

STRUCTURAL ENGINEERS ASSOCIATION OF CALIFORNIA
SEISMOLOGY COMMITTEE



COMMENTARY AND RECOMMENDATIONS

on

FEMA 350

Prepared by

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January 2002

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EXECUTIVE SUMMARY

In July 2000 FEMA published four documents comprising the final recommendations of its Program to Reduce the Earthquake Hazards of Steel Moment-Frame Structures. The four documents, produced by the SAC Joint Venture and numbered FEMA 350 through FEMA 353, are reference documents for engineers and resource documents for code-writing organizations. Design provisions for new construction are given in FEMA 350, titled *Recommended Seismic Design Criteria For New Steel Moment-Frame Buildings*.

The Commentary and Recommendations presented here are intended to bridge a potential gap between FEMA 350 and the building code. They are intended to help engineers and building officials implement FEMA 350 while consensus standards and code provisions are being developed by others. This document, produced as a service to SEAOC members, addresses issues in steel moment frame design in California in light of FEMA 350, the SAC research, and other pertinent work. It also presents the position of the SEAOC Seismology Committee regarding implementation of FEMA 350.

The SEAOC Seismology Committee encourages engineers and building officials to read FEMA 350 and to use it as the reference it was intended to be. Some of the FEMA 350 criteria represent significant changes relative to previous design practice, and it is incumbent upon engineers and building officials to be familiar with this new state of practice.

This SEAOC Seismology Committee document highlights and supports many of the new criteria recommended by FEMA 350. As a commentary, it offers additional reference information and attempts to identify potentially critical design conditions. In some places, the SEAOC Seismology Committee position differs from FEMA 350. The SEAOC Seismology Committee's supporting, amending, and dissenting positions are summarized in Table 1.

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PART A INTRODUCTION

1. Objectives and Limitations

This Commentary and Recommendations document presents the position of the SEAOC Seismology Committee regarding implementation of FEMA 350. It addresses Special Moment Frames and, to a lesser extent, Ordinary Moment Frames. (The Committee expects to address OMFs in more detail in a separate document.) This Commentary is intended to supplement the FEMA documents as a service to practicing engineers, and it should be viewed as a continuation of FEMA's efforts to improve moment frame performance. Hopefully, it will also open conversation among practicing engineers, researchers, building officials, and other stakeholders regarding incorporation of the FEMA recommendations into building codes and standards.

The specific objectives of the Commentary and Recommendations are to:

1. Identify those FEMA 350 recommendations most likely to affect current design practice in California.
2. Provide guidance where FEMA 350 does not offer specific recommendations.
3. Identify areas where further research is needed before specific design guidelines can be recommended by the SEAOC Seismology Committee.

For now, the task group's focus is on design criteria for new construction, covered in FEMA 350. The final July 2000 versions of FEMA 350-353 were available for the task groups' review in preparing this document. The 100 percent draft versions of the State of the Art reports, including FEMA 355D, were also used in preparing this document. Final versions of the FEMA 355 State of the Art reports and FEMA 354, a Policy Guide, have since become available.

This document is the result of an examination of FEMA 350 and its supporting documents, with particular attention to the subject of prequalified connections. Connections not prequalified by FEMA 350 have not been examined and are not discussed in detail here. However, several of the findings presented here may also be applicable to non-prequalified connections. For beam-column connections outside of the FEMA 350 prequalification parameters, the SEAOC Seismology Committee strongly recommends qualification testing as outlined in FEMA 350. This is of particular importance for deep columns and very large beam sections.

Even as the SEAOC Seismology Committee is reviewing the FEMA recommendations and preparing this Commentary for SEAOC members, other organizations (such as AISC and BSSC) are adopting or modifying some of FEMA's recommendations. Some jurisdictions may adopt related code requirements ahead of others, and inconsistencies between various codes and standards are likely to persist for at least several more years. Some jurisdictions, as of October 2001, accept as a matter of policy the use of FEMA documents for design, detailing, and construction of moment frame connections. Neither the FEMA documents nor this Commentary supercedes the design criteria or code provisions of local building departments.

