s survey. For example, at room
temperature, values as low as CVN = 15 ft-lbs (20 Joules) are reported in the AISC survey
whereas the lowest reported by Jaquess et al. is CVN = 67 ft-lbs (90 Joules). Reasons for the
differences are not clear, except that perhaps the AISC survey included samples from a larger set
of producers, or perhaps there have been improvements in mill practices over the past five years
that have increased the consistency with which producers can deliver high toughness steels.

2.5 CVN Toughness of Weld Metal

CVN temperature-transition curves for weld metals tested by Johnson (1999) are shown in Fig.
2.9. The tested weld samples were extracted from many of the same sources as the tensile
coupon data reported earlier in Section 2.3. The curves shown in Fig. 2.9 (based on Eq. 2.1) were
fit to Johnson’s data for the purpose of calculating a temperature shift to account for loading rate
effects.

Data for the E70T-4 weld metal in Fig. 2.10 lie in the lower-shelf to lower-transition regions
with CVN roughly equal to 10 ft-lb. room temperature (70°F). Previous studies report even
lower toughness values of CVN = 5 to 10 ft-lb for E70T-4 material (e.g., Kaufmann et al., 1995,
1996). For all other weld metals, which have a specified minimum notch toughness requirement,
the fracture behavior at 70°F is in the upper-transition to upper-shelf regions. However, there is
a wide variability in toughness of these welds at room temperature (70°F), ranging from a low of
CVN = 37 ft-lb for E70T-6 up to values of 72, 80, and 137 ft-lb for E70TG-K2, E71T-8, and
E7018, respectively.

Given that weld toughness values are in the transition region at room temperature, they should be
adjusted for differences in loading rates between the impact rates applied in CVN tests and the
quasi-static rates that connections are subjected to in lab tests (or intermediate rates of actual
earthquakes). Further information on how loading rates can affect fracture behavior is provided
in the appendix. To adjust for loading rate effects, we shifted the CVN temperature-transition
curves from impact to static loading rates following guidelines by Barsom and Rolfe (1987). For
data in the transition region, this involves shifting the curve to the left by \( \Delta T = -120\, ^{\circ}\text{F} \) (see Fig.
2.10) resulting in an apparent increase in toughness where transition region behavior governs.
In the upper-shelf region, the CVN values are reduced by 10\% for static versus impact loading,
reflecting the fact that upper-shelf toughness is related to yield strength which reduces with
decreasing loading rate. Included with the resulting dynamic (impact) and static temperature-
transition in Fig. 2.10 are points (shown by shaded circles) corresponding to the static toughness
at room temperature. By accounting for this shift, the toughness of E70T-4, as given by the data
in Fig. 2.10a, increases from a dynamic (impact) value of \( CVN_d = 10 \text{ ft-lb} \) to a static equivalent of \( CVN_s = 40 \text{ ft-lb} \). It should be noted, however, that the increase from 10 to 40 ft-lbs strongly depends on the shape of the transition curve in Fig. 2.10a, and \( CVN_s = 40 \text{ ft-lb} \) is probably an optimistic value for E70T-4 weld metal. For example, data by Kaufmann and Fisher (1996) reproduced in Fig. 5 of the appendix would result in a toughness of roughly \( CVN_s < 20 \text{ ft-lb} \) for E70T-4 at room temperature. It is further interesting to note that the calculated \( CVN_s \) for the E70T-6 weld metal (\( CVN_s = 50 \text{ ft-lb} \) from Table 2.3) is not markedly different than that for the E70T-4 weld metal. However, given that E70T-6 is operating closer to upper shelf behavior (Fig. 2.10b), there are likely to be differences in behavior that may show up more clearly in more precise fracture tests (e.g., slow or intermediate rate CTOD tests). Changes in CVN between dynamic and static are slight for E7018 and E70TG-K1, and lacking sufficient data to define a clear upper shelf value for E71T-8, its static toughness is estimated at about \( CVN = 110 \text{ ft-lbs} \) based on the slope of the transition curve.

### 2.6 Fracture Toughness Indices

While CVN data provide a qualitative check on fracture toughness, the CVN absorbed energies do not in themselves directly relate to fracture indices calculated by analysis. Therefore, for use in fracture mechanics analyses it is necessary to correlate the CVN data to more rigorous fracture toughness indices. Three mode I fracture indices considered in our analyses are:

- \( K_{lc} \) - Elastic stress intensity factor, a measure of the critical value of stress singularity at the crack tip under predominately elastic conditions.

- \( \text{CTOD}_c \) - Inelastic Crack Tip Opening Displacement, whose critical value is a measure of the maximum blunting of the crack tip at the onset of crack initiation.

- \( J_{lc} \) - A fracture index based on the strain energy release associated with crack extension that is applicable for both elastic and inelastic analyses. Under the assumption of proportional loading behavior, the \( J \)-integral is equal to the linear or nonlinear elastic energy release rate, \( G \). The \( J \)-integral provides a convenient numerical technique to determine \( K_{lc} \) and \( \text{CTOD}_c \) from the finite element analyses.

The elastic fracture toughness index \( K_{lc} \) can be empirically converted from the static CVN curve (following Barsom and Rolfe 1987) by one of the following two equations, depending on whether the material toughness falls within the transition or upper-shelf region:

\[
\frac{K_{lc}^2}{E} = 5CVN \quad \text{for the transition region} \tag{2.2}
\]

with CVN in ft-lbs, \( K_{lc} \) in \( \text{psi}\sqrt{\text{in}} \), and \( E \) in \( \text{psi} \), and

\[
\left( \frac{K_{lc}}{\sigma_y} \right)^2 = 5 \left( \frac{CVN}{\sigma_y} - 0.05 \right) \quad \text{for the upper-shelf region} \tag{2.3}
\]
with CVN in ft-lbs, $K_{lc}$ in ksi√in, and $\sigma_t$ in ksi. Using $K_{lc}$ from either of these equations, CTOD$_C$ can then be determined from the following,

$$K_{lc} = \sqrt{m\sigma_f CTOD}$$  \hspace{1cm} (2.4)

where $m=1.6$ for plane strain conditions, and the flow stress $\sigma_f = (F_y + F_u)/2$ where $F_y$ and $F_u$ are the tension yield and ultimate stress of the material, respectively. Equation 2.4 is dimensionally consistent, so that all terms should be input with consistent units. Equation 2.4 can further be combined with Eq. 2.2 or 2.3 (as applicable) to demonstrate a linear relationship between CTOD and CVN.

Finally, for plane strain conditions, $K_I$ and CTOD are related to $J_I$ by the following equations:

$$K_I = \sqrt{\frac{E J_I}{(1-v^2)}} \quad \text{(for elastic analyses)}$$  \hspace{1cm} (2.5)

$$CTOD = \frac{J_I}{m\sigma_f} \quad \text{(for inelastic analyses)}$$  \hspace{1cm} (2.6)

where $v = 0.3$ (Poisson’s ratio for steel) and the other terms are as defined above.

Based on Eqs. 2.2 to 2.6 and the material toughness and stress-strain data reported previously, critical material toughness values for the base and weld metals are summarized in Table 2.3. Note that the conversion from CVN$_{static}$ to $K_{lc}$ depends on whether the material is behaving in the transition or upper shelf region. Based on the CVN$_{static}$ versus temperature curves shown in Fig. 2.10, all the weld metal except for E70T-4 are assumed to be operating in the upper shelf region at room temperature where the CVN$_{static}$ to $K_{lc}$ conversion is based on Eq. 2.3. The fracture toughness values in Table 2.3 are generally consistent with values expected for these materials, although the E70T-4 values are higher than has been reported by others. For example, Kaufmann et al. (1995) calculated $K_{lc} = 40$ to 60 ksi√in for E70T-4 with corresponding values of CTOD$_C = 0.0005$ to 0.0012 inch (see Chi et al. 1997).

The data shown in Table 2.3 are cited later in this report as the basis for interpreting the results and implications of the fracture analyses. Aside from uncertainties introduced through the empirical basis for deriving $K_{lc}$ and CTOD$_C$ from CVN, another assumption inherent in the $K_{lc}$ and CTOD$_C$ criteria is that the crack front is highly constrained and the stress/strain field satisfied the conditions for small scale yielding. These conditions are not strictly met in the connections, particularly since the surface cracks are shallow and sometimes engulfed by yielding. According to Sorem et al. (1991), in such cases the effective material toughness can be twice (or more) that of the values reported in Table 2.3 for highly constrained conditions. This is a point to be kept in mind when comparing the calculated toughness demands to the critical values in Table 2.3.
2.7 Finite Element Modeling Techniques and Parameters

Fracture toughness demands imposed in the welded connections are determined using two- and three-dimensional finite element models of the connection subassembly, such as shown in Fig. 2.11. The refined mesh around the crack tip region was arrived at through several mesh convergence studies based on the accuracy of evaluating $J$. The two-dimensional model, shown in Fig. 2.11(a), consists of 8-noded quadrilateral and 6-noded triangular elements. To simulate the high constraint conditions at the crack tip, the weld and surrounding base metal are modeled with plane strain elements, but elsewhere the two-dimensional elements are plane stress. The three-dimensional model in Fig. 2.11(b) consists of 20-node brick elements with 8-point integration. For reasons of modeling practicality, in the three-dimensional analyses and some of the inelastic two-dimensional analyses, the initial flaw is located a small distance (roughly 0.1 inch) away from the column face. This small shift in location has been shown by comparative two-dimensional analyses to have a negligible influence on the results. All analyses presented were run using ABAQUS Version 5.8 (Hibbitt et al.1998) on a DEC Alphastation (266 MHz, 128 Mbyte RAM, UNIX OSF 1).

From the elastic analyses, fracture toughness demand, $K_t$, is calculated through a J-integral approach (using Eq. 2.5) with a rosette of quarter-point triangular (2D) or collapsed brick (3D) finite elements which capture stress singularities around the crack tip. Toughness demand from the inelastic analyses is defined by Crack Tip Opening Displacement (CTOD) which requires the collapsed elements to expand and thereby simulate crack tip blunting, as shown in Fig. 2.11b. CTOD was determined in two ways, either explicitly by measuring the deformed crack tip finite element mesh or converting from J-integral energy, using Eq. 2.6. The J-integrals are evaluated using the built-in feature of ABAQUS based on the domain integral method, and as a check on the accuracy at least five integration contours are checked for each calculation.

Elastic steel properties were assumed as $E = 29,000$ ksi (200,000 MPa), and $v = 0.3$. Inelastic analyses were run using $J_2$ incremental plasticity theory with a von Mises yield criterion and the base and weld metal stress-strain data summarized in the previous sections. For the large deformation analyses engineering stress-strain data is converted into true stress-log strain space by the following equations:

$$\sigma_{\text{true}} = \sigma_{\text{eng}} \left( 1 + \varepsilon_{\text{eng}} \right) \quad (2.7)$$

$$\varepsilon_{\ln} = \ln \left( 1 + \varepsilon_{\text{eng}} \right) \quad (2.8)$$

Shown in Fig. 2.12 are curves of generic stress-strain properties based on the average yield and ultimate strengths of $F_y = 55$ ksi and $F_u = 72$ ksi for A572 Gr. 50 base metal and $F_y = 65$ ksi and $F_u = 86$ ksi for E70 weld metal. The assumed yield strength of the weld metal is taken as a lower estimate to the averages reported in Fig. 2.5 since previous analyses of weld matching effects by the authors (Chi et al. 1997) have shown a lower value gives a conservative measure of the toughness demand. The true-stress versus true-strain curves shown in Fig. 2.12 are input to the finite element analyses in a piecewise linear fashion through the isotropic hardening model in ABAQUS.
2.8 Micro-Mechanical SMCS Ductile Crack Initiation Criterion

Two shortcomings of the conventional fracture indices $K$, CTOD, and $J$ are that (1) they require prescription of an initial flaw or pre-existing crack in the structure and (2) they are based on the assumptions of high constraint and small-scale yielding in the crack tip region. The first point presents obvious limitations for evaluating conditions where there are high stresses/strains in the material but no apparent flaws. One example of this is in the evaluation of through-thickness failures that are thought to initiate inside the base material of the column flange. The second requirement of high constraint and small scale yielding raises concerns about applying conventional fracture models where there is significant yielding around the crack tip. Specifically, the application of conventional indices ($K$, CTOD, and $J$) to evaluate shallow surface cracks in connections with large inelastic deformations can lead to excessive conservatism since the apparent toughness under these conditions is larger than that measured in standard fracture test specimens.

As an alternative to the conventional fracture indices, we have included some analyses that utilize a micro-mechanical model called the Stress Modified Critical Strain (SMCS) criterion to examine ductile crack initiation in the connections. This technique simulates material behavior at a micro scale using a continuum model that is not subject to the limitations of conventional fracture indices. The SMCS criterion does, however, require finite element models that are much more highly refined than those needed for conventional fracture analyses. With mesh length scales that are about an order of magnitude smaller than for conventional fracture models, the modeling and computational demands for SMCS models are about two orders of magnitude larger for two-dimensional analyses and three orders larger for three-dimensional analyses. Additionally, micro-mechanical models are still under development by the research community, and hence procedures for implementing and interpreting the models are not as well established as for conventional fracture indices. Nevertheless, these models provide a viable alternative to evaluate ductile (large scale yielding) fracture conditions in steel structures. The SMCS micro-mechanical model adopted for this study follows a similar application of the model to evaluate weld fractures by Panontin and Sheppard (1995). Texts by Anderson (1995) and Saxena (1998) provide an introduction to recent developments of micro-mechanical models.

Ductile Fracture Initiation Behavior: As shown in Fig. 2.13, at the micro-level ductile fracture in steel occurs through three stages - void nucleation, growth, and coalescence (Anderson 1995). Void nucleation results from the presence of high stresses and strains around inclusions or precipitates in the steel microstructure. Nucleation is followed by void growth under increasing stresses and strains, and coalescence is the stage at which material between adjacent voids necks down and eventually tears to initiate a macroscopic ductile fracture. What is important to recognize from the behavior shown in Fig. 2.13 is the difference between the conventional structural engineering view of behavior at the macroscopic level, i.e., the level where conventional elastic-plastic continuum models apply, versus the true microscopic behavior. For example, whereas at the macroscopic level standard plasticity rules for metals imply that there is no plasticity under pure hydrostatic tension stresses, at the local level the inclusions and irregular grain boundaries create nonuniform stresses and strains that cause localized yielding. In essence, micro-mechanical models provide the means to relate localized phenomena, such as shown in Fig. 2.13, to continuum macroscopic models that are convenient to implement in standard finite
element methods. Here the term “continuum” refers to models that do not explicitly represent
the features of the microstructure shown in Fig. 2.13, but instead treat the material as
homogenous and capture the micro-behavior through semi-empirical stress-strain models and
indices.

**SMCS Criterion:** Underlying the Stress Modified Critical Strain (SMCS) criterion are two
fundamental parameters to characterize conditions for ductile crack initiation, i.e., macro-scale
void coalescence. One parameter quantifies the critical plastic strain (for crack initiation) as a
function of the applied stresses, and the second is a length scale that defines the critical volume
of material over which the critical plastic strain must exist to permit void coalescence.

Based on concepts that can be traced to early work by McClintock (1968), Rice and Tracey
(1969), Hancock and Mackenzie (1976), and others, the critical value of plastic strain is
described as a function of triaxial continuum stresses through the following equation:

\[
\varepsilon_p \mid_{\text{critical}} = \alpha \exp \left( -1.5 \frac{\sigma_m}{\sigma_e} \right)
\]  

(2.9)

where

\[
\varepsilon_p = \text{the von Mises equivalent plastic strain}
\]

\[
= \left( \frac{2}{3} \varepsilon_{ij} \varepsilon_{ij} \right)^{1/2}
\]

\[ \alpha \] is a material constant
\[ \sigma_m = \text{hydrostatic stress} = \sigma_{ii}/3 \]
\[ \sigma_e = \text{effective von Mises stress} = \left( \frac{3}{2} S_{ij} S_{ij} \right)^{1/2} \text{ and } S_{ij} \text{ is deviatoric stress} \]

One can think of Eq. 2.9 as a limiting criterion for plastic strain for material under triaxial
stresses in the same vein that the von Mises yield criterion describes the critical value of triaxial
stresses that cause yielding. The material constant \( \alpha \) can be obtained through notched bar tensile
tests that provide a means for testing the limiting strain in the material under different ratios of
stress triaxiality, \( \sigma_m/\sigma_e \). From Eq. 2.9, Panontin and Sheppard (1995) formalized the notion of
the Strain Modified Critical Strain criterion (SMCS) index as the difference in the imposed
plastic strain \( \varepsilon_p \) and the critical plastic strain \( \varepsilon_{p,cr} \) according to the following equation:

\[
SMCS = \varepsilon_p - \alpha \exp \left( -1.5 \frac{\sigma_m}{\sigma_e} \right)
\]  

(2.10)

Using Eq. 2.10, ductile fracture initiation is assume to occur when SMCS ≥ 0 over a critical
length (volume) of material, termed the characteristic length, \( l^* \). Referring back to Fig. 2.13, the
characteristic length is dependent on the typical distance between voids, which in turn depends on the microstructure of the material. As discussed by Panontin and Sheppard (1995), the characteristic length is roughly equal to the distance between ductile fracture dimples (or craters) that characterize appearance of ductile fracture surfaces such as one sees in tension coupons, three-point bend specimens, and other cases governed by Mode I ductile fracture.

For mild steel, Panontin and Sheppard (1995) suggest that the characteristic length is roughly equal to 2 to 3 times the typical grain size, and in a study of A516 Gr. 70 plate material they determined the SMCS parameters to be $\alpha = 2$ and $l^* = 0.003$ inch. In tests on notched bar tension coupons and three-point bend specimens extracted from the longitudinal direction of a A572 Gr. 50 column, Chi (1999) measured material parameters $\alpha = 3$ and $l^* = 0.0035$ to 0.004 in. Thus, for the purposes of this study, the values determined by Chi ($\alpha = 3$ and $l^* = 0.0035$ in.) are considered as representative of ductile fracture properties for the base metal in the rolling direction. However, these properties do not necessarily represent those in the direction transverse to rolling or in welding Heat Affected Zones (HAZ). Lacking test data to rigorously quantify conditions in the through thickness direction or in weld HAZ, we have assumed values of $\alpha = 1$ and $l^* = 0.0035$ in. as conservative lower bound properties in our finite element analyses.
3. Elastic Behavior and $K_I$ Toughness Demands

3.1 Overview

This chapter examines elastic fracture toughness demands for weld flaws in the standard (i.e., unreinforced or non-RBS) connection detail modeled after that shown in Fig. 1.1. The main objective is to describe the stress intensity factor, $K_I$, as a function of the connection geometry and the nominal applied bending stress. To do this, we ran numerous two- and three-dimensional finite element analyses. Following a general review of the influence of weld backing bars, flaw location, and other effects, a systematic parametric study is presented to address surface (exterior) and interior cracks located at the column flange-to-weld interface. Simplified prediction equations are then developed and calibrated to estimate stress intensity factors in terms of the applied stress and connection geometry. Beyond providing quantitative data on toughness demands, the analyses provide insight on the influence that various design and detailing parameters have on fracture.

3.2 General Behavior and Trends

Based on the analyses previously reported by the authors (Chi et al. 1997, 2000), a general overview of elastic fracture in the beam-column connections is presented in this section. These preliminary analyses are based on a single connection geometry with a W36 x 150 beam and W14 x 257 column. Section 3.3 follows with systematic parametric study to evaluate other beam and column sizes and changes in the weld configuration.

3.2.1 Basic Characteristics of Stress Intensity at Weld Root Defect

Shown in Fig. 3.1 are contour and line plots of longitudinal beam flange stresses in the vicinity of the weld access hole, under an applied moment corresponding to a nominal beam bending stress at the column face of $\sigma_n = M/S = 25$ ksi. Normalized by the applied nominal bending stress ($\sigma_n = 25$ ksi), the line plots compare stresses predicted by beam theory with the finite element results at the three locations (A, B, and C) shown in the figure. The significant nonlinearity of stresses is an indication of subtleties in fracture behaviors that are not apparent in traditional “engineering models” of the connections. For example, from the stress distribution at location C, it is clear that initial flaws located on the lower side (extreme fiber) of the beam flange would be much more susceptible to fracture than flaws on the inner (upper) surface of the flange. Since the bottom backing bar is located on the extreme fiber, and whereas the top flange backing bar is on the inside surface of the flange, this large stress gradient helps explain the prevalence of bottom flange versus top flange fractures observed in the post-earthquake building inspections and connection tests.

Stress intensity factor results shown in Fig. 3.2 provide a quantitative fracture mechanics interpretation of the high stresses at the weld root defect. Here, $K_I$ is normalized by the nominal
bending stress and plotted versus the weld root length, \( a_o \), which is the crack length in excess of the backing thickness \( W \). The finite value of \( K_I/\sigma_n = 1.2 \) for \( a_o = 0 \) arises due to the inherent flaw created by the tip of the backing bar gap. Analyses for multiple backing bar thicknesses \((w = 0 \text{ to } 0.5 \text{ inch})\) show that the stress intensity factor does not vary with the backing bar gap size, which follows from the fact that the backing bar is an unstressed attachment. This behavior enables simplification of subsequent analyses in which the backing bar is omitted from the finite element models where the initial weld root defect is assumed to be larger than \( a_o > 0.1 \text{ inch} \).

Referring to Fig. 3.2, the presence of any size backing bar or small weld root defect would cause a stress intensity factor equal to \( K_I \approx 1.2 \sigma_n \). For larger defects \( K_I \) increases nearly linearly with increasing \( a_o \). The data in Fig. 3.2 would imply, for example, that for welds of low toughness E70T-4 with \( K_{lc} = 60 \text{ to } 75 \text{ ksi} \sqrt{\text{in}} \) (see Table 2.3) and small flaw \((K_I = 1.2 \sigma_n)\), fracture would occur at a nominal bending stress of \( \sigma_n = 50 \text{ to } 63 \text{ ksi} \). On the other hand, for higher toughness weld materials, \( K_{lc} = 130 \text{ to } 190 \text{ ksi} \sqrt{\text{in}} \) (see Table 2.3), the analyses would predict fracture at stresses of \( \sigma_n = 108 \text{ to } 160 \text{ ksi} \). Of course, where the predicted fracture stresses exceed the yield stress, inelastic behavior and analyses should be considered. Also note that the stress ranges mentioned here are only for illustrative purposes since they do not account for three-dimensional and residual stress effects.

Questions have been raised as to whether the invariance to backing bar thickness, \( w \), is related to the length over which the backing bar is fused to the weld, since increased fusion length allows more stress transfer through the backing bar. To investigate this, results of analyses for cases with fusion lengths of \( L = 0.5 \text{ to } 1 \text{ inch} \) are shown in Fig. 3.3. These data show that the longer fusion length does increase the stress intensity factor but to a fairly modest degree. For the smallest root defect where the differences are greatest, doubling the fusion length from \( L = 0.5 \text{ to } 1 \text{ inch} \) only increases \( K_I \) by about 10% and tripling the length from 0.5 to 1.5 inch increases it by about 20%. These differences decrease as the flaw increases to the point at which for flaw sizes larger than \( a_o = 0.3 \text{ inch} \) there is practically no difference. Overall, these results indicate that, compared to other effects, the backing bar geometry (thickness and fusion length) does not have a significant effect on the toughness demands.

### 3.2.2 Effect of Flaw Location

While the most likely locations of flaws in the pre-Northridge connections are at the weld-column interface, it is useful to consider how \( K_I \) varies for flaws at other locations that may be critical for improved details where the backing bars are removed and root pass cracks repaired. As shown in Fig. 3.4, analyses for flaws located at three different points on the bottom of the flange indicate that \( K_I \) does not change much for flaw locations in close proximity (up to 0.5 inches away) from the column face. On the other hand, the toughness demand for a flaw on the top of the beam flange is about 75% less than that for the bottom surface flaw. This large difference follows from the stress distributions shown in Fig. 3.1 and provides further evidence to explain the smaller likelihood of top flange fractures where the backing bar flaws are on the inside face.

While removal of the weld backing and repair of root pass flaws have clear advantages, a less costly modification that has been proposed is to leave the backing in place and close the backing
bar gap with a reinforcing fillet weld below the backing bar. The thought is that this will reduce the stress intensity factor at the root flaw crack tip by changing the initial defect from an edge to interior crack. Referring to Fig. 3.5, two different cases with fillet weld leg dimensions of 1/8 and 3/8 inch are investigated, the former being a lower bound weld size and the latter a more likely size, roughly equivalent to the backing bar thickness. Note that since closure of the edge crack introduces a new crack tip at the fillet weld root, $K_I$ data for two crack tips are shown in Fig. 3.5 for the upper (U) and lower (L) crack tip. The results show that the 1/8-inch fillet weld reduces the elastic $K_I$ to about one-half its original magnitude and the reduction is slightly larger for the 3/8-inch fillet weld. Two points not reflected in these analyses which can affect the results are: (1) as noted by Chi et al. (1997, 2000), when inelastic behavior is considered, the 1/8 inch fillet weld yields and loses its effectiveness at higher stresses, whereas the 3/8 inch fillet continues to provide a significant reduction in toughness demand, and (2) if the weld root fusion length is significantly less than $L = 0.5$ inch (as may occur due to a tight fit up), the bottom fillet weld will not provide much benefit since there is no stress path from the beam flange/weld into the backing bar.