2. FEMA Documents

The FEMA criteria and State of the Art reports are available as noted below. As noted throughout this Commentary, designers are strongly encouraged to familiarize themselves with FEMA-355D, which provides important and useful information on test results and design procedures.

FEMA documents can be ordered free of charge by calling 800-480-2520. SAC Background Documents, listed in the back of each FEMA publication, are expected to be made available through ATC, and eventually through the SAC website, www.sacsteel.org. As this Commentary neared completion, errata to FEMA 350 and 353 became available on the AISC web site along with other FEMA documents. The AISC home page is www.aisc.org.

FEMA 350, July 2000, *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings.*

FEMA 350 Errata, March 16, 2001

FEMA 351, July 2000, *Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings.*

FEMA 352, July 2000, *Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings.*

FEMA 353, July 2000, *Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications.*

FEMA 353 Errata, March 16, 2001

FEMA 354, November 2000, *A Policy Guide to Steel Moment-Frame Construction.*

FEMA 355A, *State of the Art Report on Base Metals and Fracture.*

FEMA 355B, *State of the Art Report on Welding and Inspection.*

FEMA 355C, *State of the Art Report on Systems Performance of Steel Moment-Frames Subject to Earthquake Ground Shaking.*

FEMA 355D, *State of the Art Report on Connection Performance.*

FEMA 355E, *State of the Art Report on Past Earthquake Performance of Moment-Resisting Steel Frame Buildings.*

FEMA 355F, *State of the Art Report on Performance Prediction and Evaluation of Steel Moment-Frame Buildings.*

PART B FINDINGS

This section presents the Commentary and Recommendations of the SEAOC Seismology Committee with respect to specific FEMA 350 provisions. The findings are organized to correspond to FEMA 350 chapter and section numbers. Each finding is also classified according to which of the three principal objectives it most serves (see Part A). The findings are of four types, indicated in the text by indented, italicized notes. Table 1 summarizes the findings.

Types 1a and 1b

These are indicated by either:

- * 1a * This FEMA 350 recommendation is a significant change in previous practice.*
- or
- * 1b * This FEMA 350 recommendation is a significant change in previous practice. In addition, the SEAOC Seismology Committee recommends the following: ...*

Type 1 findings identify those FEMA 350 recommendations most likely to affect current design practice in California. These findings represent Commentary that essentially agrees with FEMA 350. Their main purpose is to call attention to significant changes relative to pre-Northridge or pre-FEMA 350 design practice. In some cases (Type 1b), the findings may also offer advice for implementing the particular FEMA recommendation.

Type 2

These are indicated by:

- * 2 * The SEAOC Seismology Committee recommends additional considerations (revisions) as follows: ...*

Type 2 findings provide guidance where FEMA 350 does not offer specific recommendations. These findings supplement or correct the FEMA recommendations.

Type 3

These are indicated by:

- * 3 * The SEAOC Seismology Committee's position or conclusions vary from FEMA 350. The SEAOC Seismology Committee recommends:...*

Type 3 findings indicate areas where further research is needed before specific design guidelines can be recommended by the SEAOC Seismology Committee. These findings represent cases where the Committee's position is contrary to the FEMA recommendations. In most cases, the difference represents the Committee's opinion that the FEMA recommendation is not sufficiently supported by research results, does not reflect enough of a consensus judgment among California engineers and building officials, or is otherwise at variance with standard practice in California.

Table 1. Summary of Findings

PART B SECTION	FEMA 350 REFERENCE SECTION	FINDING TYPE ¹
1 State of the Art Reports	1.1 and 3.1	--
2 FEMA 350, Building Codes, and Current Practice	1.2, 1.4 and 2.2	1a
3 Shear on Flanges	1.3 (top of page 1-8); FEMA 355D Section 2.1.2	1b
4 Qualifying Inelastic Connection Rotation Angles	2.5.3	1b
5 Computer Modeling of Panel Zone Stiffness	2.8.2.3	2
6 Column Moment Magnification	2.9.1	1b
7 Panel Zone Performance	3.3.3.2 and 2.9.3	1b
8 Columns Deeper Than W14	2.9.6; FEMA 355D Section 4	1b
9 General Design Equations	3.2.7 and 3.3.3.2	1b
10 Beam Flange Thickness Effects	3.3.1.4	2
11 Lateral Bracing of Beam Flanges near Plastic Hinges	3.3.1.5	2
12 Weld Interface	3.3.2.1	--
13 Base Material Properties	3.3.2.2 and 3.3.2.3	--
13.1 Toughness		1b
13.2 Yield Strength/Ductility		1b
14 Weld Metal Toughness	3.3.2.5 (with 3/16/01 Errata); FEMA 353 Sections 2.1.1.2 and 2.4.1.1	1b
15 Modified access hole	3.3.2.7	1a
16 Low Cycle Fatigue	3.3.2.7	1a
17 Welding Quality and Inspector Certification	3.3.2.8	1a
18 Basis of Connection Prequalification	3.4	1a
19 Prequalified Fully Restrained Connections	3.5 and 3.6	--
19.1 WUF-W	3.5.2	3
19.2 FF	3.5.3	3
19.3 WFP	3.5.4	3
19.4 BFP	3.6.3	3
19.5 BUFP and BSEP	3.6.1 and 3.6.2	1b
19.6 RBS	3.5.5	1a
20 Application to IMF and OMF Systems	3.5, 3.6, 3.7, 3.9.2, and 4.6.2	1b
21 Welding Parameters and Categories	3.5 and 3.6	1b
22 Connection Details at the Roof	3.5, 3.6, and 3.7	2
23 Testing Procedures and Acceptance Criteria	3.9.1 and 3.9.2	1b
24 Prequalification Testing Criteria	3.10	--
25 Immediate occupancy Performance Level	4.2.2	1b

Note 1: Refer to accompanying text on previous page for explanations of each finding type. The following abbreviated descriptions of "finding type" are used throughout Part B of this report:

*** 1a *** *This FEMA 350 recommendation is a significant change in previous practice.*

*** 1b *** *This FEMA 350 recommendation is a significant change in previous practice. In addition, the SEAOC Seismology Committee recommends the following: ...*

*** 2 *** *The SEAOC Seismology Committee recommends additional considerations (revisions) as follows: ...*

*** 3 *** *The SEAOC Seismology Committee's position or conclusions vary from FEMA 350. The SEAOC Seismology Committee recommends:...*

1. State of the Art Reports
(FEMA 350 Sections 1.1 and 3.1)

As stated in FEMA 350 Section 3.1, the research that supports the FEMA 350 recommendations regarding prequalification is summarized in the FEMA 355D State of the Art report, and detailed test results are found in separate research reports. The SEAOC Seismology Committee advises engineers to use these summaries and research reports to understand expected connection performance at a detailed level.

2. FEMA 350, Building Codes, and Current Practice
(FEMA 350 Sections 1.2, 1.4, and 2.2)

*** 1a * This FEMA 350 recommendation is a significant change in previous practice.**

FEMA 350 supercedes FEMA 267 and its updates. However, as with FEMA 267, FEMA 350-353 do not substitute for code provisions. They should be used as reference or resource documents. Refer to FEMA 350 Section 1.2.

Use of the FEMA documents for frame and connection design might not be in compliance with state or local codes or the policies of local jurisdictions. Furthermore, SAC does not intend FEMA 350 to be directly adopted into codes. From FEMA 350 Section 1.4: "... users are also warned that these recommendations have not undergone a consensus adoption process. Users should thoroughly acquaint themselves with the technical data upon which these recommendations are based and exercise their own independent engineering judgment prior to implementing these recommendations."