### 3.2.3 Two- versus Three-Dimensional Behavior

Two-dimensional analyses cannot capture two key aspects of behavior. One aspect involves out-of-plane behavior, i.e., the variation in stresses and strains across the beam flange width due to shear lag and other effects. The second aspect is related to the shape of the initial flaw itself. In a two-dimensional analysis the flaw extends the full width of the beam flange, reducing to a line in the model and the crack tip to a point. In actuality the crack is a surface and the crack front is a line. Referring back to Fig. 2.11, the three-dimensional analyses we have run with a flange through crack capture the first of these two effects, i.e., the three-dimensional distribution of stresses and strains. For connections with backing bars left in place, this sort of model with a through crack is a reasonably good representation of the weld root flaw created by the combined backing bar gap and weld root defect. Where the backing bars are removed and the crack width is smaller, the through crack assumption is less realistic. However, to properly model a three-dimensional elliptical crack geometry would require a fairly complex and time consuming meshing operation which is beyond the resources available for this project. For this reason, and because the through-crack assumption is generally conservative, the three-dimensional analyses are all based on a through-crack model.

Compared in Figure 3.6 are the normalized $K_I$ results from a two- and three-dimensional analyses for a crack length of $a_o = 0.1$ inch. The horizontal axis is the distance across the beam flange width, where the distance is measured from the flange tip. In this case, the two-dimensional results tend to provide a reasonable average of the three-dimensional results, where the maximum three-dimensional stress intensity factor under the beam web, $K_I/\sigma_{nt} = 2.0$ is about 40% larger than the two-dimensional results. The large difference between the two- and three-dimensional analyses indicates that two-dimensional analyses alone do not accurately predict the true toughness demand. However, when used in conjunction with select three-dimension analyses, two-dimensional analyses are useful for conducting parametric analyses and investigating relative effects between different connection details.
3.3 Parametric Analyses for $K_I$ at Column-to-Weld Interface

In this section, results from the preceding analyses of one connection geometry (a W36 x 150 beam connected to a W14 x 257 column) are extended through parametric analyses to investigate the variations in $K_I$ for alternative beam and column sizes. These parametric analyses are structured to provide the basis for developing a simplified equation to calculate $K_I$ for two flaw types – an edge crack and an interior crack at the column-to-weld interface. Based on the previous results, these two flaws are considered to provide representative elastic fracture toughness requirements for welded beam-column joints. Aside from the geometric parameters, the analyses consider the influence of welding-induced residual stresses on fracture toughness demands.

3.3.1 Calibration Approach

Following from Fig. 3.2, it is anticipated that a general linear relationship can be developed to relate the normalized stress intensity factor, $K_I/\sigma_n$, to flaw length, $a$. This linear relationship can be written as:

$$K_I/\sigma_n = C_1 + C_2 \times a$$  \hspace{1cm} (3.1)

in which $C_1$ and $C_2$ are functions of connection geometries, and $\sigma_n$ is the nominal applied bending stress based on beam theory, $\sigma_n = M/S$. Referring back to Fig. 3.2, the initial crack length, $a$, is defined as the distance between the crack tip and the fusion line between the beam flange and the backing bar. When $a$ is close to zero, the crack tip at the top of the backing bar is a stress riser that leads to a finite (non-zero) stress intensity factor, represented by the coefficient $C_1$ in Eq. 3.1. When backing bars are not present, these analyses provide a conservative measure of elastic toughness demand for small weld flaws that represent a minimum flaw criterion for weld acceptance.

The primary parameters considered in the elastic calibration exercise are the geometric dimensions of the beam and column shown in Fig. 3.7a, including $t_{wb}$, $b_{fb}$, $d_b$, $t_{fb}$, $t_{wc}$, $b_{fc}$, $d_c$, and $t_{fc}$. Beginning with the standard connection geometry previously analyzed (W14 x 257 column and W36 x 150 beam), the beam and column dimensions are varied over the range in Table 3.1. As shown in Figure 3.7b, we investigated two initial crack geometries representing an edge and interior flaw at the column-to-weld interface. Meshes of the weld region for the two- and three-dimensional models are shown in Fig. 3.8.

The investigation begins for the edge crack using two-dimensional analyses where the basic strategy is as follows. First, the influence of the beam geometry is investigated by stiffening the column, as if the beam is framing into a rigid support, and separately varying the beam dimensions. Next, the influence of the column geometry is studied by holding the beam size constant and varying the column dimensions. Finally, variations in dimensions of the two members are considered simultaneously. Variations in the beam geometry are normalized through the relative elastic section moduli of the beam web to flange, and variations in the column properties are normalized by the ratio between the stiffness of the column joint panel and
beam. Following a systematic parametric study of these effects using two-dimensional analyses, a selected set of three-dimensional analyses are run to further investigate specific parameters.

### 3.3.2 Parametric Study using 2-D Analyses

**Calibration of Beam Geometry Effects:** Under the assumption of a rigid column and edge crack lengths of \( a_o = 0.1 \) and 0.2 inch, variations of normalized stress intensity factors, \( K/\sigma_n \), as a function of beam dimensions are shown in Fig. 3.9. The following observations and implications can be drawn from this data:

- Except for beam flange thickness, \( t_{fb} \), \( K/\sigma_n \) varies nearly linearly with the beam dimensions, \( d_b, b_{fb}, \) and \( t_{wb} \). Further, the shallow slopes of these curves suggest that \( K/\sigma_n \) is not very sensitive to the beam geometry (exclusive of the flange thickness).
- The curves for \( a_o = 0.1 \) and 0.2 inch are parallel to each other, thus indicating that the linear variation in \( K/\sigma_n \) with flaw length generally holds for different connection geometries.
- The nonlinear variation in \( K/\sigma_n \) with flange thickness is limited to the region for the \( a_o = 0.2 \) inch flaw where the ratio \( a_o/t_{fb} > 0.2 \), suggesting that the nonlinearity is the result of edge distance effects. For \( a_o/t_{fb} < 0.2 \), the variation is fairly linear and gradual.
- Note that overall the range of \( K/\sigma_n = 0.4 \) to \( 0.6\sqrt{\text{in}} \) in Fig. 3.9 is lower than the range of \( K/\sigma_n = 1.4 \) to \( 1.6\sqrt{\text{in}} \) observed previously in Fig. 3.2 to 3.4. The large difference in \( K/\sigma_n \) (on the order of \( K/\sigma_n = 1\sqrt{\text{in}} \)) is due to the fact that in the analyses for Fig. 3.9 the column is assumed to be infinitely stiff, whereas in the previous analyses the column stiffness corresponds to the W14 x 257.

Data from Fig. 3.9 and other analyses combining beam geometric effects are replotted in Fig. 3.10a as a function of the ratio between beam web and beam flange elastic moduli, \( S_w/S_F \). With the exception of the few highlighted points related to the \( t_{fb} \) (edge effects) the section modulus ratio provides a reasonably good index to determine the change in \( K/\sigma \). To account for the edge effect, a second parameter \((A_{flange}/A_{ref})^2\) is introduced in Fig. 3.10b which further improves the linear correlation.

Based on the data and observations of Figs. 3.9 and 3.10, the linear relationship proposed by Eq. 3.1 is re-written as:

\[
\frac{K_I}{\sigma_n} = C_{1}^{*} \left[ \left( \frac{S_{web}}{S_{flange}} \right) \left( \frac{A_{flange}}{A_{ref}} \right) \right]^n + C_{1}^{**} + C_{2} \times a \quad \text{(3.2)}
\]

where \( A_{ref} = 11.3 \text{ in}^2 \) is the flange area of the standard beam of W36 x 150, \( C_{1}^{*} \) and \( C_{1}^{**} \) are coefficients reserved to account for column effects and the initial finite stress intensity, and \( C_2 \) remains as the coefficient on \( a \). The exponent \( n \) is introduced to account for potential nonlinearities introduced when the joint panel/column effects are included in the analyses.
**Calibration of Joint Panel/Column Geometry Effects:** To isolate the influence of the column geometry and joint panel stiffness, the beam size of W36 x 150 is held constant while the column flange thickness, $t_{fc}$, depth, $d_c$, and web thickness, $t_{wc}$, are varied. The column flange width, $b_{fc}$, is not varied in the two-dimensional analyses but is considered later in the three-dimensional analyses. As shown in Fig. 3.11, changes in the column dimensions lead to larger variations in $K/\sigma_n$ compared to those previously observed for the beam dimensions. The general range of values in Fig. 3.11, $K/\sigma_n = 1.2$ to $1.6\sqrt{\text{in}}$, are similar to those in Fig. 3.2 to 3.4.

Data from Fig. 3.11 is replotted in Fig. 3.12a as a function of the ratio between the beam and column stiffness, $K_{beam}$ and $K_{joint}$, respectively, which are defined as follows:

$$K_{beam} = \frac{4EI_b}{1.5d_b}$$ (3.3)

and

$$K_{joint} = 0.8t_{wc}d_bG + \frac{2Eb_{fc}t_{fc}^3}{d_b}$$ (3.4)

$K_{beam}$ is the elastic flexural stiffness for a length of beam equal to $1.5d_b$ (representative of a plastic hinge length), and $K_{joint}$ is based on an elastic stiffness model for the joint panel that was first introduced by Krawinkler (1982) with refinements by Chi (1999) to calibrate it to finite element data by El-Tawil (1998).

In Fig. 3.12a, slopes of the lines relating $K/\sigma_n$ to $K_{beam}/K_{joint}$ due to variations in $t_{wc}$ and $d_c$ are similar, but the index $K_{beam}/K_{joint}$ does not fully capture the column flange thickness effect. After considering several alternatives, the normalization factor shown in Fig. 3.12b,

$$\left(1 + \frac{K_{beam}}{K_{joint}}\right)^{0.8}/\sqrt{t_{fc}}$$, is proposed as a reasonable index to capture variations in the column properties. Further evidence of how well this index models the behavior is shown in Fig. 3.13 where results are compared for a multiple combinations of column/joint panel geometries and the two crack lengths with a constant beam size of W36 x 150. As shown, the variation of $K/\sigma$ with $\left(1 + \frac{K_{beam}}{K_{joint}}\right)^{0.8}/\sqrt{t_{fc}}$ is fairly linear, with a maximum error of 7% between the finite element results and linear regression.

Combining the column calibration factor $\left(1 + \frac{K_{beam}}{K_{joint}}\right)^{0.8}/\sqrt{t_{fc}}$ with the beam calibration factor, Eq. 3.2 evolves to:

$$\frac{K_I}{\sigma_n} = C_1 \left(1 + \frac{K_{beam}}{K_{joint}}\right)^{0.8} \left[\left(\frac{S_{web}}{S_{flange}}\left(\frac{A_{flange}}{A_{ref}}\right)\right)^n + C_1^{**} + C_2 \times a\right]$$ (3.5)
where the calibration coefficients are the same as described for Eq. 3.2.

**Full Calibration of Joint Panel and Beam Effects:** Thus far, the calibration has been done by separately varying the beam and column (joint panel) dimensions. To complete the calibration, we next ran multiple analyses combining variations in both beam and column geometries, and through linear regression with Eq. 3.5 obtained coefficients of $n = 0.8$, $C_1^* = 0.5$, $C_1^{**} = 2.5$, and $C_2 = 0.7$. A comparison of the resulting equation with finite element results for more than 400 combinations of column and beam geometries is shown in Fig. 3.14 where the maximal error is only 7%.

### 3.3.3 Calibration for Three-dimensional Effects

To extend Eq. 3.5 to account for three-dimensional behavior, we next ran a select set of three-dimensional analyses considering variations in the beam dimensions ($t_{wb}$, $b_{fb}$, $d_b$, $t_{fb}$), column dimensions ($t_{wc}$, $b_{fc}$, $d_c$, and $t_{fc}$), and the beam shear span. Representative examples of the results are shown in Figs. 3.15 and 3.16, where results are expressed in terms of the stress intensity ratio, $K_{3-D}/K_{2-D}$.

Shown in Fig. 3.15 are variations in $K_I$ across the beam flange width for models with different widths, $b_{fb}$. In the top figure the horizontal axis is the absolute distance across the flange width, whereas in the lower figure the distance is normalized by the flange width. Both figures indicate that the ratios of the maximum three- to two-dimensional stress intensities range between $K_{3-D}/K_{2-D} = 1.57$ to 1.68, however, the different trends in the variation of stress intensity across the beam flange are not intuitively obvious. Referring back to Fig. 3.6, in the first case that we studied (W14 x 257 column and W36 x 150 beam), the $K_{I,3D}$ was larger than $K_{I,2D}$ at the beam centerline and less at the flange tips, such that the average value for $K_{I,3D}$ was roughly equal to $K_{I,2D}$. Referring to the top plot in Fig. 3.15, this is not generally the case as the analyses show that for narrow flanges $K_{I,3D}$ is larger than $K_{I,2D}$ across the entire flange width. Moreover, the peak ratios of $K_{3-D}/K_{2-D}$ at the centerline do not appear related to the flange width, as the curves in the top plot of Fig. 3.15 tend to coincide. The lower plot of Fig. 3.15, where the flange width distances are normalized, does suggest that the shear lag effects reduce for the narrower beams and that the maximum $K_{3-D}/K_{2-D}$ approaches a constant value around 1.5 to 1.6.

The invariance of the $K_{3-D}/K_{2-D}$ ratio is further apparent in Figs. 3.16a through 3.16h where variations in the other beam and column dimensions are considered. Variations in shear span (Fig. 3.16h) are investigated to determine whether extreme variations would affect the results. The span variations of $S = 60$ to 250 inches correspond to shear span/beam depth ratios of $S/d_b = 1.6$ to 7.0. Larger values may occur in practice, but it is assumed that results for $S/d_b = 7$ are representative of cases with $S/d_b > 7$. Overall, the ratios of maximum values for the parameters investigated in Fig. 3.15 and 3.16 range between $K_{3-D}/K_{2-D}$ from 1.46 to 1.77, with an average close to 1.6.

### 3.3.4 Summary of Prediction Equation for Edge Crack

To summarize, the following equation is proposed to estimate toughness demands for an edge crack:
where

\[
\frac{K_I}{\sigma_n} \bigg|_{2D-MAX} = 1.6 \left( \frac{K_I}{\sigma_n} \right)_{2D}
\]  

(3.6)

and where \(K_{beam}\) and \(K_{joint}\) are determined per Eqs. 3.3 and 3.4, respectively. \(S\) is the elastic section modulus, calculated separately for the beam web and flange, \(A_{flange}\) is the area of the beam flange, \(A_{ref}\) is a reference flange area (\(A_{ref} = 11.3\) in\(^2\)), \(a\) is the initial crack length, and \(t_{fc}\) is the column flange thickness. Note that Eq. 3.6 is not dimensionless and \([K_I/\sigma_n]_{2D}\) carries units of \(\sqrt{in}\) and \(a\) and \(t_{fc}\) should be input in inch units.

### 3.3.5 Calibration of Predictive Equation for Interior Cracks

Study of interior cracks is based on the flaw location and weld geometry shown previously in Figs. 3.7b and 3.8b, where it is assumed that weld-backing bars have been removed and a small fillet weld installed. Three fillet sizes are assumed, 0, 3/16 and 3/8 inch, and the lower crack tip location is assumed to be held fixed at \(h = 0.1\) inch above the lower flange surface. Shown in Fig. 3.17 is an example of how \(K_I/\sigma_n\) varies with fillet size and crack length, for the basic W36 x 150 beam connected to a W14 x 257 column. Comparing Fig. 3.17 to the basic exterior crack in Fig. 3.2, toughness demands for any of the interior cracks are on the order of one-third to one-half those for exterior cracks. It is also obvious from Fig. 3.17 that the resulting stress intensity factors, \(K_I/\sigma_n\) are smaller for the larger fillet reinforcement.

In a similar manner as the edge crack calibration, we conducted over 200 two-dimensional elastic parametric analyses to calibrate an equation to estimate \(K_I/\sigma_n\) for the interior cracks. Assuming that the fillet reinforcement is likely to be on the order of 3/8 inch, the calibration is focused on this case with initial crack lengths of \(a_o = 0.1\) and 0.2 inch. Trends observed in the edge crack calibration generally held for the interior crack as well, with the one key exception that the interior crack \(K_I/\sigma_n\) was not as sensitive to the column flange thickness as the edge crack. As a result, the calibration equation for the interior crack does not include the term, \(1/\sqrt{t_{fc}}\), that appears in Eq. 3.7.

The following equation to estimate \(K_I/\sigma_n\) for interior cracks resulted from this study:

\[
\frac{K_I}{\sigma_n} \bigg|_{2D-interior} = 0.1 + 0.1 \left[ \left( 1 + \frac{K_{Beam}}{K_{Joint}} \right) \times \frac{S_{Beam \ Web}}{S_{Beam \ Flange}} \times \left( \frac{A_{flange}}{A_{ref}} \right)^2 \right] + 1.6a
\]  

(3.8)
where the terms are defined as for Eq. 3.7. Shown in Fig. 3.18 is a comparison of values calculated from this equation with results from more than 200 analyses with different combinations of column and beam geometries. It should be emphasized that Eq. 3.8 is appropriate for interior cracks with a fillet weld reinforcement of at least 3/8-inch leg dimension. For cases with less reinforcement, it is probably more appropriate to use the edge crack equation for all cases given the likelihood that interior cracks may be quite close to the surface of the weld and are prone to “popping through” to the surface. Finally, no three-dimensional analyses with interior cracks were run, but it is assumed that the same 1.6 factor to relate $K_{I,3D}/K_{I,2D}$ would apply to interior cracks as well as edge cracks in Eq. 3.6.

3.4 Residual Stress Effects

In an earlier study, Chi et al. (1997) conducted preliminary analyses on the effect of welding-induced residual stresses on fracture, but due to limitations of the modeling techniques applied at the time, the results were inconclusive. In this study we have revisited the question of residual stresses to assess their significance on elastic ($K_I$) and inelastic (CTOD) toughness demands. An overview of the techniques to model residual stresses and the elastic results are included in this section. Results of inelastic analyses are included in Chapter 4.

Residual stresses can alter the fracture behavior and resistance of the connections in a few ways. First, tensile residual stresses that are transverse to the weld (parallel to the beam flange) will tend to add to externally imposed stresses, thus increasing the resulting toughness demand. This behavior is probably more significant for elastic behavior since large scale yielding that occurs for inelastic analyses will tend to “wash out” the residual stresses. Second, residual stresses longitudinal to the weld will alter the yielding behavior. For example, assuming that longitudinal tensile stresses in the weld are counterbalanced by compressive stresses in the adjacent flange material, under transverse tensile loading of the weld, the residual stresses will tend to accelerate yielding in the adjacent base material and retard yielding of the weld. This, in essence, tends to create an apparent overmatching condition which should improve the weld behavior by limiting inelastic loading at the crack tip. Lastly, aside from their influence on the applied stresses, the residual stresses will alter constraint conditions in the welds, and thus will lead to variations in the apparent (measured) fracture toughness of the material.

**Residual Stress Simulations:** Using a complex thermo-mechanical technique to simulate the heating/cooling and phase transformation process in welds, Zhang and Dong (1998) modeled welding induced stresses in a typical beam flange groove weld for a W36 x 150 beam. As shown by the dashed line in Fig. 3.19, their simulations predict a distribution of transverse stresses that are in tension at the top and bottom of the weld balanced by compression stresses inside the weld. The jaggedness of the stress plot is due to the pass-by-pass simulation of the welding procedure where Zhang and Dong assumed the weld to be made using nine weld passes. Maximum stresses are on the order of $\pm 40$ ksi, except for very large peak tensile stresses at the lower face of the weld. These peak stresses are created when the upper surface of the weld cools and moment equilibrium leads to large tensile stresses at the bottom of the weld to counteract those induced by the cooling/contracting upper surface of the weld. The large peak in stresses at
the bottom of the beam flanges will impose large stress intensity factors at the backing bar/weld root defect.

_Eigenstrain Modeling Technique:_ Using as a target the residual stress distribution simulated by Zhang and Dong (1998), we have utilized an eigenstrain technique proposed by Hill and Nelson (1995) with subsequent modifications by Matos and Dodds (2000) and Chi (1999) to input the residual stresses into the finite element model. As described in further detail by Chi (1999), basically this technique involves imposing eigenstrains (or inherent strains) in the finite element model by assigning anisotropic thermal strains. This is accomplished by defining anisotropic expansion coefficients to each element and then performing a thermal equilibrium analysis under a uniform temperature increase.

_Residual Stress Distributions and Imposed Inherent Elastic Fracture Demand:_ The residual stresses modeled in the two-dimensional fracture analyses using the eigenstrain technique are shown in Fig. 3.19. Contour plots of transverse, $\sigma_{xx}$, and longitudinal (out-of-plane), $\sigma_{zz}$, stresses are shown in Fig. 3.19b and 3.19c, respectively, and in Fig. 3.19a the $\sigma_{xx}$ stresses are compared to those predicted by Zhang and Dong’s simulation. Assuming an initial weld root crack of $a_o = 0.1$ inch, the stress intensity factor caused by the residual stresses is equal to $K_I = 21$ ksi$\sqrt{\text{in}}$. This corresponds to about one-third of the fracture toughness of $K_{IC} = 60$ to 75 ksi$\sqrt{\text{in}}$. for low toughness E70T-4 welds. Alternatively, assuming the relation that $K_I/\sigma_n = 2\sqrt{\text{in}}$, the fracture toughness demand associated with residual stresses is about equal to that created by an applied bending stress of $\sigma_n = 10$ ksi.

### 3.5 Summary

The following summarizes major findings in this chapter for the linear-elastic fracture behavior as characterized through the mode I stress intensity factor, $K_I$:

- $K_I$ values for weld root defects at backing bars are shown to be insensitive to the backing bar thickness and fusion length (Fig. 3.2 and 3.3). Further, stress intensity factors for these weld root defects are similar to those for any small edge defects in the weld or HAZ on the lower surface of the bottom beam flange (Fig. 3.4).

- $K_I$ values for initial flaws on the upper side of the bottom flange are approximately one-fourth of those for similar size flaws on the lower surface of the flange (Fig. 3.4). Thus, in the bottom flange the likely weld root flaw locations at the backing bar gap occur at the point of maximum stress and $K_I$. This is in contrast to conditions for the top flange, where the backing bar gap and weld root defects on are the inside (lower) surface of the flange where the stresses and $K_I$ are lower. The large difference in stresses between the inner and outside face of the flange, combined with the different flaw locations, helps to explain the prevalence of observed bottom versus top flange fractures.

- Reinforcing fillet welds used to seal backing bar gaps reduce $K_I$ values for weld root defects to about one-third to one-half of their unreinforced values (Fig. 3.5).
• Equations are developed to estimate $K_I$ for edge and interior flaws as a function of the applied stress and geometry of the connections (Fig. 3.7 and Eq. 3.6 to 3.8). In general, $K_I$ is most sensitive to the initial flaw length and the column/joint panel properties and is less sensitive to changes in the beam geometry. For edge defects of $a_o = 0.1$ inch, the maximum value of $K_I$ ranges from about $K_I/\sigma_n = 2$ to $3\sqrt{\text{in}}$ (Fig. 3.13 and Eq. 3.6). For interior defects with $a_o = 0.1$ inch and 3/8 inch fillet reinforcement, the maximum $K_I$ ranges from about $K_I/\sigma_n = 0.5$ to $1.2\sqrt{\text{in}}$ (Fig. 3.18 and Eq. 3.6).

• Welding induced residual stresses are shown to generate $K_I = 20 \text{ ksi} \times \sqrt{\text{in}}$. for an edge defect with $a_o = 0.1$ inch. This is roughly equivalent to the toughness demand imposed by a nominal applied bending stress of $\sigma_n = 10 \text{ ksi}$. 
4. Inelastic Behavior and CTOD Toughness Demands

4.1 Overview

Compared to the elastic behavior investigated in Chapter 3, inelastic fracture is influenced by a broader range of variables whose analysis is more computationally demanding. Therefore, it is not feasible to study all possible effects through as systematic a parameter study as was done for elastic response. Rather, our approach in this chapter is to conduct selective inelastic analyses of specific parameters and sets of inter-related parameters. Among the connection design and detailing parameters considered are the following (1) beam and column sizes, (2) yield strengths of the beam, column and weld metal, (3) location and size of initial cracks or flaws, (4) welding-induced residual stresses, and (5) connection details, e.g., continuity plates, Reduced Beam Section, weld profile and access hole geometry, etc. Unless otherwise noted, all analyses in this chapter are for the general connection geometry of Fig. 1.1 with flaws of length $a_o = 0.1$ inch continuous across the beam flange.

The chapter begins (Subsection 4.2) with an overview of general inelastic behavior by reviewing inelastic analysis results from a previous SAC Project (Chi et al., 1997). This is followed in Section 4.3 with a series of analyses modeled after the SAC connection tests conducted at the University of Michigan (Stojadinovic, et. al. 1998). From these analyses, a simplified equation is developed to estimate CTOD as a function of the inelastic connection rotation. Next, the combined effects of panel zone strength, column flange thickness, and continuity plates are studied in Subsection 4.4 and the influence of residual stresses and weld metal strength in Subsection 4.5. In Subsection 4.6, the SMCS ductile fracture criterion (introduced in Section 2.8) is used to investigate the influence of weld access hole geometry. Finally, effectiveness of the Reduced Beam Section (RBS) detail, relative to the standard detail, is evaluated in Section 4.6, followed in Section 4.7 by a brief recapitulation of major findings.

4.2 General Behavior and Trends

Summarized in Fig. 4.1 through 4.6 are general characteristics of inelastic fracture behavior from a previous study by the authors (Chi et al. 1997, 2000) that are extended herein. These analyses are modeled after connection subassemblies tested during SAC Phase I, consisting of a W36 x 150 beam connected to a W14 x 257 using the standard pre-Northridge connection detail, i.e., a bolted-web welded-flange detail with weld backing bars left in place. Material yield strengths for three connection specimens (UCB-L, UCB-H, and Generic) are summarized in Table 4.1, along with corresponding beam and joint panel strengths. Two values of beam and joint panel strengths are reported, one corresponding to yield and the second to the full plastic strength. Properties for UCB-L and UCB-H are from measured properties of connections tested by Popov et al. (1996) and those for the “generic” case are the average stress-strain properties for Gr. 50 rolled shapes and E70 weld metal, as reported in section 2.7 of this report. The primary difference between UCB-L and UCB-H is in the beam yield strengths ($F_{y,flange} = 40$ ksi in UCB-L versus $F_{y,flange} = 60$ ksi in UCB-H), an effect that alters both the relative beam-to-joint panel
strength and the weld-to-base metal strength. The generic properties lie between those of the two UCB connections.

Referring to Table 4.1, the strength of UCB-H ($M_n = M_{n,j} = 29,020$ k-in) is clearly limited by the nominal joint panel strength, whereas in UCB-L and the Generic case the beam plastic moment ($M_{pb}$) is less than the nominal joint strength ($M_{n,j}$) but larger than the joint yield strength ($M_{y,j}$). Thus, in all cases joint panel zone deformations significantly contribute to the total connection rotation, although the contribution is largest in UCB-H. Referring to Fig. 4.1a, the limiting strength of the joint panel explains the small difference in overall load-deflection behavior between UCB-L and UCB-H. The relative rotation contribution from the beam and joint panel are compared in Fig. 4.1b where, for example, at beam tip displacement of $\Delta_{tip} = 5$ inches panel zone deformations account for about 60% of the total in UCB-L and 80% in UCB-H.

Following the definitions of total and inelastic rotation components given in Fig. 4.2, the beam displacement of $\Delta_{tip} = 5$ inches roughly corresponds to a total connection rotation of $\theta_{total} = 0.045$ radian with an inelastic connection rotation of $\theta_{inelastic} = 0.03$ radian. The definition of $\theta_{inelastic}$ follows that of the AISC testing protocol (AISC 1997). As shown by the plastic strain contours (Fig. 4.1c-d), there is significant joint panel yielding in both cases, with modest beam yielding in UCB-L and only very localized beam yielding in UCB-H.

Compared in Fig. 4.3 are CTOD demands in UCB-L and UCB-H from two- and three-dimensional analyses. Results of two-dimensional analyses (Fig. 4.3a) reveal large differences in CTOD, attributed in large part to shielding of the crack tip associated with large overmatching weld strengths in UCB-L ($F_{y,w}/F_{y,f} = 1.5$) compared to exact matching conditions in UCB-H ($F_{y,w}/F_{y,f} = 1.0$). This shielding behavior is illustrated by comparing the weld plastification in Fig. 4.4 at the two load levels A and B, referenced in Fig. 4.3a. As described previously by Chi et al. (1997, 2000), driven by the high stress concentration at the weld root, the large overmatching in UCB-L forces plastification into the adjacent beam flange and thereby shields the crack tip from increasing stresses/strains between load levels A and B. On the other hand, in UCB-H, all of the yielding is concentrated directly around the weld root flaw, leading to much larger CTOD demands. One of the conclusions from this is that weld overmatching can serve to reduce fracture toughness demands for weld root flaws. Note, however, that the benefits of overmatching would not generally apply to flaws in other locations, such as at the weld-to-beam flange interface where shielding of the crack tip would not occur.

Results of two- and three-dimensional analyses for UCB-L are compared in Fig. 4.3b, where three-dimensional results are plotted for three locations across the beam flange. As with the elastic analyses, CTOD is largest directly beneath the beam web, and although somewhat diminished, the shielding effect is still evident. Elastic versus inelastic analysis results are compared in Fig. 4.3c, where differences in the calculated toughness demands become apparent beyond a nominal flange bending stress of about $\sigma_b = 30$ to 40 ksi or roughly 80% of the base metal yield strength. Finally, CTOD results from two- and three-dimensional analyses of UCB-L and UCB-H are plotted versus inelastic connection rotation in Fig. 4.3d. Here the largest (3D) toughness demands at $\theta_{inelastic} = 0.03$ radian are about CTOD = 0.008 inch and 0.025 inch for UCB-L and UCB-H, respectively. The large difference arises from the combined effects of weld matching, beam stresses at a given connection rotation, and the amount of panel zone deformations, all of which are more critical for UCB-H compared to UCB-L.
Two-dimensional analysis results from UCB-L and UCB-H are compared to those of the Generic specimen in Fig. 4.5. Together, the four plots in this figure show relations between nominal applied bending stress, displacements, inelastic rotations, and CTOD. Referring back to Table 4.1, the Generic specimen has joint panel-to-beam strength ratios \( M_{y,j} / M_{pb} = 0.83 \) and \( M_{n,j} / M_{pb} = 1.04 \) and a weld matching ratio \( F_{y,w} / F_{y,b} = 1.18 \), lying between those of UCB-L and UCB-H. Thus, it follows that the CTOD versus \( \theta_{inelastic} \) response (Fig. 4.5d) of the Generic specimen falls between that of UCB-L and UCB-H, but is closer to UCB-L. Although the Generic specimen carries higher nominal bending stresses at \( \theta_{inelastic} = 0.03 \) radian due to its stronger panel zone (Fig. 4.5a), it also experiences lower CTOD for a given bending stress (Fig. 4.5c) - presumably because of smaller panel zone deformations and a larger weld metal yield strength than the UCB specimens. From Fig. 4.5d, the CTOD demands for the Generic case are fairly modest with CTOD \( \approx 0.005 \) inch at \( \theta_{inelastic} = 0.03 \) radians, versus CTOD \( < 0.003 \) inch for UCB-L. Recall, however, that these results are from two-dimensional analyses which are valid for comparative analyses but not for determining absolute toughness demands for design.

A final comparison presented in Fig. 4.6 contrasts CTOD demand at three flaw locations for two variations of the Generic connection. Results in Fig. 4.6a are for the basic Generic connection described above, and results in Fig. 4.6b are for a case where the joint panel zone thickness has been doubled, thereby increasing the joint panel-to-beam strength \( M_{n,j} / M_{pb} > 2.0 \) so that panel zone deformations are only a small fraction of the total connection rotation. Several important observations can be drawn from Fig. 4.6. First, CTOD demand is significantly larger (on the order of three times larger) for the same size flaws located at the weld-to-beam flange interface compared to those at the weld-to-column flange interface. This is quite different from the elastic response where \( K_I \) demands at the two locations are almost equal. We also see that CTOD demand is very small for interior flaws, CTOD \( < 0.001 \) inch, following trends noted in the elastic \( K_I \) analyses. A third major observation is that for a given inelastic rotation, CTOD demand is significantly smaller for the case with a strong panel zone. At \( \theta_{inelastic} = 0.03 \) radians, the CTOD for edge cracks at the weld-to-column edge interface for the connection with the strong panel zone are just about half those for the standard Generic case with the weaker panel zone. For flaws at the weld-to-beam flange interface, CTOD for the stronger panel zone connection is about two-thirds of that for the weaker panel zone case.

The inelastic analyses described up this point are meant to provide an overview of some important inelastic behavioral issues. In the following sections, these effects and their implication on toughness demands (CTOD) are more systematically analyzed.

4.3 Analysis of SAC/Michigan Connection Tests

With the motive to evaluate a representative sample of beam and column proportions, we developed and performed a series of analyses modeled after the SAC test program conducted at the University of Michigan (Stojadinovic, et. al. 1998). The connection detail is similar to the basic pre-Northridge geometry, except that the backing bars are removed and the weld is built-up as shown in Fig. 4.7a. Two- and three-dimensional finite element models used to analyze these cases are shown in Figs. 4.7b and 4.7c, respectively. The analyses are for an \( a_o = 0.1 \) inch deep
surface crack located at the weld-to-beam flange interface, which as indicated above is more severe than for a flaw at the weld-to-column flange interface. The specimens tested by Stojadinovic et. al. (1998) typically failed by extensive ductile tearing initiating through ductile fractures at the weld-to-beam flange interface. Combinations of beam and column sizes included in the study are summarized in Table 4.2. To expand the study to cover a broad range of joint panel zone strengths, in addition to the five basic cases included in the test program, we analyzed connections with joint panel Doubler Plates (DP) of thickness equal to 0.5\(t_{wc}\) and 1.0\(t_{wc}\), where \(t_{wc}\) is the column web thickness. As indicated in Table 4.2, the resulting joint panel to beam strength ratios range from \(M_{n,jp}/M_{p,b} = 0.75\) to 2.78.

Shown in Fig. 4.8 are comparisons of CTOD versus connection rotation for three connections with joint panel to beam strengths of \(M_{n,jp}/M_{p,b} = 0.75\), 1.04 and 2.78. Distinguished between the component deformations, \(\theta_{total}, \theta_{jp}\) and \(\theta_{beam}\), the plots in Fig. 4.8 describe differences in the CTOD versus deformation behavior that arise from panel zone versus beam hinge deformations. Connection 1-1 with \(M_{n,jp}/M_{p,b} = 1.04\) is the standard case with a W14 x 257 column and W36 x 150 beam that forms the benchmark for many of the analyses in this report. Case 2-3 (\(M_{n,jp}/M_{p,b} = 2.78\)) has a strong panel zone and is dominated by beam yielding. For this connection the CTOD versus \(\theta_{beam}\) plot begins with a steep slope but then has a distinct kink where slope of CTOD demand decreases. This kink occurs as the beam becomes fully yielded at the column face and yielding progresses away from the column into the beam span. Alternatively, case 4-1 with a weak panel zone (\(M_{n,jp}/M_{p,b} = 0.75\)) is dominated by panel zone deformations and the CTOD demand increases fairly linearly with \(\theta_{jp}\) while there is limited beam rotation \(\theta_{beam}\). Case 1-1, with a balanced beam and joint strength, experiences CTOD contributions from both \(\theta_{jp}\) and \(\theta_{beam}\), and due to the manner that CTOD increases, Case 1-1 has a larger CTOD demand than either of the other two connections.

Based on the observations from Fig. 4.8, the total CTOD demand is approximated as the sum of three components,

\[
CTOD_{total} = CTOD_{o} + CTOD_{jp} + CTOD_{beam}
\]  

(4.1)

where CTOD\(_{o}\) is the initial elastic value required to reach yield, and CTOD\(_{jp}\) and CTOD\(_{beam}\) are functions of panel zone and beam hinge rotations, respectively. Inspired by the shapes of the response curves in Fig. 4.8, the CTOD\(_{jp}\) and CTOD\(_{beam}\) components are further related to their respective rotations by the following equation, shown schematically in Fig. 4.9:

\[
CTOD = \frac{C_{1}\theta}{[1 + (C_{1}\theta/C_{2})^{2}]}^{1/2} + C_{3}\theta
\]  

(4.2)

where \(C_{1}\), \(C_{2}\), and \(C_{3}\), are constants and \(\theta\) is the rotation component. The initial linear slope is approximately equal to \(C_{1}\), and the asymptotic slope at large rotations is \(C_{3}\). The coefficient \(C_{2}\) controls the rate of the transition between the two linear segments.

Shown in Fig. 4.10a are CTOD demands calculated from the 15 cases in Table 4.2, plotted versus the total connection rotation. These same CTOD values are plotted against the two
rotation components, \( \theta_{jp} \) and \( \theta_{beam} \), in Fig. 4.10b. Fitting Eqs. 4.1 and 4.2 to this data, the following values are obtained:

\[
CTOD_o = 0.8 \times 10^{-3} \tag{4.3a}
\]
\[
CTOD_{jp} = \frac{0.26(\theta_{jp} - \theta_o)}{1 + \left( \frac{0.26(\theta_{jp} - \theta_o)}{0.0014} \right)^2} + 0.14(\theta_{jp} - \theta_o) \tag{4.3b}
\]
\[
CTOD_{beam} = \frac{0.66(\theta_{beam} - \theta_o)}{1 + \left( \frac{0.66(\theta_{beam} - \theta_o)}{0.0054} \right)^2} \tag{4.3c}
\]

where \( \theta_o = 0.0025 \) radian is the estimated initial yield value for joint panel and beam rotations at \( CTOD = CTOD_o \). As seen in the resulting coefficients (and following from Fig. 4.8), \( CTOD_{jp} \) is nearly linearly related to \( \theta_{jp} \), but the relationship between \( CTOD_{beam} \) and \( \theta_{beam} \) is more bi-linear.

CTOD values from the finite element analyses and the calibration equation are shown to compare fairly well in Fig. 4.10c.

As developed above, Eqs. 4.3a-c are calibrated to two-dimensional analysis results, which are known to underestimate the maximum three-dimensional toughness demand. Therefore, the next step is to consider results from three-dimensional analyses, which provide a basis for refining the calibration. Given the intensive computational demands of three-dimensional analyses, only a limited set of three connections (Cases 1-1, 1-3, and 5-1) are analyzed, but these are chosen to bracket the range of behavior in the 15 cases of Table 4.2. Shown in Figs. 4.11a through 4.11c are comparisons of two- and three-dimensional results for CTOD across the beam flange width at beam tip displacements of 1 to 4 inches, roughly corresponding to the onset of plastification up to rotations of \( \theta_{inelastic} = 0.02 \) radian or \( \theta_{total} = 0.035 \) radian. As expected, CTOD varies considerably across the flange width and often reaches its maximum value at the center of the flange under the web (Fig. 4.11c).

Summarized in Fig. 4.11d are ratios of three-dimensional CTOD, measured at the beam centerline, to the two-dimensional results. Between \( \theta_{inelastic} = 0.02 \) to 0.04 radian, the ratio of \( CTOD_{3D}/CTOD_{2D} \approx 1.2 \) to 1.6. These values correspond to the beam centerline and do not, therefore, reflect the spike in CTOD at the flange tip, which in some cases is larger than the CTOD at the beam centerline. In particular, for Case 1-3 (W36x150 beam to W14x257 column with doubler plates, Fig. 4.11b) the CTOD value at the free surface is about 1.5 times the mid-plane CTOD or about 1.8 times \( CTOD_{2D} \). As observed from the deformed FEM mesh and stress/strain contours, these spikes at the flange tips are apparently due to “dishing” of the beam flange as it is pulled up by the beam web. Interpretation of these large \( CTOD \) values at the flange tip are not straightforward, however, due to the fact that (1) the initial defects would typically be smaller at the flange tip than beneath the web, and (2) the out-of-plane conditions at the flange tip are closer to plane stress than plane strain, under which conditions the material has a higher
apparent toughness. Therefore, even though the calculated flange tip values are sometimes larger than the centerline values, we will assume that the centerline values represent the most critical condition.

Based on the ratios shown in Fig. 4.11d, in the absence of more accurate information it seems reasonable to assume a simple multiplier of 1.6 to convert from two- to three-dimensional CTOD for these connections. An alternative approach that would not require a fixed multiplier, and may therefore be more accurate, would be to compare CTOD to beam flange strains and, thereby, relate two-dimensional fracture indices to three-dimensional conditions based on the ratio of strains in two- versus three-dimensional continuum analyses. For example, shown in Fig. 4.12 is a comparison of CTOD versus normalized average fiber strain from the two- and three-dimensional fracture analyses. The results are very consistent with one another, suggesting a linear relationship of CTOD = 0.4ε_{average}, where CTOD is in inch units and ε_{average} is measured over a one-inch gage length along the fibers crossing the initial flaw location. The coefficient (0.4) in this linear relationship is unique to the geometry, material properties, and flaw size in this study, but presumably the linearity of the CTOD-strain relationship generally applies. Thus, one could envision developing similar coefficients for other connections using two-dimensional fracture analyses that would then be used to predict the peak three-dimensional CTOD based on strains measured from conventional three-dimensional continuum analyses, such as the type conducted by El-Tawil et al. (1998).

As the last comparison in this series of analyses, CTOD data presented in Fig. 4.13 describe the change in CTOD as a function of the ratio of panel zone to beam strength (M_{nj}/M_{pb}). Two sets of results are shown. Results in Fig. 4.13a are for the connection geometry investigated in this section, i.e., the detail shown in Fig. 4.7 with flaws at the weld-to-beam flange interface. The second set of results in Fig. 4.13b are for details without weld reinforcement where the flaws are located at the weld-to-column interface, i.e., similar to the detail analyzed in Section 4.2. Of particular interest is that the maximum CTOD for both cases occurs at about M_{nj}/M_{pb} = 1.0, and for connections with strong panel zones (M_{nj}/M_{pb} > 2) the CTOD is reduced in half compared to these maximum values. This runs counter to the prevailing design philosophy that permits (and in some cases encourages) connections with matched beam to joint panel strengths. Large differences in CTOD between the two flaw locations (Fig. 4.13a versus 4.13b) follow the trend noted previously in Fig. 4.6 where the CTOD for weld-to-column flaws is less than half that for the weld-to-beam flaw locations. This behavior results from the fact that cracks at the weld-to-column interface are in a less constrained region subjected to larger strains and do not benefit from the shielding effect that helps limit CTOD for the weld root cracks. However, the low-constraint conditions at the weld-to-beam interface also mean that the CTOD probably overestimates the actual fracture toughness demand, as compared to demands calculated for more highly constrained conditions. Therefore, the true difference in the likelihood of fracture is probably less than that indicated by the difference in CTOD demand in Figs. 4.13a and 4.13b.

4.4 Parametric Study of Panel Zone, Flange Thickness and Continuity Plates

To establish the combined influence of continuity plates, panel zone strength, and flange thickness on fracture toughness demands, we performed three-dimensional fracture analyses of
several beam-column connections with an \( a_p = 0.1 \) -inch weld root defect at the weld-to-column flange interface. Thus, the connection geometry and weld detail are similar to the pre-Northridge connections discussed in Section 4.2. The weld root crack location is selected for this study since it is expected to be most sensitive to variations in the column flange thickness and panel zone strength.

Investigation of these combined effects begins with the basic geometry using a W36x150 beam and W14x257 column, followed by modifications to (a) remove the flange continuity plates i.e., the stiffener plates welded inside the column flanges, (b) add web doubler plates in the panel zone, and (c) change to alternative column sizes with different joint panel proportions and flange thickness. Summarized in Table 4.3 are the four column size/panel zone models investigated, each of which is analyzed with and without continuity plates for a total of eight three-dimensional analyses. Material properties are based on the “generic” properties previously cited, i.e., \( F_y,\text{base} = 55 \) ksi and \( F_y,\text{weld} = 65 \) ksi.

Referring to Table 4.3, rationales for the choice of column sizes and panel zone strengths are as follows. Case 1 is the basic case with a W14x257 column in which most of the inelastic connection deformation occurs through panel zone yielding. In case 2, the joint panel zone thickness is doubled so as to force nearly all of the panel zone deformations into the beam plastic hinge. Therefore, cases 1 and 2 serve to bracket the range of likely panel zone participation that should occur in seismically designed connections. In case 3 the column size is increased to W14x550 so as to double the flange thickness. The change in larger column size strengthens the panel zone slightly relative to case 2, but it is assumed that the difference in panel zone strengths between case 2 and case 3 will not affect the behavior since the panels in both cases are strong enough to behave elastically. Therefore, the main difference between cases 2 and 3 is in the column flange thickness. In case 4, the column size is changed to a W24 shape so as to reduce the flange thickness to about one-half of that in the W14x257 while maintaining enough bending capacity to prevent flexural yielding of the column. Panel zone doubler plates are also added to minimize panel zone yielding in case 4. While the ratio of panel zone to beam plastic hinge strength \( (M_{n,jp}/M_{pb}) \) for case 4 is not as large as cases 2 and 3, the analyses indicate that the panel zone of case 4 remains fairly elastic during loading.

Compared in Fig. 4.14 are CTOD demands across the beam flange at beam tip displacements of \( \Delta_{\text{tip}} = 2 \) and 4 inches, which roughly correspond to inelastic connection rotations of \( \theta_{\text{inelastic}} = 0.007 \) and 0.021 rad. Shown in Fig. 4.14a are results for the basic case 1 with and without continuity plates, where much of the inelastic rotation is due to panel zone deformations. Shown in Fig. 4.14b are results for case 2, same column size as case 1 but with doubler plates, such that all of the inelastic deformation is shifted into the beam plastic hinge. Contour plots of plastic strains from these analyses are shown in Figs. 4.15 through 4.18, where the plots in Fig. 4.15 and 4.17 correspond to the mid-plane of the members.

Referring to Fig. 4.14a and Fig. 4.16, in case 1 with a weak panel zone there is a steep gradient of stress/strain and CTOD across the beam flange width – behavior that is more pronounced in the connection without continuity plates. At the location of maximum fracture toughness demand (beneath the beam web) the case without continuity plates has a peak CTOD about 30% larger than with continuity plates - a change from CTOD = 0.0085 to 0.0110 inch.
Contrasting case 1 (Fig. 4.14a, 4.15 and 4.16) to case 2 (Fig. 4.14b, 4.17 and Fig. 4.18), we see that stiffening the joint panel with web doubler plates (case 2) dramatically reduces both the overall CTOD demand and the large gradient across the beam flange. The peak CTOD for case 2 is reduced to about half that in case 1. Referring to Fig. 4.17, for case 2 most of the plastic deformations occur in the beam plastic hinge away from the column face. This tends to equalize the stresses/strains across the beam flange width before they reach the column face, thereby explaining the flatter CTOD curves in Fig. 4.14b. For case 2, removal of the continuity plates increases the CTOD at \( \Delta_t = 4 \) inches by about 45% from CTOD = 0.0038 to 0.0057 inch. While this is a larger relative increase than in case 1, the absolute increase in CTOD due to removal of the continuity plates is less in case 2. Comparing the plastic strain distributions of cases 1 and 2 with and without continuity plates (Fig. 4.15 and 4.16 versus Fig. 4.17 and 4.18), the continuity plates play a greater role toward equalizing the stress/strain distributions in case 1 with the weaker panel zone.

Compared in Fig. 4.19 are results for cases 2, 3, and 4 that reflect the change in CTOD as a function of the column flange thickness, \( t_{fc} \). As mentioned earlier, all three of these cases have relatively strong joint panel zones, such that the main difference between them is in column flange thickness that varies in the proportions 4:2:1 for cases 3, 2, and 4, respectively. Comparing results with and without continuity plates, these figures clearly demonstrate that the influence of continuity plates is very much a function of the column flange thickness. Case 3 with the W14 x 550 column represents a limiting condition where the column flanges are thick enough to resist transverse bending and thus the continuity plates have essentially no effect. On the other hand, for case 4 with a thin flange, CTOD increases by about 60% when the continuity plates are not present.

Aside from the expected interaction between column flange thickness and continuity plates, the analyses indicate that even with continuity plates present, the CTOD demand is significantly influenced by the column flange thickness. This trend is clearly seen in Fig. 4.20, where the maximum centerline values of CTOD from Fig. 4.19 (measured under the beam web) are plotted versus flange thickness. For cases with continuity plates, the maximum CTOD = 0.010 for case 4 with \( t_{fc} = 0.96 \) inch is 4 times larger than the minimum value of CTOD = 0.0024 inch for case 3 with \( t_{fc} = 3.82 \) inch. For the cases without continuity plates the change is ever larger - an increase of 6.5 times from CTOD = 0.0025 to 0.016 inch. Particularly for the cases with continuity plates where bending of the column flange is restrained, this strong correlation between the column flange thickness and CTOD is somewhat surprising. The interdependence of toughness demand and column flange thickness is also observed in the elastic parametric study of Chapter 3.