The AISC Seismic Provisions for Structural Steel Buildings is the reference or source document for major model codes such as the UBC and IBC. Code adoption, however, lags behind publication of AISC updates. The SEAOC Seismology Committee advises engineers designing steel seismic-resisting structures to be familiar with the latest version of the AISC Seismic Provisions, even though it may not yet be adopted into code. Consideration should also be given to the use of the latest Provisions for design. AISC's Supplement No. 2 to the 1997 Provisions (AISC, 2000) incorporates initial findings from the SAC Phase 2 project. AISC Technical Committee TC-9, as of October 2001, is reviewing the FEMA documents and is preparing the next update to the AISC Seismic Provisions. It is anticipated that this update will be available in early 2002.

As of October 2001, the 1997 Uniform Building Code serves as the model code for building design in California. The code references the 1992 edition of the AISC Seismic Provisions for Structural Steel Buildings. Some jurisdictions have amended the UBC to reference the 1997 AISC Seismic Provisions.

The 1997 Uniform Building Code will serve as the model building code in California through 2004. (Application of the 1997 UBC structural provisions to hospitals will be new.) State agencies, such as DSA and OSHPD, have proposed an amendment to the UBC that would reference and amend the 1997 AISC Seismic Provisions, including Supplement No. 1 (AISC, 1999). However, this amendment, if approved by the California Building Standard Commission, will only apply to buildings regulated by these State agencies.

Unless noted otherwise, references in this SEAOC Commentary and Recommendations document to the AISC Seismic Provisions also include both the 1997 provisions (AISC, 1997) and Supplement No. 2 (AISC, 2000). The SEAOC Seismology Committee's commentary on the 1997 AISC Seismic Provisions can be found in Chapter 7 of the 1999 Blue Book.

3. Shear on Flanges

(FEMA 350 Section 1.3, top of page 1-8; FEMA 355D Section 2.1.2)

*** 1b *** *This FEMA 350 recommendation is a significant change in previous practice. In addition, the SEAOC Seismology Committee recommends the following: For conditions that impose high shear demand, shear plates should be welded. Use of welded shear plates should be considered for frame members with low span-to-depth ratios (less than about 8 or 9). Beam web attachment details should be similar to the WUF-W connection (see FEMA 350 Section 3.5.2), which generally exhibited improved performance.*

Richard et al (1995) and Goel et al. (1997) have shown that significant shear can be carried by the beam flanges, resulting in significant stress concentrations at the beam flange to column flange interface. This is discussed in FEMA 350 Section 1.3. However, the design methodologies for the prequalified connections (e.g. WUF-B, WUF-W, FF, WFP and BFP) do not account for shear on the flanges. The beneficial effects of welded shear plates are demonstrated by SAC tests.

Shear in the beam flanges can be significant, on the order of 25 percent of the total beam shear in each flange. Factors that influence the shear acting on flanges include:

- Vertical shear at the column face can increase as beam span decreases, depending upon joint and frame configuration, thus increasing the shear force resisted by beam flanges.
- Bolted shear plates, which permit some slip, may not be sufficiently effective in carrying shear.
- It is assumed that shear force resisted by beam flanges increases as beam flange thickness increases, owing to greater relative stiffness of the thicker flanges.

Refer to Part B Section 15 for commentary on effects of modified access holes on flange shear.

4. Qualifying Inelastic Connection Rotation Angles

(FEMA 350 Section 2.5.3)

*** 1b *** *This FEMA 350 recommendation is a significant change in previous practice. In addition, the SEAOC Seismology Committee recommends the following: Engineers should consider the drift demands expected for specific structural designs. FEMA 350 assumes an elastic drift capacity of 0.01. Frames that reach full yield at drifts less than 0.01 might require higher inelastic drift capacity in order to resist the required total interstory drift.*

FEMA 350 uses “interstory drift angle” to characterize both connection demand and connection capacity. A connection’s interstory drift angle capacity is the sum of its elastic capacity (used here to mean maximum possible elastic rotation) and its inelastic capacity. FEMA 350 assumes a typical elastic capacity of 0.01 radians. FEMA 355D reports the maximum inelastic rotations achieved in tests by SAC and others. Combining the two gives a total interstory drift angle capacity that can be compared with an expected drift angle demand.