Another trend observed for case 3, with thick flanges and strong panel zones, is that the maximum CTOD demand shifts from the beam centerline to flange tips – behavior also observed in previous analyses (e.g., Fig. 4.11). This is most apparent in Fig. 4.19a where the CTOD is fairly constant across the beam flange until it approaches the flange tips. Such behavior is present in all the analyses but is most significant in case 3 where the tip CTOD exceeds that at the beam centerline.
4.5 Residual Stresses and Weld Matching Ratio

The effect of welding-induced residual stresses on inelastic CTOD behavior is examined in this section. Variations in weld yield strengths - or more specifically the weld-to-base metal strength ratio – are investigated since weld matching and residual stress effects are known to be interrelated. Residual stresses are input to the model using the eigenstrain approach described in Chapter 3, with the residual stress pattern shown previously in Fig. 3.19. Results are presented for two-dimensional inelastic analyses with a weld root defect of \(a_o = 0.1\) inch.

Residual Stress: Plots of CTOD for the basic specimen with generic stress-strain properties \((F_{y,\text{base}} = 55\) ksi and \(F_{y,\text{weld}} = 65\) ksi) are shown in Fig. 4.21 where CTOD from analyses with and without residual stresses are compared. Referring to Fig. 4.21a, at low loads where the behavior is elastic, the primary effect of residual stresses is to impose an initial demand of CTOD = 0.0001 inch, which is consistent with \(K_I = 20\) ksi\(\sqrt{\text{in}}\) that was calculated using elastic analyses in Chapter 3. CTOD = 0.0001 inch is slightly less than the toughness demand created by an externally applied bending stress of about \(\sigma_b = 10\) ksi. Referring to Fig. 4.21a, when shifted up by \(\sigma_b = 10\) ksi, the plot of CTOD with residual stresses would match the curve without residual stresses up to about \(\sigma_b = 50\) ksi. Beyond this, small differences arise due to the pattern of yielding, but then the curves converge at stresses of around \(\sigma_b = 55\) ksi. Note that the curve for the analysis with residual stresses does not appear to exhibit any overmatching behavior because the tensile residual stresses lead to early yielding around the crack tip. Referring to the plot of CTOD versus inelastic rotation in Fig. 4.21b, we again see that the effect of residual stresses becomes less significant at larger rotations. For example, at \(\theta_{\text{inelastic}} = 0.03\) rad., the residual stresses account for only a 10 to 15% increase in CTOD.

Welding Matching: To examine the influence of weld matching strength, and the possible influence of residual stress effects on weld matching, we next ran analyses where the weld metal yield strength was varied from \(F_{y,\text{weld}} = 45\) to 85 ksi, holding the base metal strength constant at \(F_{y,\text{base}} = 55\) ksi. This results in a range matching ratios of \(F_{y,\text{weld}}/F_{y,\text{base}} = 0.82, 1.00, 1.18, 1.36, \) and 1.55. Comparison plots of CTOD versus bending stress and inelastic rotation are shown in Fig. 4.22 and Fig. 4.23. In general, the plots with and without residual stresses (Fig. 4.22a versus 4.22b and Fig 4.23) show only modest differences due to residual stresses. However, the CTOD versus \(\theta_{\text{inelastic}}\) plots in Fig. 4.22 reveal dramatic differences due to weld matching, where connections with matching \((F_{y,\text{weld}} = 55\) ksi) or undermatching \((F_{y,\text{weld}} = 45\) ksi) welds lead to extremely large CTOD. On the other hand, even the small overmatching provided by the basic case \((F_{y,\text{weld}} = 65\) ksi) is shown to effectively limit the CTOD demands. Higher overmatching strengths \((F_{y,\text{weld}} = 75\) and 85 ksi) lead to further reductions, but not as dramatic as the difference achieved going from \(F_{y,\text{weld}} = 55\) to 65 ksi. In considering the large influence that small changes in weld strength can have, it is important to recognize the large variability that can occur in weld metals due to the welding procedure. As reported in Chapter 2, average measured yield strengths of E70 weld metals ranged from \(F_{y,\text{weld}} = 55\) to 75 ksi, which can easily tip the balance from an undermatched to overmatched condition. The comparison of results with and without residual stresses in Fig. 4.23 confirms the previous observation that the residual stress effects are less important at higher deformations. The main effect seems to be that the residual stresses introduce an initial offset in CTOD that is fairly constant throughout the loading.
4.6 Influence of Weld Access Hole and Residual Stresses Using SMCS Model

As discussed in Section 2.8, the Stress Modified Critical Strain (SMCS) model provides an alternative to conventional fracture indices to evaluate ductile fracture initiation based on the combined effects of plastic strains and stress triaxiality (the ratio of hydrostatic to effective von Mises stress). Advantages of the model are that (a) it can be applied in a continuum analysis without requiring definition of an initial flaw, and (b) it is not limited by the small scale yielding requirements of traditional fracture indices. In this section, SMCS is used to investigate the effects of weld access hole details and residual stresses on ductile fracture initiation. As shown in Fig. 4.24, the three-dimensional finite element mesh for these analyses is finely discretized in the beam weld region, even though there is no crack tip modeled in this case. For these analyses the SMCS criterion is calculated with a material constant of $\alpha = 1$, which is assumed as a lower bound estimate of the strain capacity for the weld metal and HAZ.

Compared in Figs. 4.25 through 4.29 are stress and strain indices for three cope hole details, referred to as the (a) standard cope, (b) shortened cope, and (c) shortened cope with weld reinforcement. The standard cope is one that was used in the Phase I tests by Popov et al. 1996 reviewed in Section 4.2, and the shortened cope with weld reinforcement is similar to one used in the Phase II tests by Stojadinovic, et. al. 1998 that are analyzed in Section 4.3. The contour plots in Fig. 4.25 provide an overview of behavior, while the line plots in Fig. 4.26 through 4.29 provide quantitative data for the weld HAZ regions in the standard and shortened cope hole details.

Comparing the standard and shortened cope, the only perceptible differences are at the weld-to-beam flange fusion line where all the stress/strain indices are larger in the shortened cope detail. This is most apparent in Fig. 4.27 and to some extend in the contour plots of Fig. 4.25 where the stress/strain indices reflect a sharp increase at the point where the web meets the flange at the weld toe. No significant changes are apparent at the weld root (weld-to-column flange interface). In spite of the differences between the standard and shortened cope, however, in neither case does the SMCS reach its critical value of SMCS > 0 for ductile fracture initiation. Thus, it is not clear whether differences between the two access hole details would result in perceptible differences in actual response.

Comparing the shortened cope with and without weld buildup, the weld buildup improves (lower) all the stress/strain indices at the weld root (Fig. 4.25 and 4.28), but tends to increase the values at the beam flange-to-weld interface (Fig. 4.29). As will be shown later in Chapter 5, the discontinuity between the weld and beam flange does in fact create a sharp stress/strain riser which the SMCS model detects as the most critical region around the weld detail. From these analyses, however, the SMCS does not reach its critical value of SMCS > 0, so as with the previous comparison, the data are inconclusive as to whether there would be a perceptible change in behavior due to the geometrical differences between the joints with and without the weld buildup.
The influence of residual stresses on the stress/strain indices is examined for the shortened weld access hole detail in Figs. 4.30 and 4.31. For these analyses, the initial residual stresses are input using the eigenstrain approach and have a distribution similar to that shown in Fig. 3.19. Results are compared at applied beam displacements of $\Delta_{tip} = 1$ and 4 inches, roughly corresponding to points when the beam first reaches $M_{pb}$ and at $\theta_{inelastic} = 0.025$ rad. Overall, very little difference is discernable between the analyses with and without residual stresses at $\Delta_{tip} = 1$ inch, and no differences are apparent at $\Delta_{tip} = 4$ inches. These observations are consistent with ones made in the previous section using conventional fracture indices.

The final two comparisons made using the stress/strain indices examine the influence of panel zone distortions on the shortened weld access holes without (Fig. 4.32) and with (Fig. 4.33) weld reinforcement. In both cases there are distinct differences between specimens with and without web doubler plates, where conditions at both the weld-to-column and weld-to-beam interfaces are more critical for the connections with weaker panel zones. Differences are larger at the weld-to-column interface (Fig. 4.32a and 4.33a). These trends substantiate the previous observations regarding the effect of panel zone deformations, although as in the other analyses presented in this section, in no cases does the SMCS reach its critical value.

### 4.7 Comparative Study of RBS Connection Detail

As shown in the previous sections, increasing panel zone strength reduces the fracture toughness demands by shifting inelastic deformations away from the column face into the beam plastic hinge region. This is carried one step further through the Reduced Beam Section (RBS) detail that shifts the highly strained plastic hinge region away from the critical flange weld by reducing the plastic moment strength of the beam a short distance away from the column face. Testing has shown that this can readily be achieve by reducing the beam flange area using semi-circular radius cuts in the top and bottom flanges. The main goal in sizing the RBS is to limit the maximum beam moment that can develop at the column face to about 85% to 100% of the beam’s expected plastic moment capacity.

**Geometry of RBD Cut and Finite Element Models:** Figure 4.34 shows the geometry of a radius cut RBS. According to the 50% SAC guidelines (1999), the dimensions $a$, $b$ and $c$ should meet the following criteria: $a \equiv (0.5 \text{ to } 0.75)b_f$; $b \equiv (0.65 \text{ to } 0.85)d_b$; and $c \leq 0.25b_f$. The dimension $c$ is the depth of the cut that controls the maximum moment developed at the RBS and, thereby, limit the maximum moment and shear at the column face. The upper bound applied to $c$, limits the reduction of the flange to 50% of its original width.

**Finite Element Fracture Analyses:** We conducted several three-dimensional analyses of the W14 x 257 column to W36 x 150 beam, where the RBS flange reduction is varied between 25% ($c = 0.125b_f$) and 50% ($c = 0.25b_f$) and the joint panel thickness is varied from $1/2$ to 2 times its original value. The connection has the generic material properties, and except for the trim down, the other RBS dimensions are held constant at $a = 8$ inches and $b = 24$ inches. Shown in Fig. 4.35 are the finite element models with built-in initial cracks. Table 4.4 shows a comparison of joint panel strength and beam/reduced beam strength for these connections. The joint panel
strength \( M_{n,jp} \) and beam strengths \( M_{pb} \) or \( M_{p,rbs} \) all correspond to the maximum beam moment that can be resisted at the column face.

Case 1 (see Table 4.4) is the standard connection geometry and case 2 is the same detail with a web doubler plate. The RBS for cases 3 through 6 is the maximum amount of 50% trim down permitted by the SAC Guidelines (1999), and in cases 7 and 8 the trim down is 25%. The joint panel thickness is varied in cases 4 through 6 to examine different joint panel to beam (RBS) ratios ranging between \( M_{n,jp}/M_{p,rbs} = 0.8 \) to 2.4, which bracket the ratios of 1.0 and 1.9 for the non-RBS cases 1 and 2. Cases 7 and 8 also have about the same joint panel to beam strength ratios as the basic connection.

Figure 4.36 shows a comparison of effective plastic strain contours for cases 1 and 3 (i.e., the standard connection without and with the RBS) at 4-inch beam tip displacement. For the non-RBS connection, it is obvious there is significant inelastic panel zone deformation in addition to localized yielding of the beam flanges adjacent to the weld. On the other hand, in the RBS detail there is only slight yielding in the joint panel with most of the inelastic action confined to the RBS.

CTOD demands for these connections, shown in Fig. 4.37, can be broadly characterized into three groups. The uppermost set of three curves includes the standard detail and two RBS designs – case 6 with a 50% RBS and 40% strength reduction of the joint panel zone and case 7 with a 25% RBS reduction. The similarity of response indicates that the two RBS reductions are not large enough to significantly limit inelastic panel zone deformations to reduce CTOD demand below what occurs in the standard connection. This is further indicated by the fact that the joint panel to beam strength ratios are similar for all three connections, i.e., \( M_{n,jp}/M_{pb} = 1.0 \) for case 1 and \( M_{n,jp}/M_{p,rbs} = 0.8 \) and 1.1 for case 6 and 7, respectively.

At the opposite extreme are the three lowest curves in Fig. 4.37 which demonstrate that with a fully effective RBS detail, CTOD demand can be constrained to \( \text{CTOD} < 0.003 \) inch, which is within the available toughness for all notch toughness rated weld metals. The three lowest curves are two 50% RBS details and one 25% RBS, all with strong panel zones, i.e., \( M_{n,jp}/M_{p,rbs} = 1.9, 2.4, \) and 1.9, respectively. Between the lowest and highest sets of curves are ones with intermediate toughness demands for cases 2 and 3, the standard detail with a strong panel zone and the 50% RBS detail with the original panel zone. The joint panel to beam strength ratios for these cases are \( M_{n,jp}/M_{pb} = 1.9 \) for case 2 and \( M_{n,jp}/M_{p,rbs} = 1.4 \) for case 3. For these cases the toughness demand of \( \text{CTOD} \approx 0.004 \) to 0.005 inch at \( \theta_{\text{inelastic}} \approx 0.03 \) rad is also within the range that can be resisted by notch toughness rated weld metal, however, the CTOD demand is not as tightly controlled as when yielding is confined to the RBS region of the connection. Finally, shown in Fig. 4.38 is a comparison of the CTOD distribution across the beam flange width for the standard connection with and without the RBS detail. In addition to the reduction in peak CTOD at the centerline of the beam, this figure suggests that the RBS detail will also prevent the sharp CTOD spike observed at the flange tip in some of the standard (non-RBS) details.

It can be concluded from the data in Fig. 4.37 that with proper balance between the RBS reduction relative to the joint panel zone, the CTOD demands can be contained below \( \text{CTOD} < 0.004 \) inch. On the other hand, without sufficient RBS reduction to minimize panel zone...
deformations, the toughness demands in the RBS detail can be as large as in the standard detail. Keep in mind, of course, that CTOD demands in Fig. 4.37 are based on an assumed 0.1 inch weld root defect and the demand will vary for different size flaws and flaws at other locations.

4.8 Summary and Design Implications

The inelastic analysis results in this chapter provide data and information on the effects of various design and detailing parameters that affect fracture toughness demands in the critical beam flange weld and surrounding HAZ region. The following is a summary of some of the major observations made from these analyses:

- **Weld Matching** – The relative yield strength of the weld to adjacent base material, or the so-called matching ratio – is shown to have a significant effect on CTOD for weld root defects and other flaws at the weld-to-column interface (Figs. 4.3, 4.4 and 4.22). Even the small overmatching of $F_{yw}/F_{yb} = 65/55 = 1.2$ provided by the average expected strengths of E70 weld and A572 Gr. 50 base materials helps limit CTOD demands to reasonable levels (Fig. 4.5). However, weld overmatching does not provide much benefit to flaws located at the weld-to-beam interface since the stress/strain distribution in this region does not provide shielding for the crack tip. Therefore, weld overmatching is only likely to improve the behavior for cases where the largest (most critical) defects are at the weld-to-column interface, such as the case when backing bars are left in place.

- **Crack Location** – Contrary to the elastic analyses where variations in the crack location along the lower flange surface did not have much effect, for inelastic analyses the differences can be dramatic (Fig. 4.6, Fig. 4.13). For example, at larger inelastic rotations, the CTOD for flaws at the weld-to-beam flange interface is more than twice that for flaws at the weld-to-column interface. The difference is due in large part to the higher constraint and shielding behavior that inhibits CTOD in cracks at the weld-to-beam interface since the stress/strain distribution in this region does not provide shielding for the crack tip. Thus, in terms of weld acceptance criteria, flaw sizes at the weld-to-beam flange interface should be strictly controlled.

- **Panel Zone Deformation** - The analyses indicate that for a given inelastic connection rotation, higher panel zone strengths significantly reduce CTOD demands. Interestingly, the largest demands occur where the beam hinge and panel zone have equal strength ($M_{nj}/M_{pb} = 1$), and the demand reduces in half for ($M_{nj}/M_{pb} > 2$), see Fig. 4.13. This is further confirmed by the SMCS analyses (Fig. 4.32 to 4.33). The reduction occurs because the stiff panel zone shifts the large inelastic strains away from the weld and into the beam plastic hinge at some distance away from the column face. Shifting inelastic behavior away from the weld also tends to equalize the stress and strain distributions before they reach the column face and thus decreases the gradient of the CTOD distribution across the width of the beam flange. It is recognized, however, that this analysis data appears counter to the prevailing opinion (supported by some test data) that a balanced condition will lead to the largest rotation capacity. Reasons for the difference are not clear, except that when interpreting these analysis results one must remember that they are limited by the assumptions inherent in the analyses and fracture models. Thus, the observations may not hold for cases with large
inelastic cyclic loading excursions that may change the stress/strain states from those represented by the analyses.

• Predictive CTOD Equation – An equation is developed to estimate the CTOD demand as a function of the two connection rotation components related to beam hinge and panel zone deformations. (Eq. 4.1 and 4.3, Fig. 4.9). The equation is developed for edge cracks of \( a_o = 0.1 \) inch located at the beam-to-weld interface (Fig. 4.7) and is calibrated to results from fifty two-dimensional inelastic analyses (Fig. 4.10). Comparisons of two- versus three-dimensional analyses (Fig. 4.11) are then used to estimate the peak three-dimensional CTOD demand.

• Continuity Plates - Continuity plates do help to reduce fracture toughness demands, but their effectiveness is related to the panel zone strength and the column flange thickness (Fig. 4.14 to 4.19). By resisting transverse bending of the column flange, the continuity plates decrease the gradient of the CTOD distribution across the width of the beam flange and thereby reduce the peak CTOD. For the cases investigated with weak panel zones and/or thin column flanges, the continuity plates reduce the CTOD demands by up to 60% compared to cases without continuity plates. However, if the column flange is thick enough to provide strong transverse bending resistance or the panel zone strength is high enough to shift the inelastic deformation into the beam plastic hinge, the continuity plates have little if any effect on the CTOD demand. This was the case, for example, in a connection between a W36 x 150 beam and a W14 x 550 column where the continuity plates did not affect the CTOD demand.

• Column Flange Thickness - Even when continuity plates are present to resist local distortions, the analyses indicate that the column flange thickness can have a significant effect on the fracture toughness demand (Figs. 4.19 and 4.20). This is of concern for connections are made with deeper columns (e.g., W24 to W36) where the column flanges are thinner than in W14 columns with similar moment and panel zone strength.

• Residual Stresses – At low stress levels when the connection is elastic, the inelastic CTOD values confirm that, as with the elastic analyses, the welding-induced residual stresses impose an additional toughness demand roughly equivalent to an applied bending stress of \( \sigma_d = 10 \) ksi. However, when the connection plastifies at larger inelastic deformations, the influence of residual stresses reduces to the point that at \( \theta_{inelastic} = 0.03 \) rad they only increase the toughness demands by about 10 to 15% (Figs. 4.21, 4.23). This trend for is further substantiated by the SMCS analyses where the residual stresses are shown to diminish greatly at large inelastic deformations (Fig. 4.30 to 4.31).

• Cope Details - Investigation of three alternative cope hole geometries using the SMCS model reveal some differences in stresses and strains (Figs. 4.25 to 4.29), but the analyses themselves cannot establish whether or not these differences will dramatically alter the deformation capacity.

• RBS and Panel Zone Interaction - The analyses demonstrate the importance of considering the relative strength of the RBS beam hinge and the joint panel when proportioning the RBS detail (Fig. 4.37). Where the RBS detail limits the beam moment at the column face to less
than half the joint panel strength, or alternatively $M_{nj}/M_{p,RBS} > 2$, the toughness demand is reliably limited to $\text{CTOD} < 0.003$ inch. On the other hand, for connections where the RBS beam hinge and joint panel strength are about equal, the RBS detail does not appreciably reduce the CTOD below the non-RBS detail. The data suggests it is reasonable to require a minimum joint panel to RBS strength of $M_{nj}/M_{p,RBS} > 1.5$ in order for the RBS detail to reliably limit toughness demands on the weld.
5. Transferability of Fracture Data Between Pull-Plate and Connection Tests

5.1 Overview of Pull-Plate and Weldment Studies

In conjunction with tests of full-scale connections, part of the SAC project has involved monotonic tension tests of groove-welded subassemblies to investigate various aspects of weld details, welding materials and processes, loading rate effects, and column through-thickness properties. In this chapter, analyses of two types of pull-plate component tests are described to relate data from these tests to the behavior and design criteria for beam-column connections.

Included are analyses of T-stub weldment tests (Ricles et al. 1999) and through-thickness pull-plate tests (Dexter and Melendrez 1999) conducted as part of SAC Task 7.05 and Task 5.12, respectively. The purpose of the T-stub tests (Task 7.05) are primarily to evaluate local weld detailing issues, such as the use of different welding electrodes, notched backing bars, weld reinforcement, etc. To the extent that the geometric and material parameters from these tests are known, data from these tests provide information on the insitu fracture toughness of the weldments. Through analysis, these data can then be related to conditions in full-scale beam-column connections. The through-thickness pull-plate tests (Task 5.12) have the objective to determine limiting criteria for stresses and strains in the column material behind the beam flange weld that is subjected to large transverse tension stresses.

5.2 SAC Task 7.05 T-Stub Weld Tests

The T-stub specimen is intended to simulate the bending-induced tensile stresses applied to the beam flange weld, and thereby provide an inexpensive test to evaluate weld detailing questions. As shown in Fig. 5.1, the specimen consists of a flange plate (on the right side) that is groove welded to an assembly that mimics the column flange and continuity plates. Note that the vertical stiffener plate located above the flange weld is only present during fabrication and removed prior to testing. The intent of this plate is to simulate the obstruction of the beam web and weld access hole to field welding of the flange connections, and to some extent the plate helps to control welding-induced distortions of the pull-plate. While these tests generally represent the weld geometry of beam-column connections, as will be shown there are differences between the local stress/strain fields in the T-stubs and beam-column connections which raise questions as to correlation of results between the two. Primarily, the tensile stresses in the T-stub specimen are more uniform than in beam-column connections, the latter being influenced by beam shear and column joint panel deformations. Additionally, loads are applied monotonically in the T-stub tests whereas most connection tests are cyclically loaded. While our analyses do not reflect cyclic loading characteristics, they can help to reconcile differences due to the nonuniform stress/strain fields between the specimens.

A typical finite element model for the T-stub analyses is shown in Fig. 5.2. The model is two-dimensional with similar elements and crack tip meshing characteristics as previously described for the connection analyses. Note that the weld and backing bar region shown in Fig. 5.2 corresponds to analyses for the grooved backing bar (Fig. 5.1) where the weld root fusion length
is equal to ½ inch (as originally detailed) and the flaw length is varied. Other analyses were run without the backing bar where the flaw began at the bottom of the weld, i.e., in line with the bottom of the beam flange. We ran both elastic and inelastic fracture analyses, where for inelastic analyses the material strengths were typically assumed as (1) $F_y = 58$ ksi and $F_u = 82$ ksi for the base metal and (2) $F_y = 65$ ksi and $F_u = 80$ ksi for the weld metal. These base metal strengths are equal to measured values for the flange plates reported by Ricles et al. (1999), and the weld metal strengths are the same as the average values reported in Chapter 2. Where noted below, in some analyses we varied the weld and base metal strengths to investigate questions related to weld matching.

5.2.1 Basic Comparison of Elastic and Inelastic Behavior

**Elastic Analyses:** Compared in Fig. 5.3 are normalized $K_I/\sigma_n$ results from elastic analyses of the T-stub and connection. Negative values on the horizontal axis correspond to cases where a notched backing bar is used and the initial crack length is less than the backing bar thickness. Thus, a “zero” crack length on the horizontal axis corresponds to cases where the crack length is equal to the backing bar thickness and the crack tip is even with the bottom surface of the beam. Three curves are plotted for the T-stub analyses, corresponding to cases where the centerlines of the flange and continuity plate are eccentric by $e = \pm 1/8$ inch, which is considered as a typical fabrication tolerance. Also plotted in Fig. 5.3 are values calculated using the following equations developed by Mohr (1999) as part of a SAC project on weld acceptance criteria:

$$K_I = \sigma \frac{F_t \sqrt{\pi a}}{\sqrt{(1 - a/t)^3}}; \quad F_t = 1.003 - 0.62(1 - \frac{a}{t}) - 0.0736 \left(1 - \frac{a}{t}\right)^{13.4}$$  \hspace{1cm} (5.1)

$$K_{I}' = K_{I(eq,5.1)} + \Delta F \sigma \sqrt{\pi d / 2}$$  \hspace{1cm} (5.2)

where $t$ is the plate thickness, $a$ the crack length, $d$ the crack width, and $\Delta F$ is a parameter given in Mohr’s report. The additional term in the second equation is based on the solution for an elliptical crack front where the point of maximum $K_I$ varies along the crack front.