This logic is incorrect, however, if the maximum possible elastic drift contribution is less than the design assumes. A connection’s maximum possible elastic contribution can vary depending on the connection type and the geometry of the beam and column framing. Frames with many closely-spaced columns and short beam spans are likely to exceed elastic limits at lower interstory drifts than are the more conventional frame configurations represented by most of the SAC and non-SAC tests. If the connection’s maximum possible elastic contribution is less than the assumed value, then the connection must make up the difference with greater inelastic capacity, which might not be available.

5. Computer Modeling of Panel Zone Stiffness
(FEMA 350 Section 2.8.2.3)

- * 2 *** *The SEAOC Seismology Committee recommends additional considerations (revisions) as follows: Schneider et al. (1998), Lee and Foutch (2000), and El-Tawil et al. (1998) are recommended references on panel zone stiffness and analytical modeling.*

FEMA 350 Section 2.8.2.3 calls for frame stiffness to be calculated using centerline dimensions, but it allows for “more realistic assumptions” regarding panel zone and connection stiffness when justified by “appropriate analytical or test data.” The references listed above may be useful in this regard.

Schneider et al. note that the use of a 50%-reduced panel zone (a common design practice) can be unconservative. They recommend a fully rigid panel zone modified by analytical methods or test data that account for the actual rigidity of the specific connection type.

6. Column Moment Magnification
(FEMA 350 Section 2.9.1)

- * 1b *** *This FEMA 350 recommendation is a significant change in previous practice. In addition, the SEAOC Seismology Committee recommends the following: Designers should select beam and column sizes to provide more favorable strong column/weak beam relationships. Refer to AISC (2000) for column compactness and lateral torsional bracing requirements based on joint strength ratio.*

Column moment magnification can result in column moments significantly higher than simplified analytic methods would predict (Paulay and Priestley, 1992; Bondy, 1996). FEMA 350 acknowledges this as well, noting in Section 2.9.1 that plastic hinging of columns can occur even with strong-column-weak-beam conditions “because the point of inflection in the column may move away from the assumed location at the column mid-height once inelastic beam hinging occurs, and because of global bending induced by the deflected shape of the building.”

The potential for column yielding can be affected by conditions not typically considered by designers. These include:

- Unknown beam-to-column connection behavior due to column hinging. None of over 400 tests monitored by SAC, to the knowledge of the SEAOC Seismology Committee, exhibited unexpected column yielding outside the panel zone.
- Reduction in the overall stability of the frame.
- Unknown ability of large members and members with thick webs or flanges to develop plastic hinges.

See SEAOC (1999), section C703.5, for further discussion of this subject.

7. Panel Zone Performance
(FEMA 350 Sections 2.9.3 and 3.3.3.2)

- * 1b *** *This FEMA 350 recommendation is a significant change in previous practice. In addition, the SEAOC Seismology Committee recommends the following:*

1. *Engineers should determine the shear stresses in the panel zone due to the application of the sum of column moments. These should not be significantly greater or less than the panel zone shear stresses that occurred in the applicable test specimens (see FEMA 355D). Note: Use of panel zones sized to match qualification tests may result in non-compliance with the FEMA 350 panel zone design procedure.*

2. *Engineers are advised to examine applicable test results to determine the degree to which tested performance relied on panel zone yielding and to understand the relative contributions of beams, columns, panel zones, and connections to the sub-assembly's total drift angle capacity.*

Panel zone yielding strongly affects connection performance (FEMA 355D, Section 2.2.2). A stiff panel zone that remains elastic while other components yield contributes less to the total plastic rotation capacity than a weak, yielding panel zone. Too weak a panel zone, however, can result in kinking of the column flange and subsequent poor performance. For a given connection type, the panel zone design should therefore attempt to match the panel zone deformation of the applicable qualification test specimens.