The results in Fig. 5.3 clearly reveal significant differences in $K_I/\sigma_n$ between the T-stub and connection. For example for a small root crack of $a_o = 0$ to 0.1 inch, the stress intensity in the T-stub is only about half that of the beam-column connection. Both the T-stub and connection analyses reflect the potential reduction in toughness demand achieved using notched backing bars ($a_o < 0$), notwithstanding the large absolute differences between the two. Incidentally, while the analyses reveal potential advantages of the notched backing bars, the weldment tests have shown that the notched backing bars introduce fabrication problems related to fit-up tolerances and undercutting of the weld. Thus, their use is not recommended.

The T-stub (pull-plate) results for $e = \pm 1/8$ inch in Fig. 5.3 further show that the results are not sensitive to small eccentricities caused by misalignment of the plates. This indicates that slight fabrication misalignment should not significantly contribute to scatter of the test data compared, for example, to scatter introduced variations in weld quality. Finally, Fig. 5.3 indicates that the proposed Eq. 5.1 by Mohr captures the elastic T-stub toughness demands fairly well. The second
Eq. 5.2 does not match the finite element results, but this likely due to differences in the underlying modeling assumptions, e.g., the assumption of a through thickness crack in the two-dimensional finite element analyses versus an elliptical crack in Mohr’s formulation.

**Inelastic Analyses:** Compared in Fig. 5.4 are results from inelastic analyses of the T-stub and connection where CTOD demand is plotted versus applied stress in Fig. 5.4b and versus strain in Fig. 5.4c. Average strain in the pull plate and connection is measured over a 6-inch gage length of the beam flange, beginning from the root of the flange groove weld. In Figs. 5.4a and 5.4c, average strains are related to the inelastic connection rotation for $\theta_{\text{inelastic}} = 0.01, 0.02$ and 0.03 radians. As with the elastic analyses there are clear differences between the T-stub and connection behavior, where at strains corresponding to $\theta_{\text{inelastic}} = 0.03$ radian the CTOD for the connection is about twice that in the T-stub specimen. Even larger differences exist between CTOD measured at constant stress levels (Fig. 5.4b).

5.2.2 Sensitivity of T-stub to Weld Strength, Residual Stresses, and Fillet Reinforcement

Aside from the absolute differences between the T-stub and connection results, the relative influence of design and detailing parameters, such as the weld matching ratio, residual stresses, and the use of fillet reinforcement, are also different. For example, shown in Fig. 5.5 are results for cases with matching welds where both the base and weld metals have a yield strength of $F_y=65$ ksi. Comparing the relative difference in Fig. 5.5 (matching welds) to Fig. 5.4 (overmatching welds), the connection is much more sensitive to the reduction from overmatching to matching. Whereas there is about a factor of two difference in CTOD between the connection and T-stub for the overmatching case, for the matching case the difference is about a factor of three, i.e., CTOD for the connection is about three-times that for the T-stub.

On the other hand, referring to Fig. 5.6, the T-stub seems to be more sensitive to residual stresses than the connection. As shown in the top plot of Fig. 5.6, the residual stresses impose an initial CTOD = 0.00015 inch, which is about 50% larger than CTOD = 0.0001 inch for the connections (Fig.4.22). At larger inelastic deformations (lower plot of Fig. 5.6), the increase of 0.001 inch in CTOD caused by residual stresses is also about 50% larger than in the connections (Fig. 4.21).

Finally, as shown in Fig. 5.7, reductions in CTOD provided by the fillet reinforcement will also vary between the T-stub and connection. Here we see that the absolute reduction in $K/\sigma_n$ for the T-stub is on the order of one fourth for the connection, although on a relative basis, the percent reductions for each case are not too different.

5.2.3 Modification of T-stub Specimen

One of the questions motivated by differences between the T-stub and connection behavior is whether the T-stub specimen can be modified to improve correlation between the two. For example, one simple modification would be to attach eccentric pull-plates so as to induce bending stresses in the weld. Using attachments, such as those shown in Fig. 5.8, we found that while it is possible to modify the behavior, none of the techniques we tried were able to achieve consistent agreement between the T-stub and connection over the full range of loading. As shown in Fig. 5.9, by attaching eccentric pull-plates, the response curves relating $K/\sigma_n$ and crack
length can be shifted, but the shape of the curves are still different. More importantly, when yielding sets in, the eccentricity is lost as the T-stub tends to straighten out. The result, as shown in Fig. 5.10, is that while the CTOD response curves for specimens with attachments begin differently, at some load level all of the T-stub curves converge back to the original case without attachments.

Beyond looking at simply attaching eccentric loading plates to the standard T-stub specimen, we also investigated loading the T-stub specimen in bending (Fig. 5.11b) and using an entirely different bending specimen (Fig. 5.11c). Unfortunately, as shown in Fig. 5.12, none of these yielded the desired objective to match the connection response over the full range of loading.

5.2.4 Interpretation and Design Implications of T-stub Tests

While fracture behavior ($K_I$ and CTOD demand) in the T-stub tests does not match that in connections, the T-stub tests do provide data on the weldment performance that is relevant to the connection. However, relating the T-stub to connection performance requires analyses. For example, whether or not a weld fractures in a T-stub test does not necessarily mean the same will hold true in the connections. But, coupled with fracture analyses, such as those summarized above, data from the T-stub tests can help establish the insitu toughness characteristics of welds and the surrounding Heat Affected Zone. Moreover, while standard ASTM fracture tests (e.g., three point bending or compact tension CTOD tests) would provide a more precise measure of the material properties, the T-stubs are in fact more representative of actual fracture toughness in the weldment for the types and locations of flaws that are characteristic of those in the connections.

Comparison of Analysis Results and Test Data: Ricles et al. (1999) summarized test results of fifteen pull-plate specimens, of which ten can be related to the analyses described above. Five of the specimens were fabricated with E70T-4 electrodes and five with E70TG-K2 electrodes.

The five E70T-4 specimens (specimens 2-1 to 2-3 and 4c-5 and 4c-6) all fractured through the weld-to-column flange fusion zone at applied pull-plate stresses of $\sigma_u = 38$ to $55$ ksi. Three of these were fabricated with grooved backing bars, although weld root flaws on the order of $a_o = 0.1$ to 0.2 inches (discovered after the specimens fractured) indicate that the grooved bars were not very effective in reducing built-in weld defects. Moreover, one problem that arises with the grooved backing bars is that they require a rather narrow weld root gap (0.2 inch) that can create problems with achieving good weld fusion. The other two E70T-4 specimens had regular backing bars with an E7018 fillet added to seal the backing bar gap. From our previous analyses of the connections and pull-plates, one would expect the seal welds to significantly improve the behavior, but this was not the case. The reason for this has to do with the fact that the backing bars remained attached to the column when the weld fractured (rather than being pulled off with the pull plate). Accordingly, the backing bars and the seal welds were not engaged by the pull-plate, and the seal weld was ineffective at reducing toughness demand at the top of the backing bar/weld root defect. This ineffectiveness appears to be due to the fact that the weld root gap was too narrow to achieve fusion between the backing bar and weld metal.
Two of the E70TG-K2 specimens (4c-1 and 4c-2) failed in the weld at pull-plate stresses of $\sigma_u = 71$ to 76 ksi and three (4c-3, 4c-4, and 6-1) failed by rupture of the pull plate at $\sigma_u = 82$ ksi. Both specimens that failed in the weld had grooved backing bar details with intentional misalignment of the backing bar grooves. The specimens that failed in the plate had either properly aligned grooved backing bars or standard backing bars. Thus, the difference in toughness between E70T-4 and E70TG-K2 welds is clearly enough to alter the performance of the T-stubs.

Assuming that these pull-plate tests can be reasonably represented by analyses with a through flange edge defect of $a_o = 0.1$ inch, material toughness data can be back calculated from applied stresses at failure using the elastic analysis results in Fig. 5.3 or the inelastic results (with and without residual stresses) in Fig. 5.6. Using for the elastic analyses, $K_I/\sigma_n = 0.65 \sqrt{\text{in}}$, and the range of failure stresses noted above, back-calculated values of toughness range from $K_{Ic} = 25$ to 36 ksi$\sqrt{\text{in}}$ for E70T-4 weld and $K_{Ic} = 46$ to 53 ksi$\sqrt{\text{in}}$ for the E70TG-K2 weld. These are both significantly lower than the values of $K_{Ic} = 76$ ksi$\sqrt{\text{in}}$ for E70T-4 and $K_{Ic} = 172$ ksi$\sqrt{\text{in}}$ for E70TG-K2 presented in Table 2.3. These large differences suggest that the elastic analyses without residual stresses do not model the true behavior very well.

Using inelastic analysis results from Fig. 5.6, the measured failure stresses correspond to the following values of implied toughness:

- E70T-4 without residual stresses; CTODc = 0.0002 to 0.0005 inch
- E70T-4 with residual stresses; CTODc = **0.0009 to 0.0014 inch**
- E70TG-K2 without residual stresses; CTODc = 0.0019 to > 0.005 inch
- E70TG-K2 with residual stresses; CTODc = **0.0036 to > 0.008 inch**

The large differences in these back-calculated toughness values reflects the variability inherent in response due to the residual stresses, the initial flaw sizes, and the weld metal toughness. Note that the higher end of the range for E70TG-K2 welds are lower bound values since these specimens failed outside of the weld in the flange plate. Results for the analyses with residual stresses are fairly consistent with the toughness data of Table 2.3. For example, from Table 2.3, CTODc = 0.0012 to 0.0018 inch for E70T-4 welds compared to the back-calculated range of CTODc = 0.0009 to 0.0014 inch, and CTODc = 0.0082 inch for E70TG-K2 versus the range of CTODc = 0.0036 to 0.008 inch given above. Keep in mind that there are many uncertainties inherent in these comparisons, related to both the analysis assumptions and the estimated toughness data in Table 2.3. Given these uncertainties, the overall agreement is good between the T-stub failure stresses, the analytical predictions, and the estimated weld toughness.

5.3 SAC Task 5.12 Through-Thickness Pull-Plate Tests

5.3.1 Overview of Test Data: The pull-plate tests conducted by Dexter and Melendrez (1999) were designed to investigate the potential for “through-thickness” failure of the column flange where it is subjected to large transverse tensile stresses directly behind the groove weld. This through-thickness investigation was motivated by the prevalence of “divot” type fractures observed in welded moment connections, coupled with concerns that the lower strength and
toughness in the through-thickness direction of rolled steel was a contributing factor to the fractures.

Dexter and Melendrez conducted forty tests of tee-shaped specimens, typically consisting of W14 stub column sections with opposing pull-plates welded to each face (Fig. 5.13). The welds and pull plates were made of high strength steel with the intent to force failures into the W14 column material. Most of the column material was A572 Grade 50 steel (although a few specimens had A913 Grade 65 material), and the pull plates were of high strength steel with measured yield and ultimate strengths of \( F_y = 104 \text{ ksi (719 MPa) and } F_u = 113 \text{ ksi (783 MPa)}, \) respectively. The pull plates were 1 inch (25 mm) thick and varied in width from 4 to 12 inches (100 to 305 mm). The plates were attached with double-sided groove welds made with Lincoln Electric ER100S-G electrodes with measured yield and ultimate strengths of \( F_y = 102 \text{ ksi (704 MPa) and } F_u = 109 \text{ ksi (752 MPa)}. \) Welds were generally of high quality with heat inputs controlled to be less than 25 kJ/cm. However, there were some exceptions to these general welding parameters, including a few specimens where either (1) welds were made with an E70T-4 root pass, (2) backing bars were left in place, or (3) heat input was intentionally increased to 35 kJ/cm. The high heat input was used to deteriorate the HAZ properties of the column base metal.

Of the 40 tests, 33 failed by tensile rupture of the 100 ksi flange plates, and only one of the seven specimens that failed in the welded joint region did so at an applied axial stress less than the nominal yield strength (100 ksi) of the pull-plates. This was a specimen with an E70T-4 weld root pass and backing bars left in place that failed at an applied pull plate stress of \( \sigma = 95 \text{ ksi (654 MPa)}. \)

Although the average stress in the pull-plates themselves typically exceeded 100 ksi at failure, the corresponding stress imposed on at the weld-to-column interface was often less than 100 ksi due to the beveled geometry of the weld that increased the cross sectional area at the weld-column interface. For example, in tests with Gr. 50 column material that failed by yielding and rupture of the pull-plates, the highest tensile stress reached at the weld-to-column interface was \( \sigma_{\text{max}} = 81 \text{ ksi (556 MPa)} \) and the average from all tests was \( \sigma_{\text{avg}} = 58 \text{ ksi (403 MPa)}. \) In tests with Gr. 65 column material, the weld build-up was removed by grinding, and the maximum tensile stress applied at the weld-to-column interface was \( \sigma_{\text{max}} = 118 \text{ ksi (813 MPa)} \) with the average of all tests equal to \( \sigma_{\text{avg}} = 107 \text{ ksi (737 MPa)}. \) In general, these results do suggest that through-thickness failure of the column flange would not be critical for connections with beams of Gr. 50 material where the maximum longitudinal stress in the beam flange would be on the order of \( \sigma_u = 70 \text{ to } 80 \text{ ksi (480 to 550 MPa)}. \) However, because the stress/strain state in the beam-column connections is more nonlinear that that in the pull-plates, questions remain as to whether the pull-plate results are sufficient to rule out the possibility of through-thickness failures in the beam-column connections. In other words, how much more severe are conditions in the beam-column connection compared to the pull-plates. To answer this one would like to have a model to assess the limiting stress/strain state that can be resisted at the weld-to-column interface.

Of the seven tests that failed in the weld/column flange region, only one exhibited a failure that resembled the divot fractures previously observed in connections. This was specimen #33 with a 12-inch (305 mm) wide pull plate without continuity (stiffener) plates and with the weld build-up (reinforcing) ground away. The tensile stress applied to the weld-column interface at failure was
\[ \sigma_{\text{max}} = \frac{P}{A} = 92 \text{ ksi (635 MPa)}, \] which was the largest average transverse stress resisted by any of the specimens with Gr. 50 column material. Since the specimen did not have continuity plates it is likely that the maximum transverse stress at the mid-plane of the pull-plate was, in fact, larger than the 92 ksi average stress. Therefore, of all forty pull-plate tests, this one (#33) provides the best measure of the likely fracture strength of the Gr. 50 column material behind the groove weld.

**5.3.2 SMCS Analyses of Pull-Plate:** Shown in Fig. 5.14a is the two-dimensional finite element model that we developed to analyze the pull-plate specimen using the SMCS ductile crack initiation model described in Section 2.8. Relying on symmetry conditions, only one-quarter of the specimen is modeled. As shown in the expanded view of Fig. 5.14a, the corner at the weld-to-column flange interface is modeled with a small \( r = 0.04 \text{ inch} \) radius to simulate a small but finite fillet that is assumed remain after the weld build-up is ground away. In the region of high stresses/strains in the column flange, the mesh is refined to have elements on the order of 0.0035 inch in size – roughly equal to the assumed characteristic length \( l^* \) by which the SMCS criterion is evaluated (see Section 2.8). This mesh would not be fine enough to accurately capture large stress/strain gradients ahead of a sharp crack, but they are considered to be reasonable in this case given that the stress/strain gradients are less severe. Moreover, since our main interest is in comparing conditions in the pull-plate and beam-column connection, we emphasized developing consistent models for the pull-plate and beam-columns.

Contours of the calculated SMCS criterion, based on the assumed SMCS parameter of \( \alpha = 1 \) (Section 2.8), are shown in Fig. 5.14b. The small red (dark gray in B&W) region just inside the fillet corresponds a region where SMCS \( \geq 0 \), i.e., where the effective plastic strain exceeds the strain capacity. Given that this region of SMCS \( \geq 0 \) region is about equal to the size of one element or \( l^* \), the ductile fracture initiation criterion is met, subject to the assumption of \( \alpha = 1 \) and \( l^* = 0.0035 \text{ inch} \). These contours correspond to a nominal applied tensile stress in the pull-plate about \( \sigma = 115 \text{ ksi} \) which is larger than the measured stress of \( \sigma_u = 92 \text{ ksi} \) at failure in Dexter’s specimen #33. Differences in the predicted and measured failure load can be attributed to several factors and uncertainties in the model including that (a) the SMCS coefficients \( \alpha = 1 \) and \( l^* = 0.0035 \text{ inch} \) are only estimates of the through-thickness and HAZ properties of the steel, (b) the SMCS finite element analysis does not reflect small flaws or larger inclusions that may be present in the weld or base metal, and (c) the SMCS analysis neglects any stable crack growth that may occur after crack initiation. The first two of these factors (a and b) would help explain why the calculated failure stress is larger than the measured stress, whereas the third factor (c) would have the opposite effect. Although the agreement between the SMCS analysis and pull-plate failure stress is not as good as one would like, the behavior is reasonably consistent so as to substantiate using SMCS as a basis for comparing conditions in the pull-plate and connection specimens.

Contours of stress, plastic strain and triaxiality from which the SMCS are derived are shown in Figs. 5.15 and 5.16. As seen Fig. 5.15c and 5.16b, stress triaxiality is highest in the center of the pull-plate at the column interface and quickly recedes near the surface. The plastic strain contours (Fig. 5.15b and 5.16a) confirm that the middle of the pull-plate at the weld-to-column interface remains elastic and that there are large plastic strains (up to \( \varepsilon_p = 0.3 \)) at the surface of the weld fillet. Referring to Figs. 5.15a and b, the curved band of high von Mises stresses and
plastic strains that extends from the weld corner into the column is suggestive of the path that a fracture might follow once it initiates. These contours further reveal that the relatively high SMCS in the center of the plate (at the weld-to-column interface) is due to the large triaxiality that reduces the plastic strain capacity. However, when the pull-plate starts to plastify, the triaxiality in the center of the plate tends to remain constant while plastic strains increase rapidly and the critical value of SMCS > 0 is reached closer to the surface.

5.3.3 SMCS Analyses of Beam-Column Connection: To relate conditions in the pull-plate to those in the connection, we next ran two-dimensional finite element analyses modeled after connection specimens tests at the University of Michigan (Stojadinovic, et al. 1998) with a W14 x 257 column and W36 x 150 beam. Analyzed are cases where the weld backing bars are removed and the weld is built up on top and bottom as shown in Fig. 5.17. To investigate conditions with less weld build-up, we also analyzed a case with the weld geometry shown in Fig. 5.18 where the small fillet at the weld corner is equal to that used in the pull-plate analyses (Fig. 5.14a). The mesh is refined in the HAZ regions of the weld where the largest stresses/strains are expected. As in the pull-plate analyses, the elements in these regions are sized about equal to the characteristic length $l^* = 0.0035$ inch. Yield strengths of the beam and column are $F_{yb} = 55$ ksi (expected yield for Gr. 50) and for the weld is $F_{yw} = 65$ ksi (E70).

The resulting SMCS contours are shown in Figs. 5.19 and 5.20 for four levels of beam tip displacement (where 5 inch tip displacement roughly corresponds to 0.03 radians of inelastic connection rotation). Referring to Fig. 5.19, initially the largest SMCS occurs inside the weld at the weld/column flange interface, driven by large stress triaxiality at this location. But, as in the pull-plate tests, the SMCS at this location ceases to increase once the material begins to yield. As the beam yields, the largest increase in SMCS occurs in a slim band of HAZ material on the beam edge of the weld. This behavior is more apparent in Fig. 5.20 where the peak SMCS occurs at the underside of the weld at the beam-weld HAZ. As in the pull-plate specimen, the small red (dark grey in B & W) region at the critical location corresponds to the condition where SMCS $\geq 0$. In other words, the critical SMCS condition that was previously observed in the pull-plates (SMCS $\geq 0$ over $l^* = 0.0035$ inch) under the applied tensile stress of $\sigma = 115$ ksi is here reached in the connection at a beam tip displacement of 5 inches. Results for the detail without the weld build-up are shown in Fig. 5.21. Here conditions turn out to be more favorable than those in Fig. 5.20, and the critical condition of SMCS $\geq 0$ is not reached anywhere near the weld root, even up to beam tip displacements of 8 inches.

5.3.4 Relating Through-Thickness Pull-Plate and Connection Behavior: Overall, results of the connection and through-thickness pull-plate analyses indicate that ductile fracture initiation is not likely to occur in the column material before ductile fractures would initiate somewhere else. Comparisons of the pull-plate analyses to the pull-plate test #33 indicate that the SMCS model with $\alpha = 1$ and $l^* = 0.003$ is a representative lower bound of critical conditions for through-thickness fractures in the column flange. The SMCS analyses of the two connections with different weld details show that equivalent conditions (evaluated through the same SMCS model and coefficients) are not reached in the columns of the connections, even at large inelastic deformations. In the one case where a critical condition is reached (see Fig. 5.20), it occurs at the weld-to-beam HAZ. Thus, provided that the column material possesses similar properties to the columns tested in the pull-plate tests, then it seems unlikely that the column material would
fail in the through-thickness direction, except by a fracture propagation from a defect fracture. One should recognize, however, that the SMCS criteria and these analyses are dependent on the microstructure of the column material which can be affected by the chemistry (sulfur and other substances that create inclusions) and the grain sizes.
6. Summary and Conclusions

6.1 Summary

The overall objective of this investigation has been to provide quantitative data on the fracture behavior of welded beam-column connections to assist in the development of design and detailing requirements. The main portion of the investigation relies upon elastic and inelastic finite element analyses to evaluate conventional fracture indices ($J$, $K_I$, CTOD) at assumed flaw locations in and around the critical flange weld detail. Also included are inelastic continuum analyses that utilize a micro-mechanical criterion (SMCS) for ductile fracture initiation in regions without pre-defined flaws. Material stress-strain properties used in the finite element analyses are based on test data from related SAC projects to categorize base and weld metal properties. Charpy V-Notch (CVN) toughness data for weld and base metals are reviewed and converted to fracture indices ($K_{IC}$ and CTOD$_c$) to provide the basis for assessing implications of calculated fracture toughness demands on connection performance.

The investigation includes both parametric analyses of full-scale beam-column connections and analyses of related weldment tests. The connection analyses address the influence of the following parameters on $K_I$ and CTOD fracture toughness demands: beam and column geometry, flaw sizes and locations, role of the weld backing bar, relative yield strengths of weld and base metal, influence of joint panel zone strength, continuity plates, effectiveness of RBS detail, welding-induced residual stress effects, and weld access hole geometry. In conjunction with these analyses, simplified equations are developed to estimate fracture toughness demands for typical connection configurations. Comparative analyses of T-stub weldment specimens and through-thickness pull plate specimens serve to relate fracture behavior from tests of these components to that of the full-scale beam-column connections.

6.2 Conclusions and Observations

Detailed conclusions and observations from the various analyses are given in the individual sections of this report. The following is a summary of the more significant overall findings on fracture behavior that have implications on the connection design and performance:

**Weld Matching:** Inelastic fracture analyses show that weld overmatching is beneficial to reduce toughness demands for weld root flaws at the weld-to-column interface where shielding effects due to yield strength mismatch occur. Overmatching does not, however, offer much benefit to weld flaws at the weld-to-beam flange interface where shielding is not present and yielding concentrates in the HAZ. The expected (average) yield strengths of A572 Gr. 50 base metal, $F_{yb}$ = 55 ksi, and E70 weld metals, $F_{yw}$ = 65 ksi, provide a reasonable overmatching ratio of $F_{yw}/F_{yb}$ = 1.2. However, scatter inherent in the material data suggest that this overmatching may not
always be achieved. For example, test data for an E70T-4 weld indicated yield strength values of only $F_{yw} = 55$ ksi, which would result in no overmatching for A572 Gr. 50 base metal.

**Survey of Base Metal Toughness:** Reported CVN values for the rolled shapes used in the SAC connection tests ranged from CVN = 70 to 260 ft-lbs at 70°F with an average of about 200 ft-lbs. The average CVN = 200 ft-lbs translates to $K_Ic \approx 230$ ksi√in and CTOD$_c \approx 0.018$ inch. Being as these toughness data far exceed the minimum toughness criteria of CVN = 20 ft-lbs at 70°F, specified for based metals in the AISC Seismic Provisions, questions arise as to whether connections built with the minimum specified toughness would perform as well as those tested for the SAC Joint Venture. As summarized below, the fracture analyses provide a framework to estimate minimum toughness requirements.