Due to allowable variations in material strength, it is often impossible for the designer to predict the actual panel zone performance. With ASTM A992 steel, which sets maximum and minimum yield stresses at 65 ksi and 50 ksi respectively, the strength ratio between a theoretically matched beam and column can range from 0.77 to 1.3. As a result, the relative contribution of the panel zone to the assembly's inelastic capacity is not easily estimated, and real structures may have significantly more or less capacity than the test specimens on which they are based.

Further, panel zone demands in two-sided joints (two beams, one on either side of a column) can be significantly different from the demands on one-sided joints. While some two-sided specimens have been tested, the majority of applicable connection tests conducted before and since the Northridge earthquake have been on one-sided specimens.

Refer to Part B, Section 19 for discussion of panel zone influence on the performance of specific connection types.

The panel zone design procedures of FEMA 350 Section 3.3.3.2 are largely based on a theoretical model of panel zone shear strength and do not necessarily reflect the observed performance of panel zones in tested assemblies. As discussed below (Part B, Section 19), the panel zones of some test specimens varied significantly from what the FEMA 350 design criteria would have required.

8. **Columns Deeper Than W14** (FEMA 350 Section 2.9.6; FEMA 355D Section 4.7)

- * 1b * *This FEMA 350 recommendation is a significant change in previous practice. In addition, the SEAOC Seismology Committee recommends the following:***
Connection designs using columns deeper than W14 should be qualified by testing per FEMA 350 Section 3.9. Deep columns might require stiffeners to control panel zone buckling, beam hinge bracing to reduce twisting moments on the column, and/or bracing to control column twist.

FEMA 350 presents eight connection types as "prequalified" for use in either Special or Ordinary Moment Frames (see FEMA 350 Section 2.10, Table 2-2). When used in SMFs, these connections are only prequalified for use with W12 or W14 columns oriented for strong axis bending. FEMA 350 makes no restrictions on column size when these eight connection types are used in OMFs. A ninth connection type (WUF-B) is prequalified for use in OMFS only and is limited to W8, W10, W12, or W14 columns.

FEMA 355D Section 4.7 (Table 4-3) lists 17 tests conducted with deep columns and a range of connection types, noting "substantial scatter in test results." It concludes that deep column sections, on average, do not perform as well as W12s and W14s. The FEMA documents cite four main reasons for poorer performance:

- Deep columns have greater need for continuity plates due to thinner webs and thinner flanges. Without continuity plates, "deterioration and loss of resistance" is noted in hysteresis curves.

- Deep columns have different panel zone characteristics since the webs are thinner, deeper, and more prone to inelastic shear buckling.
- Deep columns provide less resistance to out-of-plane and lateral torsional buckling than do W14 columns. This can allow twisting and out-of-plane deformation, leading to deterioration.
- Deep columns are more commonly rotary straightened than are W12 and W14 column sections. Rotary straightening is known to decrease the notch toughness of the steel in the K-line region and thus increase the potential for K-line fractures.

In addition to the deep column tests listed in FEMA 355D Table 4-3, a test performed at the University of Utah reportedly exhibited brittle panel zone failure and column flange kinking in a W24x176 column (No connection test report available).

FEMA 355D Table 4-3 includes a test performed at UCSD on a W27x194 column that fractured along the K-line adjacent to the beam bottom flange. Barsom and Pellegrino (2000) report on a SAC-sponsored fractographic analysis of this specimen. Barsom and Pellegrino conclude that the fracture was not caused by pre-existing defects and was not influenced by the fracture toughness of the K-area. They refer to the 1999 interim test report by the UCSD team. The full report for that test can be found in Gilton, et al. (2000a).

Gilton et al. (2000a) discuss the out-of-plane deformations and the severe twisting that can occur in deep columns. They suggest three mitigation options:

- Change the column to a section with better torsional properties.
- Provide extra lateral bracing a short distance outside the RBS region to minimize the amplitude of lateral torsional buckling.
- Prevent column twisting by bracing the column flange instead of the beam flange.