**Survey of Weld Metal Toughness:** Reported toughness values (at 70°F) for typical flux core arc welds used in building construction range from a low of CVN = 10 ft-lbs for E70T-4 welds to a high of CVN = 70 to 80 ft-lbs for E70TG-K2 and E71T-8 welds. Using empirical correlation equations, these CVN data translate to the following general fracture toughness indices:

1. low toughness E70T-4: $K_Ic \approx 60$ to 75 ksi√in and CTOD$_c \approx 0.001$ to 0.002 inch,
2. moderate toughness E70T-6: $K_Ic = 130$ ksi√in and CTOD$_c \approx 0.005$ inch, and
3. high toughness E70T-8 and TG-K2: $K_Ic \approx 170$ ksi√in and CTOD$_c \approx 0.008$ inch.

Note that since the cases we are evaluating involve shallow edge cracks with low crack tip constraint, the apparent toughness of these materials in the connections may be up twice the cited values based on correlations developed for fracture toughness tests with high constraint. This is particularly the case for the higher toughness materials where significant yielding is expected prior to fracture. The implications of these toughness data are reviewed below.

**Backing Bar Effects:** The backing bar creates an inherent edge defect at the weld root that might otherwise be avoided if the backing bar is removed and the weld root repaired with a small fillet weld. Beyond this, the thickness and fusion length of the backing bar do not significantly affect toughness demand created at the tip of the backing bar gap. The fusion length does, however, influence the effectiveness of external seal welds, located below the backing gap, to reduce the toughness demand at the weld root (see next item).

**Fillet Reinforcement:** Where weld-backing bars are left in place, the addition of a fillet weld to seal the backing bar gap is shown to reduce the elastic toughness demand ($K_I$) to about one-third to one-half of the original values (without the seal weld). Reductions for inelastic toughness demand (CTOD) are even larger provided that the seal weld has the required strength and toughness. In our analyses, a 3/8 inch fillet seal weld with a ½ inch fusion length was shown to work well for a W36 x 150 beam. However, an important qualification to these observations is that the reductions can only occur where the weld fusion length is large enough to permit significant stress transfer into the backing bar. This may not be the case, for example, where there are tight fit-up conditions with little, if any, gap at the weld root. This condition appears to have been present in some of the T-stub tension tests where the beam flange pulled away from the column leaving the backing bar and seal weld attached to the column. In such cases the seal welds would not offer much, if any, reduction in toughness demand.
Flaw Location (Top versus Bottom): Flaws located on the inside face of the beam flange (the top of the bottom flange or the bottom of the top flange) are shown to have toughness demands less than one-quarter of those for bottom surface cracks. Coupled with the typical backing bar configuration, these observations help to explain the prevalence of bottom versus top flange fractures. They also provide a rationale for leaving the top flange backing bars in place while requiring removal of the bottom flange backing bars.

Flaw Location (Edge Cracks at Extreme Fiber): For edge cracks (surface flaws) located on the outside of the beam flange, the elastic toughness demand is relatively insensitive to flaw location (i.e., distance from the column flange face). In other words, $K_I$ for equal size weld root defects at the weld-to-column interface is similar to that for defects within the weld or at the weld-to-beam interface. However, this is not the case for inelastic behavior where the CTOD demand is significantly influenced by the interaction of material yielding throughout the connection and at the weld flaw location. In particular, CTOD demands for flaws located at the beam-to-weld interface are more than twice those at the weld-to-column interface, i.e., at the weld root. These differences suggest that stricter acceptance criteria should be applied to weld flaws at the beam-to-weld interface compared to the weld root, particularly as the former are easier to detect by inspection. Flaws at the weld-to-beam flange interface correspond to the weld toe which is susceptible to shrinkage cracking.

Residual Stress Effects: Welding-induced residual stresses are shown to apply an initial toughness demand on the order of $K_I = 20 \text{ ksi}\sqrt{\text{in}}$ or CTOD = 0.001 inch to small flaws at the weld root. These roughly correspond to the toughness demand imposed by a nominal bending stress of $\sigma_b = 10 \text{ ksi}$. The relative effect of the residual stresses reduces with increasing loads until at inelastic connection deformations of about $\theta_{inelastic} = 0.03 \text{ radians}$ the net effect is only about a 10% increase in toughness demand. Thus, the effect of residual stresses is most significant when evaluating pre-Northridge type connections with lower toughness materials that may fracture below yield at low inelastic rotations.

Continuity Plates: The influence of continuity plates on toughness depends on the column flange thickness and to some extent on the panel zone flexibility. Continuity plates reduce CTOD demands as much as 60% for a W21 x 131 column ($t_{fc} = 0.96 \text{ inch}$) connected to a W36 x 150 beam. For a W14 x 257 column ($t_{fc} = 1.89 \text{ inch}$) the reduction was about 30%, and for a W14 x 550 column ($t_{fc} = 3.82 \text{ inch}$) the continuity plates had no effect. The continuity plates appear to have a larger effect on connections with relatively weak panel zones (e.g., $M_{nf}/M_{pb} \approx 1.0$) compared to connections with strong panel zones that remain elastic.

Column Flange Thickness: Aside from its influence on continuity plate effectiveness, the column flange thickness appears to directly affect CTOD demands for weld root cracks — presumably by changing the local stress and strain fields at the beam-to-column interface. For three connections with column flange thicknesses of $t_{fc} = 3.82, 1.89$ and 0.96 inch and otherwise similar characteristics (continuity plates, panel zone deformations, etc.), the corresponding ratios of peak CTOD demand were 1, 1.6, 4.2. These data indicate that fracture toughness demands are less in connections with heavy column sections compared to connections with deeper/lighter columns and thinner flanges. Thus, tests of connections with heavy columns do not provide a
conservative measure of performance (insofar as weld fracture is concerned) as compared to connections with lighter and thinner column flanges.

**Panel Zone Deformations:** The analyses consistently show that CTOD demands are smallest for connections with minimal inelastic panel zone deformations and strong joint panels. For example, CTOD demands in connections with strong panel zones ($M_{nj}/M_{pb} > 2$) are between 40% to 60% of those for connections with matching joint panel and beam hinge strengths ($M_{nj}/M_{pb} = 1$). This observation appears to be inconsistent with the prevailing view, supported to some extent by test data, that connections with balanced panel zone strengths deliver larger inelastic rotation capacity prior to fracture. Reasons for the discrepancy are not clear. There are obviously limitations to the fracture analyses, one of the primary ones being that cyclic loading effects are not simulated. Nevertheless, the discrepancy suggests a careful review of available test data regarding panel zone effects to better establish the extent to which panel zone deformations are a contributor to beam flange weld fractures.

**RBS Detail:** The RBS detail is shown to provide an effective means to limit high stress and strain demands at the critical lower flange weld. However, the effectiveness of the RBS detail depends to a large extent on limiting the panel zone deformations, based on the relative strength of the RBS hinge to that of the panel zone.

**Weld Access Hole Details:** Analyses of three weld access hole details and weld profiles do not provide clear evidence that the alternatives considered (Fig. 4.25a) alter the fracture behavior. While built-up welds do reduce fracture toughness demands at the column face, they create discontinuities that increase demands at the weld-to-beam interface. This study considered only standard access hole details and did not address the behavior of long or slotted access hole details.

**Column Flange Through-Thickness Fractures:** Comparative analyses of the through-thickness pull-plate tests (SAC Task 5.12) and the beam-column connections using the SMCS criterion indicate that the through-thickness stresses and strains generated in the pull-plate specimens exceed those in the connections. Therefore, insofar as the column material in the pull-plate tests reflects that used in building construction, through-thickness failure of column flanges can be ruled out as a likely failure mode.

**T-stub Weldment Tests:** Analyses of the SAC Task 7.05 weldment tests show that fracture toughness demands generated in these tests are generally less severe (compared on the basis of nominal applied stresses) than in beam-column connections. Moreover, the pull-plate tests are more sensitive to some effects (e.g., residual stresses) and less sensitive to others (e.g., weld matching). Therefore, while the T-stub tests are a convenient investigative and screening tool, they do not directly represent the behavior of beam-column connections. From a practical standpoint, the T-stub tests analyzed in this study help confirm the in situ fracture toughness (CTODc) for E70T-4 and E70TG-K2 weld metal. The T-stub tests also provide evidence that the effectiveness of fillet seal welds below the backing bar is dependent on the amount of fusion between the backing bar and weld.
6.3 Minimum Toughness Requirements

Based on the data presented in this report, estimates of the minimum required material toughness can be calculated for an assumed flaw and loading condition. For the purposes of establishing minimum toughness demands, we assume an edge defect of \( a_o = 0.1 \) inch to be the maximum flaw tolerance. Presumably the actual flaws would be smaller, but this is assumed as the largest flaw that might go undetected in field-welded fabrication. The connection performance criteria are stated in terms of a target stress level (elastic analysis) or hinge rotation (inelastic analysis). As summarized below, toughness demands for inelastic behavior are distinguished by flaw location and the contributing mechanisms to inelastic connection rotation, i.e., panel zone deformation versus beam or RBS hinging.

**Elastic Toughness Demand:** Toughness demands from elastic analyses range from \( K/I/s_n = 2 \) to \( 3\sqrt{\text{in}} \) for 0.1 inch edge defects where \( s_n \) is the nominal applied bending stress. Within this range, the toughness demand is most affected by the column geometry – particularly the panel zone stiffness and column flange thickness. Toughness demands for comparable size interior cracks are about one-third those of exterior cracks. Assuming \( K/I/s_n = 2 \) to \( 3\sqrt{\text{in}} \) then to resist an applied beam stress of \( s_n = 1.3 F_y = 65 \text{ ksi} \) would require a minimum material toughness of \( K_{IC} = 130 \text{ to } 195 \text{ ksi}\sqrt{\text{in}} \). The demand would, of course, be higher for larger flaws and smaller for internal flaws.

Coincidentally, the \( K_{IC} = 130 \text{ to } 195 \text{ ksi}\sqrt{\text{in}} \) demand roughly equals the fracture toughness of moderate to high toughness weld metals, i.e., E70T-6 with \( K_{IC} \approx 130 \text{ ksi}\sqrt{\text{in}} \) up to E70T-8 and TG-K2 with \( K_{IC} \approx 170 \text{ ksi}\sqrt{\text{in}} \). Recall that for shallow cracks in yielded material, the apparent toughness of these weld metals is probably larger than these measured values (due to relaxed crack front constraint). Thus, these results indicate that for edge flaws of \( a_o < 0.1 \) inch, beam flexural stresses of \( s_n = 1.3 F_y = 65 \text{ ksi} \) can be achieved reliably using notch tough weld metals or base metals with room temperature toughness of CVN \( d \geq 40 \) to 50 ft-lbs. Alternatively, the results indicate that it would be unlikely to achieve reliably this stress level using E70T-4 weld metal with \( K_{IC} \approx 60 \) to 75 ksi\sqrt{\text{in}}.

**Inelastic Toughness Demand:** Toughness demands from inelastic analyses vary depending on flaw location, the locations of inelastic action, material yield strengths, and various geometric and detailing parameters. For a typical connection with Grade 50 base metal, a slightly overmatching weld (\( F_{yw}/F_{yb} = 1.2 \)), a balanced panel zone strength (\( M_{nj}/M_{pb} = 1.0 \)), and continuity plates, the toughness demand for a 0.1 inch edge crack at the weld root is about CTOD = 0.011 inch at \( \theta_{inelastic} = 0.03 \) radians. The demand reduces to about half this amount (CTOD = 0.005 inch) for a connection with a stronger panel zone (\( M_{nj}/M_{pb} = 1.9 \)) or where the beam strength is reduced using an RBS detail (\( M_{nj}/M_{p,rbs} = 1.5 \)). Using the RBS detail in conjunction with a stronger panel zone (\( M_{nj}/M_{p,rbs} = 2.2 \)) the demand reduces further to CTOD < 0.003 inch. Alternatively, for the typical connection with a 0.1 inch edge crack at the weld-to-beam flange interface the toughness demand is almost double the amount at the weld root, i.e., about CTOD = 0.020 inch. This demand reduces in about the same proportions as demands at the weld root by employing a stronger joint panel and/or an RBS detail.
For the most severe case, a flaw at the weld-to-beam flange interface in connections with significant panel zone deformations, the toughness demand of $\text{CTOD} = 0.020$ inch is more than twice the highest weld metal toughness of $\text{CTOD}_c = 0.008$ inch (for E70T-8 and TG-K2), indicating that the connection would not reach $\theta_{\text{inelastic}} = 0.03$ radians. The SAC tests conducted at the University of Michigan confirmed this behavior where the connections failed by extreme ductile tearing initiated by ductile fractures at the weld-to-beam interface at inelastic rotations on the order of $\theta_{\text{inelastic}} = 0.01$ to 0.02 radians.

On the other hand, by employing an RBS detail with a strong panel zone, the toughness demand at the weld can be reliably controlled to $\text{CTOD} = 0.003$ inch for a weld root defect and about double this value ($\text{CTOD} = 0.006$ inch) for flaws at the weld-to-beam interface. These are within the range of the moderate to high toughness weld metals, i.e., E70T-6 with $\text{CTOD}_c \approx 0.005$ inch and E70T-8 and TG-K2 with $\text{CTOD}_c \approx 0.008$ inch. Note that since the demand is significantly higher at the weld-to-beam interface, larger flaws can probably be tolerated at the weld root location. This observation helps justify the practice of leaving weld backing bars in place, particularly for the top flange weld. It also indicates the need for stricter control of flaws at the weld-to-beam interface, e.g., at the weld toe of top flange joint.

**General Observations:** The above scenarios indicate that with notch tough materials operating close to the upper shelf with $\text{CVN}_d \geq 40$ to 50 ft-lbs (at 70°F), most connections should have sufficient fracture resistance to reach the full plastic moment strength of the beam. This is provided that welds are of high quality and excessive panel zone deformations are avoided.

To reliably achieve large inelastic rotations requires further measures such as the RBS configuration to limit stress and strain demands on the fracture-critical welded details. However, the RBS alone does not guarantee satisfactory performance. In conjunction with the RBS detail, reliable control of toughness demands requires a sufficiently strong panel zone, quality welding, and continuity plates. The analyses reported herein indicate that the ratio of joint panel to the RBS hinge strength should be maintained above $\frac{M_n}{M_{p,rbs}} > 1.5$. The analyses also suggest that it may be permissible to omit continuity plates when the column flange is sufficiently thick (e.g., a W14 x 550 with $t_f = 3.82$ inch).
References


AISC (1995), Statistical Analysis of Charpy V-Notch Toughness for Steel Wide Flange Structural Shapes, American Institute of Steel Construction, Chicago, IL


El-Tawil et al. (1998), Strength and Ductility of FR Welded-Bolted Connections,” SAC/BD-98/01, ATC, Redwood City, CA.


Kaufmann, E.J. and Fisher, J.W., (1997), Failure Analysis of Welded Steel Moment Frames Damaged in Northridge Earthquake, NISTIR 5944, National Institute of Standards and Technology, Gaithersburg, MD, pp. 166


TABLES
Table 2.1 Stress-Strain Properties for A572 Gr 50 Flanges of Rolled W-shapes (Jaquess et al. 1999)

<table>
<thead>
<tr>
<th>Property</th>
<th>Mean</th>
<th>Maximum</th>
<th>Minimum</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_y/F_{yn}^1$</td>
<td>1.13</td>
<td>1.29</td>
<td>0.96</td>
<td>8%</td>
</tr>
<tr>
<td>$F_y/F_{yn}$</td>
<td>1.09</td>
<td>1.28</td>
<td>0.96</td>
<td>7%</td>
</tr>
<tr>
<td>$F_{sy}/F_{yn}$</td>
<td>1.04</td>
<td>1.24</td>
<td>0.91</td>
<td>8%</td>
</tr>
<tr>
<td>$\varepsilon_{sy}/\varepsilon_{yn}$</td>
<td>8.73</td>
<td>14.0</td>
<td>4.29</td>
<td>33%</td>
</tr>
<tr>
<td>$E_{sh}/E$</td>
<td>0.0131</td>
<td>0.0165</td>
<td>0.0075</td>
<td>18%</td>
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<tr>
<td>$\varepsilon_{u}/\varepsilon_{yn}$</td>
<td>86</td>
<td>117</td>
<td>68</td>
<td>13%</td>
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<tr>
<td>$F_u/F_{yn}$</td>
<td>1.45</td>
<td>1.55</td>
<td>1.32</td>
<td>4%</td>
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</table>

Notes:
1) $F_{yn} = 50$ ksi is the nominal yield strength of ASTM A572 Grade 50
2) $\varepsilon_{yn} = F_{yn}/E = 50/29,000 = 1.724 \times 10^{-3}$
3) $E (=29,000$ ksi) is Young’s modulus of elasticity

Table 2.2 Comparison of Stress-Strain Properties from Strap and Round Bar Coupons

<table>
<thead>
<tr>
<th>Coupon</th>
<th>$F_{sy}$ (ksi)</th>
<th>$F_y$ (ksi)</th>
<th>$F_{sy}$ (ksi)</th>
<th>$\varepsilon_{sh}$ (in/in)</th>
<th>$E_{sh}$ (ksi)</th>
<th>$F_u$ (ksi)</th>
<th>$\varepsilon_{u}$ (in/in)</th>
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</thead>
<tbody>
<tr>
<td>Strap</td>
<td>54.6</td>
<td>52.0</td>
<td>49.5</td>
<td>0.0142</td>
<td>387</td>
<td>70.6</td>
<td>0.158</td>
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<tr>
<td>½&quot; Round</td>
<td>58.7</td>
<td>53.0</td>
<td>50.1</td>
<td>0.0127</td>
<td>451</td>
<td>72.5</td>
<td>0.161</td>
</tr>
<tr>
<td>Difference</td>
<td>8 %</td>
<td>2 %</td>
<td>1 %</td>
<td>-11 %</td>
<td>17 %</td>
<td>3 %</td>
<td>2 %</td>
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</table>

Table 2.3 Fracture Toughness of Base and Weld Metals

<table>
<thead>
<tr>
<th>Material</th>
<th>$CVN_{dynamic}$ (ft-lb)</th>
<th>$CVN_{static}$ (ft-lb)</th>
<th>$K_{IC}$ (ksi√in)</th>
<th>CTOD $C$ (x10^{-3}$ inch)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base Metal</td>
<td>196</td>
<td>196</td>
<td>230 (u)</td>
<td>17.8</td>
</tr>
<tr>
<td>E70T-4</td>
<td>10</td>
<td>40</td>
<td>76 (t)</td>
<td>1.8</td>
</tr>
<tr>
<td>E70T-6</td>
<td>37</td>
<td>50</td>
<td>130 (u)</td>
<td>4.6</td>
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<tr>
<td>E71T-8</td>
<td>80</td>
<td>110</td>
<td>190 (u)</td>
<td>10.1</td>
</tr>
<tr>
<td>E70TG-K2</td>
<td>72</td>
<td>90</td>
<td>170 (u)</td>
<td>8.2</td>
</tr>
<tr>
<td>E7018</td>
<td>136</td>
<td>145</td>
<td>220 (u)</td>
<td>14.2</td>
</tr>
</tbody>
</table>

Notes:
1) Conversions from $CVN_{static}$ to $K_{IC}$ and CTOD $C$ are based on Eq. 2.2 to 2.4 and the average yield and ultimate strength properties reported in Sections 2.2 and 2.3.
2) Data for E70T-4 are larger than previous ranges of $CVN_s < 20$ ft-lb and $K_{IC} = 40$ to 60 ksi√in by Kaufmann et al. (1995) and corresponding CTOD $C = 0.5$ to $1.2 \times 10^{-3}$ inch by Chi et al. (1997).
3) $K_{IC}$ values are calculated from $CVN_{static}$ using either Eq. 2.2 for transition region (t) behavior or Eq. 2.3 for upper-shelf (u) behavior. The distinction between the behavior mode is based on the static $CVN$ curves shown in Fig. 2.10.
Table 3.1 Variation of Beam and Column Dimensions (inches)

<table>
<thead>
<tr>
<th>Column Geometry</th>
<th>Beam Geometry</th>
</tr>
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<tbody>
<tr>
<td>Column</td>
<td>Beam</td>
</tr>
<tr>
<td>$t_c$</td>
<td>$d_c$</td>
</tr>
<tr>
<td>Lower Bound</td>
<td>1.1</td>
</tr>
<tr>
<td>W14×257</td>
<td>1.89</td>
</tr>
<tr>
<td>Upper Bound</td>
<td>2.7</td>
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</tbody>
</table>

Table 4.1 Summary of Material and Component Strengths of UCB-L, UCB-H and Generic Connection

<table>
<thead>
<tr>
<th>Name</th>
<th>Yield Strengths (ksi)</th>
<th>Beam (k-in)</th>
<th>Joint Panel (k-in)</th>
<th>Joint/Beam Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$bm(f)$</td>
<td>$bm(w)$</td>
<td>$col.$</td>
<td>$weld$</td>
</tr>
<tr>
<td>UCB-L</td>
<td>40</td>
<td>50</td>
<td>48</td>
<td>60</td>
</tr>
<tr>
<td>UCB-H</td>
<td>61</td>
<td>60</td>
<td>48</td>
<td>60</td>
</tr>
<tr>
<td>GENERIC</td>
<td>55</td>
<td>55</td>
<td>55</td>
<td>65</td>
</tr>
</tbody>
</table>

Notes:
1) Beam yield moment $M_{y,b}$ is based on the beam flange (f) yield strength; the plastic moment $M_{p,b}$ is calculated using the beam web (w) and flange (f) yield strengths.
2) Joint panel yield strength $M_{y,j}$ is determined by the shear yielding equation $V_y = 0.55F_{yc} d_c t_{wc}$ and the nominal strength $M_{n,j}$ is determined as $V_n = V_y [1+ (3 b_{fc} t_{fc}^2)/(d_b d_c t_{wc})]$ where $F_{yc}$ is the yield strength of the column; $d_c$, $b_{fc}$, $t_{wc}$ and $t_{fc}$ are the depth, flange width, web thickness, and flange thickness of the column, respectively; and $d_b$ is the beam depth.
3) Governing connection strengths (either $M_{p,b}$ or $M_{n,j}$) are shown in bold.
Table 4.2 Connection Geometries and Properties\(^1\) for Parametric CTOD Study for Flaws at Weld to Beam Flange

<table>
<thead>
<tr>
<th>Case</th>
<th>Beam</th>
<th>Column</th>
<th>Joint Panel DP</th>
<th>Beam (k-in)</th>
<th>Joint Panel(^2) (k-in)</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1</td>
<td>W36x150</td>
<td>W24x257</td>
<td>0</td>
<td>27,720</td>
<td>31,960</td>
<td>26,640</td>
</tr>
<tr>
<td>1-2</td>
<td>W36x150</td>
<td>W24x257</td>
<td>0.5(t_{wc})</td>
<td>27,720</td>
<td>31,960</td>
<td>39,960</td>
</tr>
<tr>
<td>1-3</td>
<td>W24x257</td>
<td></td>
<td>(t_{wc})</td>
<td>27,720</td>
<td>31,960</td>
<td>53,280</td>
</tr>
<tr>
<td>2-1</td>
<td>W30x99</td>
<td>W24x257</td>
<td>0</td>
<td>14,980</td>
<td>17,160</td>
<td>20,750</td>
</tr>
<tr>
<td>2-2</td>
<td>W30x99</td>
<td>W24x257</td>
<td>0.5(t_{wc})</td>
<td>14,980</td>
<td>17,160</td>
<td>31,120</td>
</tr>
<tr>
<td>2-3</td>
<td>W30x99</td>
<td>W24x257</td>
<td>(t_{wc})</td>
<td>14,980</td>
<td>17,160</td>
<td>41,500</td>
</tr>
<tr>
<td>3-1</td>
<td>W30x99</td>
<td>W14x176</td>
<td>0</td>
<td>14,980</td>
<td>17,160</td>
<td>13,620</td>
</tr>
<tr>
<td>3-2</td>
<td>W30x99</td>
<td>W14x176</td>
<td>0.5(t_{wc})</td>
<td>14,980</td>
<td>17,160</td>
<td>20,430</td>
</tr>
<tr>
<td>3-3</td>
<td>W30x99</td>
<td>W14x176</td>
<td>(t_{wc})</td>
<td>14,980</td>
<td>17,160</td>
<td>27,240</td>
</tr>
<tr>
<td>4-1</td>
<td>W30x99</td>
<td>W14x145</td>
<td>0</td>
<td>14,980</td>
<td>17,160</td>
<td>10,830</td>
</tr>
<tr>
<td>4-2</td>
<td>W30x99</td>
<td>W14x145</td>
<td>0.5(t_{wc})</td>
<td>14,980</td>
<td>17,160</td>
<td>16,250</td>
</tr>
<tr>
<td>4-3</td>
<td>W30x99</td>
<td>W14x145</td>
<td>(t_{wc})</td>
<td>14,980</td>
<td>17,160</td>
<td>21,670</td>
</tr>
<tr>
<td>5-1</td>
<td>W24x68</td>
<td>W14x120</td>
<td>0</td>
<td>8,470</td>
<td>9,735</td>
<td>6,980</td>
</tr>
<tr>
<td>5-2</td>
<td>W24x68</td>
<td>W14x120</td>
<td>0.5(t_{wc})</td>
<td>8,470</td>
<td>9,735</td>
<td>10,470</td>
</tr>
<tr>
<td>5-3</td>
<td>W24x68</td>
<td>W14x120</td>
<td>(t_{wc})</td>
<td>8,470</td>
<td>9,735</td>
<td>13,960</td>
</tr>
</tbody>
</table>

Notes
1) Based on generic stress-strain properties with \(F_{y,\text{base}} = 55\) ksi and \(F_{y,\text{weld}} = 65\) ksi.
2) Joint panel strengths calculated using same equations given with Table 4.1 and considering the added doubler plate (DP) as being fully effective.
Table 4.3 Connection Member Sizes and Properties for Continuity Plate/Panel Zone Study

<table>
<thead>
<tr>
<th>Case</th>
<th>Column</th>
<th>$t_{wc}$ (in)</th>
<th>$t_{fc}$ (in)</th>
<th>$b_{fc}$ (in)</th>
<th>$M_{n,jp}$</th>
<th>$M_{n,jp}/M_{pb}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>W14 x 257</td>
<td>1.175</td>
<td>1.89</td>
<td>16.0</td>
<td>33,300</td>
<td>1.04</td>
</tr>
<tr>
<td>2</td>
<td>W14 x 257 w/doublers</td>
<td>2.35</td>
<td>1.89</td>
<td>16.0</td>
<td>59,900</td>
<td>1.87</td>
</tr>
<tr>
<td>3</td>
<td>W14 x 550</td>
<td>2.38</td>
<td>3.82</td>
<td>1.72</td>
<td>&gt;59,000</td>
<td>&gt;1.87</td>
</tr>
<tr>
<td>4</td>
<td>W24 x 131 w/doubler</td>
<td>1.21</td>
<td>0.96</td>
<td>12.9</td>
<td>42,000</td>
<td>1.33</td>
</tr>
</tbody>
</table>

Table 4.4 Properties of Connections with Different Joint Panel and RBS Strengths

<table>
<thead>
<tr>
<th>Case</th>
<th>Total PZ Thickness</th>
<th>RBS</th>
<th>$M_{n,jp}$</th>
<th>$M_{p,b}$ or $M_{p,rbs}$</th>
<th>$M_{n,jp}/M_{p,rbs}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.0 $t_{wc}$</td>
<td>NA</td>
<td>33,300</td>
<td>32,000</td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>2.0 $t_{wc}$</td>
<td></td>
<td>59,900</td>
<td>32,000</td>
<td>1.9</td>
</tr>
<tr>
<td>3</td>
<td>1.0 $t_{wc}$</td>
<td>50%</td>
<td>33,300</td>
<td>24,600</td>
<td>1.4</td>
</tr>
<tr>
<td>4</td>
<td>2.0 $t_{wc}$</td>
<td>50%</td>
<td>59,900</td>
<td>24,600</td>
<td>2.4</td>
</tr>
<tr>
<td>5</td>
<td>1.5 $t_{wc}$</td>
<td>50%</td>
<td>46,600</td>
<td>20,700</td>
<td>1.9</td>
</tr>
<tr>
<td>6</td>
<td>0.6 $t_{wc}$</td>
<td>25%</td>
<td>20,700</td>
<td>31,200</td>
<td>1.1</td>
</tr>
<tr>
<td>7</td>
<td>1.0 $t_{wc}$</td>
<td></td>
<td>33,300</td>
<td>31,200</td>
<td>1.9</td>
</tr>
<tr>
<td>8</td>
<td>2.0 $t_{wc}$</td>
<td></td>
<td>59,900</td>
<td>31,200</td>
<td>1.9</td>
</tr>
</tbody>
</table>

Notes
1) Joint panel strength, $M_{n,jp}$, is calculated following the same equation under note 2 in Table 4.1, and reflects the maximum beam moment that can be resisted at the column face.
2) The maximum moment for RBS connections, $M_{p,rbs}$, is calculated as the maximum beam moment at the column face corresponding to the plastic moment strength at the RBS detail.
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FIGURES
Figure 1.1 - Typical Pre-Northridge Style Weld Detail
Figure 2.1 Schematic Stress-Strain Curve

Figure 2.2 Mean Stress-Strain Properties - A572 Gr. 50 (Frank et al. 1999)
Figure 2.3 Comparisons of Yield and Ultimate Strength of Mild Grade 50 Steel

(a) Yield Strength

(b) Ultimate Strength

Mean Yield Strength (54.5 ksi)
Std. Deviation (4 ksi)

Mean Ultimate Strength (72.5 ksi)
Std. Deviation (3 ksi)
Figure 2.4  Comparison of Yield Ratios of Mild Grade 50 Steel
Figure 2.5  Weld Metal Yield Strengths (Johnson 1999)
Mean Ultimate Strength (82.1 ksi)

Mean Ultimate Strength (84.5 ksi)

Mean Ultimate Strength (86.2 ksi)

Mean Ultimate Strength (80.1 ksi)

Mean Ultimate Strength (87.4 ksi)

Figure 2.6 Weld Metal Ultimate Strengths (Johnson 1999)
Figure 2.7 Average Charpy-V Notch Results of Mild Grade 50 Steel (Jaquess & Frank 1999)

Figure 2.8 Comparison of CVN Data from SAC Study and AISC Mill Survey
Figure 2.9  Weld Metal CVN Data
Figure 2.10  Weld Metal Dynamic and Static CVN Curves
Fig. 2.11 Finite Element Models of Connection (a) Two-Dimensional  (b) Three-Dimensional
Figure 2.12 Stress-Strain Curves of Base and Weld Metal

(a) Void Nucleation
(b) Void Growth and Strain Localization
(c) Necking between Voids
(d) Void Coalescence and Fracture

Figure 2.13 Void Nucleation, Growth, and Coalescence in Ductile Metals
Figure 3.1 Elastic Longitudinal Flange Stress at Weld Access Hole (2D analysis)

Figure 3.2 Variation in $K_I$ with Flaw and Backing Bar Size

Figure 3.3 Variation in $K_I$ with Backing Bar Width
Figure 3.4 Variation in $K_I$ with Locations

Figure 3.5 Effectiveness of Fillet Weld Reinforcement

Figure 3.6 Two- versus Three-Dimensional Elastic $K_I$ Response
Figure 3.7 Geometric Parameters and Flaw Locations

(a) Geometric Parameter

(b) Flaw Locations

Figure 3.8 Two- and Three-Dimensional Finite Element Models

(a) 2-D Edge Crack

(b) 2-D Interior Crack

(c) 3-D Edge Crack

Figure 3.8 Two- and Three-Dimensional Finite Element Models
Figure 3.9 Variation of Normalized $K_I$ as a Function of Beam Geometry

Figure 3.10 Variation of Normalized $K_I$ as Function of Beam Section Modulus and Flange Area Ratio
Figure 3.11 Variation of Normalized $K_i$ as Function of Column Geometry

Figure 3.12 Variation of Normalized $K_i$ as Function of Beam-to-Joint Panel Stiffness
Figure 3.13 Comparison of Normalized $K_I$ for a Fixed Beam Geometry

Figure 3.14 Comparison of Finite Element versus Estimated $K_I$
Figure 3.15  Three-Dimensional Effects of Beam Flange Width on Normalized $K_i$
Figure 3.16 Three-Dimensional Effects of Column and Beam Geometries on $K_1$
Figure 3.17 Comparison of Normalized $K_I$ for Interior Cracks with Varying Weld Fillets

Figure 3.18 Comparison of Finite Element and Estimated $K_I$ for Interior Crack
Figure 3.19  Welding Induced Residual Stresses

(a) Comparison of Transverse Weld Stresses

(b) Transverse Stress Contour, $\sigma_{xx}$

(c) Longitudinal Stress Contour, $\sigma_{zz}$
Figure 4.1 Global Behavior of Connections UCB-L and UCB-H
Figure 4.2 Definitions of Connection Rotation Measurements

\[ \theta_{\text{total}} = \theta_{\text{joint}} + \theta_{\text{beam}} \]
\[ \theta_{\text{joint}} = \theta_B - \theta_A \]
\[ \theta_{\text{beam}} = \theta_C - \theta_B \]

Inelastic Rotation
\[ \theta_{\text{inelastic}} = (\Delta_{\text{tip}} - \Delta_{\text{tip,e}}) / L_b \]

Figure 4.3 Inelastic Fracture Toughness Demands in UCB-L and UCB-H
(a) Two-Dimensional, (b) Two versus Three-Dimensional, (c) Elastic versus Inelastic, (d) Summary Comparison
Figure 4.4 Contours of Plastified Regions as a Function of Weld Matching Ratio
Figure 4.5 Comparison of Inelastic Fracture Demands: UCB-L, UCB-H, and Generic Connection
Figure 4.6 Comparison of Fracture Demands for Different Crack Locations
Figure 4.7  Weld Detail and Access Hole Geometry of the Michigan Test Specimens

(a) Weld and Access Hole Geometry

(b) 2D FEM of Weld Region

(c) 3D FEM Mesh
Figure 4.8 Comparison of CTOD versus Rotation Components

Figure 4.9 Schematic Description of CTOD versus Rotation Equation
Figure 4.10 Summary and Calibration of CTOD Response
Figure 4.11 Two- versus Three-Dimensional CTOD

Figure 4.12 2-D and 3-D CTOD Response versus Average Fiber Strain
Figure 4.13 Effect of Panel Zone Flexibility on CTOD Demand

Figure 4.14 Effect of Continuity Plates on CTOD Demand for W14x257 to W36x150 (a) Standard Case (b) with Web Doubler Plates
(a) with Continuity Plates                                       (b) without Continuity Plates

Figure 4.15  Effective Plastic Strain Contours (W14 x 257 Column & W 36 x 150 Beam)
Figure 4.16 Effective Plastic Strain Contours – Isometric View
Figure 4.17 Plastic Strain Contours: W14x257 to W36x150 with Doubler Plates
Figure 4.18 Plastic Strain Contours with Doubler Plates – Isometric View
(a) W14x550 Column, $t_{fc} = 3.82$ inch

(b) W14x257 Column w/DP, $t_{fc} = 1.89$ inch

(c) W21x131 Column, $t_{fc} = 0.96$ inch

Figure 4.19 Effect of Column Flange Thickness on CTOD Demand
Figure 4.20 Comparison of Column Flange Thickness versus CTOD

Figure 4.21 Effect of Residual Stresses on CTOD

(a) Bending Stress vs. CTOD  (b) CTOD vs. Inelastic Rotation
Figure 4.22 Effect of Weld Matching and Residual Stresses on CTOD

(a) without Residual Stress

(b) with Residual Stress
σ_y_weld = 85; σ_y_base = 55

σ_y_weld = 75; σ_y_base = 55

σ_y_weld = 65; σ_y_base = 55

σ_y_weld = 45; σ_y_base = 55

Figure 4.23 Difference in CTOD due to Weld Matching and Residual Stress
Figure 4.24 Three-Dimensional Model of Continuum Model of Connection
Effective Plastic Strain  
Maximum Principal Stress

Standard Cope

Shortened Cope

Shortened Cope with Weld Reinforcement

Figure 4.25(a) Cope Shape Comparison – Plastic Strain and Principal Stress at $\Delta_{\text{tip}} = 4$ inch
Figure 4.25(b) Cope Shape Comparison – SMCS and Triaxiality Contours, $\Delta_{\text{tip}} = 4$ in.
Figure 4.26 Cope Shape Comparison - Stress/Strain Line Plots at Column-weld Interface

Figure 4.27 Cope Shape Comparison - Stress/Strain Line Plots at Beam Flange to Weld Interface
Figure 4.28 Shortened Cope Shape with and without Built Up Weld - Stress/Strain Line Plots at Column-Weld Interface

Figure 4.29 Shortened Cope Shape with and without Built Up Weld - Stress/Strain Line Plots at Beam Flange to Weld Interface
Figure 4.30 Residual Stress Effects - Stress/Strain Line Plots at Column-weld Interface

Figure 4.31 Residual Stress Effects - Stress/Strain Line Plots at Beam Flange to Weld Interface

110
Figure 4.32(b) Shortened Copes w/ and w/o Doubler Plates - Stress/Strain Line Plots at Weld - Beam Flange Interface

Figure 4.33(a) Shortened Cope with Built Up Weld - Stress/Strain Line Plots at Column Flange - Weld Interface
Figure 4.33(b) Shortened Cope Shape w/Built Up Weld - Stress/Strain Line Plots at Beam Flange to Weld Interface
Figure 4.34 Geometry of RBS Radius Cut

Figure 4.35 Three-Dimensional Models for RBS Connections

Figure 4.36 Effective Plastic Strain Contours of (a) Standard and (b) RBS Connection
Figure 4.37 CTOD of Connections with Varying RBS and Joint Panel Strengths

Figure 4.38 Distribution of CTOD in Standard and RBS Connections
Figure 5.1 Pull-Plate Weldment Test Specimen

Figure 5.2 Finite Element Model of Pull-Plate Specimen
Figure 5.3 Elastic $K_I$ for Beam-Column Connection and Pull-Plate Specimens with Variable Crack Length, $a_0$
Figure 5.4 CTOD Response - Connection versus Pull Plate
(Weld: $\sigma_y=65, \sigma_u=80$ ; Base Metal: $\sigma_y=58, \sigma_u=82$)
Figure 5.5 CTOD Response - Connection versus Pull-Plate Specimens with Matching Welds
(Weld and Base Metal: $\sigma_y = 65$ ksi, $\sigma_u = 80$ ksi)
Figure 5.6 Effect of Residual Stresses on CTOD for Pull-Plate Specimen

Figure 5.7 Influence on Reinforcing Fillet Weld on $K_I$ and CTOD in Pull Plate and Connection
Figure 5.8 Deformed Configurations of Pull-Plate Specimens with Attachments

(a) Pull-Plate Specimen with Upper Attachments

(b) Pull-Plate Specimen with Lower Attachments

Figure 5.9 Comparison of Normalized $K_I$ for Pull-Plate Specimens with Eccentric Attachments
Figure 5.10 Inelastic CTOD Behavior of Pull-Plates with Attachments
(Weld $F_y = 60$ ksi, Beam Flange $F_y = 40$ ksi, Column $F_y = 58$ ksi)
Figure 5.11 Alternative Weldment Specimens

(a) Standard T-Stub

(b) T-Stub Loaded in Bending

(c) Bending Specimen
Figure 5.12 Comparison of Different Weldment Specimens
(Weld $\sigma_y=65, \sigma_u=80$; Base $\sigma_y=58, \sigma_u=82$)
Figure 5.13 Through Thickness Pull-plate Specimen

(a) Finite Element Model

(b) SMCS Contour at Ductile Fracture Initiation

Figure 5.14 Through-Thickness Pull-Plate Model and SMCS Contour
Figure 5.15 Stress/Strain Contours at Predicted Crack Initiation

- (a) Mises Stress
- (b) Effective Plastic Strain
- (c) Triaxiality
- (d) SMCS

Figure 5.16 Close-up View of Stress/Strain Contours

- (a) Effective Plastic Strain
- (b) Triaxiality
- (c) SMCS
Figure 5.17 Model of Built-up Weld Detail

Figure 5.18 Model of Standard Weld with Backing Bar Removed
Figure 5.19 SMCS History for Built-up Weld Detail

Figure 5.20 Magnified View of SMCS at $\Delta_{\text{tip}} = 5$ inches

Figure 5.21 SMCS Contours for Standard Weld Detail at $\Delta_{\text{tip}} = 8$ inches
APPENDIX I

THE SIGNIFICANCE OF LOADING RATE EFFECTS ON THE FRACTURE RESISTANCE OF WELDED CONNECTIONS FOR SEISMIC DESIGN

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Prepared for SAC Joint Venture
Sub-Task 5.3.3
REV: July 30, 1998

PREFACE

These notes were prepared to help establish the basis for assessing the significance of loading rate on the performance of welded steel connections. The notes are by no means an exhaustive review of the subject of load rate effects but, rather, are based on brief review of readily available material and discussions the author has had with Robert Dodds and Stan Rolfe. Moreover, the notes deal specifically with loading rate effects as they influence the fracture behavior of connections. Because currently available material toughness data is insufficient to permit definitive conclusions, this write-up focuses on explaining the issues and summarizing available information on load rate effects so as to provide a common basis for discussing how to resolve them within the context of the SAC Joint Venture.

INTRODUCTION

While it is generally acknowledged that earthquake-induced load rates are not static, the influence of loading rate on material behavior is usually neglected in structural design and in much of our research. Part of the rationale for this is that in many cases the strain rates associated with earthquake effects tend to increase material strengths above those obtained in static tests. For example, the yield strength of steel and compressive strength of concrete both increase with increasing strain rates, and conventional wisdom tells us that it is conservative to neglect this effect. Fracture is an important exception to this since, under certain conditions, material cleavage fracture toughness decreases with increasing strain rate, and therefore the likelihood of catastrophic brittle fracture increases. However, the sensitivity of fracture toughness to strain rate depends on many factors, and there are even conditions such as for very tough materials where toughness increases with higher strain rates.

These notes: (1) review the definition of "loading rate" and general categories of loading rates for fracture assessment of structural steel components, (2) summarize the effects of loading rate on fracture toughness, (3) estimate fracture loading rates in connections under earthquake loading, (4) briefly review previous and current tests that provide data on strain rate effects, and (5) summarize implications on earthquake loading rate of connections and propose recommendations for future testing and analysis.
DEFINITION OF LOADING RATES

Summarized in Table 1 are general categorizations for “slow”, “intermediate” and “dynamic” loading rates for fracture analysis and testing of steels as described in the text by Barsom and Rolfe (1987). Rates are given in terms of both nominal strain rate (dε/dt) remote from crack-like defects and crack driving force (dK/dt). Of the two, dK/dt is more definitive for addressing fracture behavior. As outlined in these notes, structural loading rates under earthquake loading generally fall into the intermediate range.

<table>
<thead>
<tr>
<th>Description</th>
<th>Toughness Classification</th>
<th>dε/dt (in/in-sec)</th>
<th>dK/dt (ksi√in/sec)</th>
<th>time to peak load (seconds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slow</td>
<td>K_{ic}</td>
<td>0.0001</td>
<td>0.5 to 2.5</td>
<td>60 to 120</td>
</tr>
<tr>
<td>Intermediate</td>
<td>K_{ic}(t)</td>
<td>0.01</td>
<td>30 to 150</td>
<td>1</td>
</tr>
<tr>
<td>Dynamic</td>
<td>K_{id}</td>
<td>10</td>
<td>10^3</td>
<td>0.001</td>
</tr>
</tbody>
</table>

Slow loading is representative of rates used in typical tension coupon and K_{ic} tests where the critical parameters (e.g., σ=σ_y, K_{I} = K_{ic}) are reached in 1 to 2 minutes. The time required to reach the maximum value is referred to in Table 1 as “time-to-peak load”, and the average loading rates are simply the peak values of strain or K_{I} divided by the time-to-peak load. For example, the material fracture toughness K_{ic} is, by definition, obtained under slow loading rates limited to a maximum of dK/dt = 30 to 150 ksi√in / min (see ASTM E-399 "Standard Method of Test for Plane Strain Fracture Toughness of Metallic Materials"). Given that K_{ic} = 30 to 150 ksi√in for typical structural steels, the maximum rate permitted by ASTM E-399 means that K_{ic} will be reached in about 1 minute (or 60 seconds). For tension coupons, the strain rate of dε/dt = 0.0001 sec^{-1} applied for 1 to 2 minutes translates to maximum nominal strains of 3 to 6 times the yield strain for steel with σ_y = 60 ksi.

Dynamic loading implies that peak values of stress or K_{I} are reached within about 0.001 second. For dynamic loading, the K_{I} toughness measure is referred to as K_{id}, a critical value used to calculate the initiation of crack extension due to explosive type loads that generate high strain rates. Dynamic K_{id} tests are much less common than K_{ic} tests for two reasons. First, for most structures the actual loading rate is closer to slow than dynamic, and hence K_{ic} is a more representative measure of toughness. Second, dynamic K_{id} testing requires special equipment and procedures to apply the load and record data during the short test duration. K_{id} is more typically used for correlation with Charpy V-Notch (CVN) tests, since the standard CVN impact loading rates are considered dynamic.

As indicated in Table 1, for structural steels intermediate rates have a time-to-peak of about 1 second, but this is only meant to indicate a general order-of-magnitude range. With a 1 second time-to-peak, intermediate loading is about two orders-of-magnitude faster than slow, but three orders-of-magnitude slower than dynamic loading. Hence, intermediate rates are an order of magnitude further from dynamic than slow. Intermediate rates are, for example, often associated with truck loading rates in highway bridges. ASTM E-399 includes supplemental procedures for intermediate rate toughness testing to determine K_{ic}(t) where (t) corresponds to the time-to-peak load. The time (t) is usually left for the user to specify, presumably to correspond with loading rates for the specific application. For beam-column connection tests, the loading duration would ideally coincide with the rate that beam moments and/or hinge rotations develop during earthquakes.
EFFECT OF LOADING RATE ON FRACTURE TOUGHNESS

Shown in Fig. 1 are schematic plots of CVN data for standard impact and slow bend tests, whose load rates roughly correspond to dynamic and slow. Compared to data for slow loading, dynamic rates tend to (1) increase the brittle-to-ductile transition temperature and (2) increase the toughness in the upper shelf region. The first of these effects is quantified in terms of a temperature shift, defined as the difference in temperatures measured at a value of absorbed energy determined by intersecting tangents to the dynamic toughness curve in the lower-shelf and transition regions (see Figure 1). For structures whose operating temperatures lie in the region affected by the temperature shift, faster loading rates can dramatically reduce the fracture toughness and increase the likelihood of cleavage (brittle) fracture. On the other hand, for structures operating well into the lower shelf region (to the left of the transition region), the loading rate will not have much effect. And, in the upper-shelf region, where behavior is governed by large-scale yielding and ductile tearing, dynamic loading tends to increase the toughness - behavior also related to the increase in yield strength at higher strain rates. In summary, this data shows that the effect of load rate is very dependent on the toughness of the material relative to the operating temperature of the structure.

A similar temperature shift effect for fracture toughness ($K_{ic}$) is shown schematically in Fig. 2a where $K_{ic}$ and $K_{id}$ from the slow and dynamic tests are plotted, and at a specific temperature, $K_I$ can be related to loading rate as shown in Fig. 2b. These curves only cover the lower-shelf and transition regions since, strictly speaking, $K_I$ does not pertain to ductile upper-shelf behavior. However, upper-shelf $K_I$ values, obtain from correlation to CVN data, are often referred to in the literature and used as an approximate way to perform fracture analyses for more ductile materials. Presumably, the upper-shelf values would increase as the CVN data increases.
For structural steels with yield strengths of $\sigma_y < 140$ ksi, Barsom and Rolfe (1987) suggest the following formula to estimate the temperature shift from slow to intermediate and dynamic rates:

$$T_{\text{shift}} = (150 - \sigma_y)(de/dt)^{0.17}$$  \hspace{1cm} (1)

where $T_{\text{shift}}$ is in degrees F, $\sigma_y$ is the material yield strength in ksi, and $de/dt$ (sec$^{-1}$) is the nominal strain rate, limited to the range $0.01$/sec $< de/dt < 10$/sec. Using Eq. 1 and the strain rates from Table 1, the predicted temperature shifts for three yield strengths are given in Table 2. Note that the temperature shifts from slow to intermediate are about 30% as large as those from slow to dynamic.

<table>
<thead>
<tr>
<th>Yield Strength (ksi)</th>
<th>Temperature Shift (°F)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Slow to Dynamic</td>
</tr>
<tr>
<td>40</td>
<td>160</td>
</tr>
<tr>
<td>55</td>
<td>140</td>
</tr>
<tr>
<td>70</td>
<td>120</td>
</tr>
</tbody>
</table>

An example of rate dependent toughness data for A36 steel plate is shown in Fig. 3. CVN data for slow and dynamic (impact) tests are shown in Fig. 3a, and $K_I$ data for slow, intermediate, and dynamic tests are shown in Fig. 3b. The temperature shifts between slow and dynamic (about 150° F) and from slow to intermediate (about 30% of the total) are consistent with Eq. 1. These data demonstrate why differences between slow and intermediate rates are usually ignored for structures with A36 steel operating at temperatures of 50 to 70° F. For instance, the $K_I$ temperature-transition curves for slow and intermediate loading (Fig. 3b) both rise steeply at temperatures below -50° F, indicating that the difference between the toughness for slow and intermediate loading at +50° F is

(a) CVN data

(b) $K_I$ data

Figure 3 - Loading rate versus toughness data for A36 steel plate (B & R 1987)
probably negligible. The CVN data indicate that there may be a load rate effect at room temperature, given that the CVN_{impact} toughness at +50°F is about 40% less than CVN_{slow-bend}. However, assuming that the slow to intermediate rate shift is only 25 to 30% of the total, the difference between slow and intermediate would only be about 10%. Moreover, given that correlation equations imply CVN is proportional to (K_{Ic})^2, the 10% change in CVN would further reduce to only a 5% change in K_{Ic}.

Additional K_{Ic} data for A572 Gr. 50 steel shown in Fig. 4 provide further support that the temperature shift between slow and intermediate should not have a significant effect on brittle fractures for steel base metals at room temperature. Here, like in the previous case (Fig. 3b), the K_{Ic} curves for both slow and intermediate loading are well into the transition region below -50°F. Moreover, considering the favorable effect of dynamic loading in the upper shelf region (Fig. 1 and 3a), slow loading may well in fact be conservative insofar as more ductile fracture/tearing of the base metal is concerned. It should be noted, however, that there are reported instances (e.g., Fisher 1995, Frank 1995) of steels in rolled shapes, particularly the core region of jumbo shapes, with lower toughness than shown in Figs. 3 and 4 that have higher transition temperatures.

In welded connections, the toughness of the weld metal and HAZ is often more critical than the base metal, particularly when welds are made with low-toughness E70T-4 electrodes. Kaufmann et al. (1995a) conducted a few compact tension K_{Ic} tests at room temperature from E70T-4 welds under slow and intermediate loads and reported a difference of about 10% where slow K_{Ic} = 60 to 65 ksi/√in versus intermediate K_{Ic}(0.5sec) = 55 to 60 ksi/√in. For the intermediate tests they used a time-to-peak of t=0.5 seconds which equates to a strain rate of dε/dt = 0.02 sec^{-1}. Further they reported that these K_{Ic} data correlated fairly well to CVN data for weld samples taken from damaged buildings from which they back-calculated slow to dynamic and slow to intermediate temperature shifts equal to 110°F and 30°F, respectively. These compare well to shifts of 120°F (dynamic to slow) and 40°F (slow to intermediate) obtained using Eq. 1 with the measured weld metal yield strength of σ_y = 65 ksi and a strain rate of 0.02 sec^{-1}.

To summarize the points made above:

- Data for A36 and A572 plate material, such as shown in Figs. 3 and 4, provide evidence that for base metal, the decrease in transition region toughness between slow and intermediate rates is probably on the order of 5% to 10%. Additionally, for conditions where upper-shelf ductile tearing governs, such as for less constrained (plane stress) fractures, the toughness for intermediate loading may improve compared to slow loading.

- Available data for welds with low toughness (e.g., E70T-4 operating near the lower shelf) indicate that K_{Ic} reduces about 10% between slow and intermediate loading at room temperature.
• The temperature shift between slow and intermediate loading for both the weld and base metal is on the order of 40 to 50°F.

These points address fairly low toughness (E70T-4 welds) and high toughness (base metals) where the data suggest that the difference in loading rates are not significant. A remaining question concerns rate effects for material with in-between toughness that might include material in the HAZ and notch-toughness rated weld metal. Shown in Fig. 5, for example, are CVN_impact data for three types of weld metal, E70TG-K2, E70T-4 and E7018, with widely varying toughness. The slow and intermediate toughness data would shift to the left of these by about 120°F and 80°F, respectively. As noted earlier, the change from slow to intermediate loading does not have much effect on the low-toughness E70T-4 since the material is operating close to the flat lower-shelf curve at room temperature. The high-toughness E7018 would also not be affected by load rate since the dynamic transition temperature is well below room temperature, so the welds are governed by upper-shelf behavior. For E70TG-K2 weld metal it is not immediately apparent whether load rate can be discounted since it appears that this material may lie in a range sensitive to a temperature shift between slow and intermediate. Unfortunately, data to confirm this for E70TG-K2, or similar weld or HAZ materials, is not available.

ESTIMATED FRACTURE LOADING RATES IN BUILDINGS

Referring to Fig. 6, assume that the loading rate for overall structural response quantities $\gamma_i$ (frame drift, beam moments, hinge deformations, etc.) can be approximated by a linear time-to-peak equal to $T/4$ where $T$ is the period of the dominant mode of building vibration (typically the first or second mode). Following this reasoning, the loading time for buildings with predominant periods of $T = 1$ to 2 seconds (representative of the first modes for 4 to 10 story buildings and second modes for taller buildings) would be on the order of $t = 0.25$ to 0.5 seconds.

Global loading rates (based on indices such as moment or hinge rotation) can then be related to local fracture indices such as $K_{1}$ through detailed fracture analyses of the connections. For example, data plotted in Fig. 7 (Chi et al. 1997) show the relationship between moment and $K_{1}$ or CTOD for a pre-Northridge connection with an assumed flaw size and material properties. Note that the deviation between the solid and dashed lines in Fig. 7 at $M = 20,000$ kip-inches signifies when yielding becomes significant and inelastic analysis should be used to calculate toughness demands.
The load level at which inelastic effects become important will increase slightly with load rate as the yield strength increases.

Assuming $K_{lc} = 60 \text{ ksi} \sqrt{\text{in}}$ for E70T-4 welds, the elastic analysis data in Fig. 7 would imply that the connection would fracture at $M = 22,000 \text{ kip-inches}$ which, for the connections in question, is equal to about 85% $M_p$. For an earthquake motion that builds up gradually, or in tests where the cyclic displacements are increased gradually from one cycle to another such as in the SAC loading protocol, it is likely that the critical value of moment (e.g., when $K_1 = K_{lc}$) would be reached near the peak of a cycle. In such cases, the $K_{lc}$ time-to-peak would be equal to the global time-to-peak of $t = 0.25$ to 0.5 seconds. On the other hand, for near field earthquakes with large pulse effects, the largest loading excursions are likely to occur early in the record and fracture may occur during a pulse when moments and/or hinge rotations are rapidly increasing towards a peak that will exceed the critical value. In this case, the time-to-peak for $K_1 = K_{lc}$ is reduced and $dK_1/dt$ increases correspondingly.

How small the local time-to-peak might be in buildings subjected to pulse type ground accelerations depends upon many things including the characteristics of the building and ground motion and the inherent fracture resistance of the connection - the latter of which is related to the material toughness, the type of detail, flaw sizes, etc. For connections that are very fracture sensitive and likely to fail elastically, the peak value would be reached sooner, and hence the time-to-peak would tend to be less than the overall building response time. On the other hand, for fracture resistant connections that might survive the entire earthquake, the time-to-peak for reaching a critical value of toughness in the connection would be larger than the global time-to-peak since it would be unlikely that the critical toughness would be reached in any one cycle.

While in concept it is possible to calculate loading rates for any given connection, for the purpose of developing design/acceptance criteria, or for planning experiments, more general information is needed. As proposed in Table 3, one approach for generalizing loading rates is to first categorize connection details based on their general fracture resistance. Then, estimate $dK_1/dt$ and $d\varepsilon/dt$ rates by assuming the toughness demand would be large enough to reach a critical value during an earthquake (or during testing under earthquake loading rates). Thus, this approach begins with the premise that fracture will occur, an assumption that is conservative for the more fracture resistant connection details. As listed in Table 3, three general classifications of connection resistance are proposed based on the type of detail and material toughness, with the lowest being representative of pre-Northridge connections and the highest of well designed connections.

The time-to-peak values in Table 3 assume global building response times of $t = 0.25$ to 0.5 seconds that are equated to the local strain and $K_1$ time-to-peak for the medium class of connections. Shorter or longer time-to-peaks are then estimated for connections with lower or higher fracture resistance, respectively. The $dK_1/dt$ rates are then obtained by dividing the assumed characteristic toughness $K_{lc}$ for each class of details by the time-to-peak. The average strain rate is calculated by linearly
adjusting the rate of $\frac{d\varepsilon}{dt} = 0.01 \text{ in/in-sec}^{-1}$ from Table 1 based on the assumed time-to-peak. Finally, a temperature shift from slow to the calculated loading rate is determined per Eq. 1 with $\sigma_y = 65 \text{ ksi}$. Reviewing the results of this in Table 3, one can see that the loading rates for the three classes of connections fall into two general ranges, one for connections with low fracture resistance (e.g., pre-Northridge connections) and the second for connections with medium to high fracture resistance (e.g., post-Northridge connections).

**Table 3 - Assumed Earthquake Loading Rates for Connections**

<table>
<thead>
<tr>
<th>Class</th>
<th>Attributes</th>
<th>Fracture Resistance</th>
<th>Time to peak (seconds)</th>
<th>$\frac{dK}{dt}$ (ksi√in/sec)</th>
<th>$\frac{d\varepsilon}{dt}$ (in/in-sec)</th>
<th>$T_{shift}$ °F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>Typical pre-Northridge connections with low-toughness welds ($K_I = 60 \text{ ksi}√\text{in}$), backing bars in place, and large weld root defects.</td>
<td>0.1 to 0.2</td>
<td>300 to 600</td>
<td>0.05 to 0.10</td>
<td>50 to 60</td>
<td></td>
</tr>
<tr>
<td>Medium</td>
<td>Enhanced unreinforced detail made with moderate-toughness materials ($K_I = 90 \text{ ksi}√\text{in}$), backing bars removed, and small flaws.</td>
<td>0.25 to 0.50</td>
<td>180 to 360</td>
<td>0.02 to 0.04</td>
<td>40 to 50</td>
<td></td>
</tr>
<tr>
<td>High</td>
<td>Premium post-Northridge detail such as reduced beam sections with high-toughness materials ($K_I = 150 \text{ ksi}√\text{in}$) and small flaws.</td>
<td>0.4 to 0.80</td>
<td>190 to 380</td>
<td>0.01 to 0.03</td>
<td>40 to 50</td>
<td></td>
</tr>
</tbody>
</table>

**OTHER FACTORS**

Aside from the direct effect on fracture toughness, increased loading rates have other effects on the material behavior that can be particularly significant for cyclic loading. One is the increase in yield and ultimate strength of steel with higher strain rates, and a second is the rise in temperature that occurs due to high plastic strain rates. These, along with a few other possible rate dependent and/or dynamic effects, are briefly reviewed below.

**Yield Strength Increase:** It is generally recognized that nominal yield strengths of mild steels at intermediate earthquake loading rates can be on the order of 5 to 10% larger than for slow (static) tests. However, the impact of higher yield strengths on connection performance is not fully understood and can either increase or decrease the likelihood of fracture. These effects and their possible consequences include the following:

- Higher yield strengths increase the internal moments and shears necessary to achieve a desired connection rotation capacity. For a given rotation capacity, this will increase the probability of fracture since it will impose larger stresses on the welds. On the other hand, for a given earthquake input, the higher strengths may tend to decrease the inelastic rotation demands on the connection.

- In addition to raising yield and ultimate strengths, higher strain rates will increase the strain hardening modulus, which in turn tends to inhibit local flange/web buckling. The net effect of this is unclear since some degree of local buckling is thought to be good (because it tends to reduce the maximum hinge moment) while too much local buckling is known to be detrimental (since it leads to unloading and can cause high local strains).

- As discussed by Chi et al (1997) and alluded to previously in Fig. 7, inelastic effects can either increase or decrease local toughness demands depending on whether or not yielding concentrates around stress concentrations or internal weld defects. Accordingly, an increase in
steel yield strength would tend to raise the threshold load at which inelastic effects become significant, and this may increase or decrease the likelihood of fracture, depending on the specific circumstances.

**Temperature Rise:** In testing of steel connections under real-time earthquake loading, it is common to observe an increase in the temperature of the specimen due to heat generated by yielding. Whereas in slowly loaded tests this heat dissipates and does not build up, at intermediate loading rates the temperature increase can be significant. For example, in a recent paper on steel connection research in Japan, Toyoda and Shimanuki (1998) suggest the values in Table 4 for temperature increase as a function of the percent plastic strain under earthquake loading rates. From the standpoint of fracture, the predicted temperature increases of $\Delta T = 36$ to $72^\circ F$ are significant because they alone can be enough to offset the temperature shift of -40 to -60$^\circ F$ that reflects the loss in toughness in going from slow to intermediate loading rates. However, given that this temperature rise is related to cyclic plastic straining, whether or not it has an effect will depend upon (1) the extent of plastic straining in the connection, (2) the proximity of the plastic straining to the location of fracture, and (3) the loading history prior to fracture. If fracture occurs in the first cycle of loading without significant plastification, such as might occur for a pulse-like earthquake, then the temperature rise would be small. However, if fracture occurs after multiple inelastic excursions, then the rise can be significant and offset the negative effects of loading rate.

**Table 4 - Temperature Rise Due to Cyclic Straining**  
*(Toyoda and Shimanuki 1998)*

<table>
<thead>
<tr>
<th>Plastic Strain</th>
<th>$\Delta T$ °F (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.05</td>
<td>36 °F (20°C)</td>
</tr>
<tr>
<td>0.10</td>
<td>54 to 72 °F (30 to 40°C)</td>
</tr>
</tbody>
</table>

**Dynamic versus Static Loading:** Thus far the discussion has centered around effects that, in general, are not dynamic in the sense of being significantly affected by dynamic inertia effects. In terms of local fracture behavior around the crack tip, it is unlikely that the intermediate load rates introduce significant inertia that affects crack initiation. However, at the global building scale there may, in fact, be secondary dynamic "system" effects due to connection fractures. Here the concern is that when a heavily loaded connection fails suddenly that it may send transient stress waves that can trigger fractures in adjacent connections. While this may be a legitimate concern, it is not directly related to the other factors discussed in these notes and is mentioned here only for completeness. This behavior is not something that can be studied through testing or analysis of individual connections, but rather would require methods that addressed system behavior.

**AVAILABLE TEST DATA**

At present there is very little connection or component test data to confirm whether or not intermediate load rates are a significant factor in the performance of welded steel connections constructed using common fabrication methods in the United States. However, tests from Japan indicate that loading rate is not a major factor for connections fabricated with high-toughness weld metals and plate materials. As noted above, some data on generic base metal and weld metal material is available, but this is limited and does not cover the variability of materials (base, weld metal, HAZ) in modern welded connections.

The following is a summary of test data that the author is aware of that includes some load rate effects:
• Lehigh Tension Weldment Tests: Tests reported by Kaufmann and Fisher (1995b) of tension subassemblies provide some data on the effect of slow versus intermediate testing for welds with E70T-4 materials. They also report intermediate rate tests for other welds (E70TG-K2 and E71T-8), but without any slow test data for comparison. Two nominally identical E70T-4 specimens with intentional weld root defects and tested at slow and intermediate rates had brittle failure loads that differed by 8% (P_u,slow = 281 kips, P_u,int = 259 kips). Comparative tests of five other connections with E70T-4 welds loaded at slow and intermediate were reported, however, there were some differences in weld heat inputs and base metals between the five specimens. Failure loads for the slow tests were 353 and 349 kips, and for the intermediate rate tests were 316, 376 and 391 kips. One cannot draw statistically valid conclusions from these test data given the inherent variability in the fracture behavior and some subtle differences in the test specimens. If anything, the lack of dramatic differences in behavior support the viewpoint that slow versus intermediate rate effects are not significant for the E70T-4 specimens.

• UCSD Connection Tests: Uang (1996, 1997) conducted tests of five tests of full-scale connections, three of which were loaded statically and two loaded dynamically. The connection details were of the pre-Northridge type with W30x99 A36 beams welded with E70T-4 electrodes. Dynamic tests were run with a cyclic period of 1 second and a corresponding time-to-peak of 0.25 second – comparable to the intermediate loading rate discussed above.

Uang reports significant variation in the static tests with one failing with very limited plastic rotation (P/P_p = 1.0, θ_p = 0.008 rad) and two sustaining significantly larger loads and deformations (P/P_p = 1.2, θ_p = 0.022 rad). One of the dynamically loaded specimens failed without any plastic deformations (P/P_p = 0.9, θ_p = nil) and one sustained moderate deformations (P/P_p = 1.1, θ_p = 0.010 rad). All failures initiated with fractures in the E70T-4 flange welds that probably had toughness values in the lower-shelf or lower-transition region where load rate effects can have some effect (note that no data were reported to confirm the weld toughness). While the tests do suggest that the dynamically loaded specimens were more prone to fracture, the large variability in response for a given loading rate makes it impossible to draw definitive conclusions or to quantify the effect. Uang did not report any significant temperature rise in the dynamically loaded specimens, but this is to be expected since there was relatively little yielding in the beams prior to fracture (most of the yielding that occurred in the dynamic tests was in the column panel zone).

Another observation made by Uang is that in the two dynamic tests the fractures propagated into the column whereas in the static tests the fractures propagated vertically through the weld. One possible explanation for this may be the effect of load rate on increasing the yield strength of the shear tab relative to the fracture load. As described in finite element fracture analyses by Chi et al. (1997), the condition or level of yielding in the shear tab at failure is one possible explanation of the direction of crack propagation. Chi’s analyses suggest that the fracture is more likely to propagate into the column where the shear tab is more intact at the instant when the flange fracture initiates.

• Japan: Since the Kobe earthquake, researchers in Japan have been conducting many tests of connections and components at slow and intermediate (earthquake) rates. Many of these tests are being done at reduced temperatures (often at 0°C) to obtain lower-bound estimates of the fracture toughness. Unfortunately, it has been difficult to obtain detailed and complete information on these tests as they are still ongoing and most of the documentation is written in
Japanese. Moreover, there are significant differences in U.S. and Japan construction practice that limits the extent to which the data can be transferred. For example, the typical practice in Japan is to use higher-toughness steel plate materials welded with gas shielded welding methods.

A recent summary paper by Takanashi (1998) provides an example of Japanese test data that deals with the issue of strain rate for connections with similar geometries those in the U.S. Takanashi reports tests of welded H-shape members subjected to slow and intermediate loading rates, where he defines intermediate - or earthquake - loading rates as having cyclic loading frequencies of 1 to 4 Hz which (periods of T = 0.25 to 1 seconds). This rate is consistent with the one proposed above in relation to Fig. 6. His paper does not report details of the weld materials, but does indicate that the plate materials used in the tests have high toughness with a ductile-brittle transition temperature at around -148°F (-100°C) and CVN = 180 ft-lbs (240 J) at room temperature. Cyclic tests are conducted at three temperatures to simulate conditions where the CVN toughness varies from mid-transition, with CVN = 110 ft-lbs at -112°F (150 J at -80°C), to upper shelf values of CVN = 165 to 180 ft-lbs at -40°F and +50°F (225 to 240 J at -40°C and +10°C), respectively.

Overall, Takanashi’s conclusions from the tests tend to support the observations made previously. Specifically, (1) tests run at temperatures where the toughness was in the upper shelf were not sensitive to loading rate, and (2) tests run at lower temperatures where the toughness was at mid-transition were sensitive to loading rate. The latter conclusion is based on comparing data from two tests run at static rates with two run at intermediate rates. Two specimens tested at static rates sustained inelastic loading cycle ductilities of $\delta/\delta y = 2$ to $3$ whereas the two tests run at intermediate rates sustained cyclic ductilities of $\delta/\delta y = 1$ to $2$. Takanashi also cites data from notched bar tests where dynamic loading increases the extent of cleavage failure, as measured by crystallinity percentage, for material with CVN = 37 ft-lb at 32°F (50 J at 0°C) but decreases the crystallinity for material with CVN = 180 ft-lb at 32°F (238 J at 0°C). Note that none of the tests reported were run for materials were operating near the lower shelf toughness.

Nakashima et al. (1998) summarized the findings from 86 full-scale tests of connections between H-shaped beams and cold-formed tube columns. The connections were made with a through-column diaphragm plate that was joined to the beam with full-penetration gas-shielded groove welds. One of the variables considered in the tests was the load-rate effect where static tests were compared to intermediate rate tests (run at cyclic loading periods of 1 to 2.5 seconds). Although no weld metal toughness values were reported, it is likely that the welds had significant toughness since all of the specimens sustained plastic rotations of at least 0.055 radians. With regard to load rate effects, the tests indicated that the intermediate rate loading increased the maximum moment by about 5 to 10% compared to the static tests and did not adversely affect the inelastic deformation capacity. Moreover, a temperature rise of about 70°F (20°C) occurred in the dynamic tests that tended to change the resulting fracture surface where the dynamically loaded specimens experienced more ductile (tearing) modes of failure.

**SUMMARY AND IMPLICATIONS**

It is clear from the previous discussion that there are conditions for which loading rate can have an effect on the fracture behavior of welded connections. However, the extent to which these effects
are significant enough to warrant special consideration in design or testing primarily depends on two factors: (1) the inherent material toughness, and (2) the overall likelihood of connection fracture.

- **Material Toughness:** Loading rate effects are probably least significant in connections with either very high- or very low-toughness materials, and most significant for materials with mid-range toughness. Base and weld metals with high toughness levels at room temperature (e.g., CVN ≈ 80 ft-lbs., KIC ≈ 130 ksi√in at 70°F) will be operating close to the upper-shelf where higher loading rates tend to increase the absorbed fracture energy. Very low-toughness materials with static toughness values close to minimum lower-shelf behavior at room temperature (CVN ≈ 5 to 10 ft-lbs., KIC = 40 to 60 ksi√in at 70°F) would similarly be less affected by slow versus intermediate loading rates. This is supported by data by Kaufmann and Fisher (1995a) who reported only a 10% change in toughness between KIC tests for E70T-4 weld metal at slow and intermediate loading rates. Conversely, material with room temperature toughness in the mid-transition region would probably be most affected, but there is currently no data to quantify this.

- **Fracture Resistance:** Connections that are more resistant to fracture will tend to be less sensitive to loading rate for two reasons aside from those related to the inherent material toughness. Compared to connections that are very prone to fracture, ones that are more fracture resistant will tend to: (1) undergo more plastic straining that will increase the temperature and, thereby, offset any reduction in toughness associated with the negative temperature shift, and (2) have slower strain and KI loading rates for the same global loading rate.

The main implication of these two factors is that efforts to understand and quantify strain rate effects should focus on connection details that have good, but not excellent, fracture resistance. This might imply connections with fully welded details, notch-tough electrodes, and removed or reinforced backing bars that are otherwise similar to pre-Northridge designs, e.g., analogous to the medium class of connections in Table 3. Further, if it is decided to run intermediate rate tests of full-scale connections, the adverse effect of loading rate may be more apparent in tests loaded by a single large cycle rather than by multiple cycles of gradually increasing deformations, since the latter would tend to cause a larger temperature rise that would offset the rate effects. (This is assuming that the critical fracture condition involves some amount of cleavage fracture and not cumulative tearing).

For the purpose of planning future tests, it is imperative to recognize that the inherent variability in fracture behavior will require a large number of tests in order to develop meaningful data on rate effects. For example, consider fracture behavior observed in four nominally identical connections tested at U.C. Berkeley and U.T. Austin during phase one of SAC (summarized in Chi et al. 1997). Failure loads in the three tests from U.T. Austin, all constructed by a single welder-fabricator, varied by up to -15% and +12% from the average. The fourth test from U.C. Berkeley (constructed by another fabricator) had a failure load 30% larger than the average from U.T. Austin. While some of these differences can be traced in part to differences in flaw sizes (as measured after the tests) it would be difficult to isolate loading rate from other variable effects with just a few tests. Put another way, one should ask the question whether anything new would have been learned by testing one of the four connections at a higher rate, assuming that for E70T-4 welds the difference in toughness for intermediate and slow rates is on the order of 10%.
Therefore, rather than attempting to isolate rate effects in large scale connection tests, a much more cost effective and worthwhile approach would be to undertake material and small-scale component tests to quantifying some of the fundamental aspects of rate effects. Reliable and statistically meaningful data on basic material and component effects, including the effect of intermediate strain rates, could then be integrated to develop sound design guidelines and criteria through analysis and design models. Such tests would include weld metal and HAZ toughness testing (CVN, $K_I$ and CTOD) and additional component tests such as done by Kaufmann and Fisher (1995b) and planned for Task 7.05 or simpler ones that are currently underway as part of the Welding and Joining TAP (Tasks 5.2.1 and 5.2.2).

REFERENCES


Appendix II

English to SI Conversions

<table>
<thead>
<tr>
<th>English Unit</th>
<th>SI Equivalent</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 inch</td>
<td>25.4 mm</td>
</tr>
<tr>
<td>1 lb</td>
<td>4.454 N</td>
</tr>
<tr>
<td>1 kip</td>
<td>4.454 kN</td>
</tr>
<tr>
<td>1 ksi</td>
<td>6.895 MPa (N/mm²)</td>
</tr>
<tr>
<td>1 ft-lb</td>
<td>1.356 Joule</td>
</tr>
<tr>
<td>1 ksi√in</td>
<td>34.75 MPa√mm</td>
</tr>
</tbody>
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