In addition, Figure C-9.3 of the AISC Seismic Provisions (1997) illustrates a doubler plate configuration that can be expected to improve the torsional properties of the column.

Flynn (2000) cites both Barsom and Pellegrino (2000) and the interim report by Gilton et al. and suggests that the use of deep columns should be a focus of further discussion and/or study by AISC and others. According to the AISC TC-9 Committee (Seismic Provisions), AISC intends to sponsor a deep column research program commencing in 2002.

It is the position of the SEAOC Seismology Committee that more research is necessary to verify various mitigation options and to confirm the Barsom and Pellegrino conclusion that metallurgical and material concerns are not a problem. Therefore, if deep columns are proposed for use on projects, the design should be qualified by testing in accordance with FEMA 350 Section 3.9.

9. General Design Equations (FEMA 350 Sections 3.2.7 and 3.3.3.2)

*** 1b * This FEMA 350 recommendation is a significant change in previous practice. In addition, this committee recommends the following:**

1. Modify FEMA 350 Section 3.2.7, Equation 3-3, to remove the factor C_y from the gravity load portion of V_p :

$$M_{yf} = S_b \cdot R_y \cdot F_y + (C_y \cdot V_p + V_G \cdot (1-C_y)) \cdot x$$

Where V_G is the gravity load beam shear

2. Modify FEMA 350 Section 3.3.3.2, Equation 3-7, to account for two sided connections. Replace M_c with SM_c :

$$t = \frac{C_y \cdot \Sigma M_c \cdot \frac{(h-d_b)}{h}}{0.9 \cdot 0.6 \cdot F_{yc} \cdot R_{yc} \cdot d_c \cdot (d_b - t_{fb})}$$

Errata to FEMA 350 were issued, dated March 16, 2001 that includes additional corrections.

10. Beam Flange Thickness Effects (FEMA 350 Section 3.3.1.4)

* 2 * *The SEAOC Seismology Committee recommends additional considerations (revisions) as follows:*

1. *Where the flange thickness exceeds 1-1/2 inches, and particularly where shop welded, double bevel welds might be useful in reducing residual weld stresses. If double bevel welds are used in the field, special prequalification tests for welders may be required. Controlled cooling per FEMA 353 Section 3.3.9 and Post Weld Heat Treatment per FEMA 353 Section 3.3.10 in highly restrained conditions can be beneficial.*
2. *Longer weld access holes (such as that given in FEMA 350 Figure 3-5) can be beneficial in reducing residual weld stresses.*

FEMA 350 limits flange thickness to 1-1/2 inches for all prequalified connections except RBS. FEMA 350 Section 3.3.1.4 warns that thicker flanges require larger welds, for which "greater control may be necessary..., and quality control may be more difficult. Additionally, residual stresses are likely to be higher in thicker material with thicker welds." Dong and Zhang (1998) showed that residual stresses can significantly affect the plastic deformation capacity of welded joints.

Tsai et al. (2001) used finite element analysis to analyze the effects of welding processes and benefits of longer access holes on reducing residual stresses.

11. Lateral Bracing of Beam Flanges near Plastic Hinges (FEMA 350 Section 3.3.1.5)

* 2 * *The SEAOC Seismology Committee recommends additional considerations (revisions) as follows: The influence and design of hinge bracing requires further investigation. The following recommendations are therefore provisional:*

1. *If bracing is provided, it should be located between $d/4$ and d from the outside (i.e. away from the column) edge of the plastic hinge region.*
2. *Bracing should be designed to resist expected force levels. In lieu of analysis or testing, this force may be taken as 6 percent of the flange force at M_p . connections of the bracing to the beam should be detailed to eliminate any appreciable slippage.*
3. *Full height vertical stiffeners, at the bracing location, should be provided to prevent cross sectional warping while providing adequate strength and stiffness. The stiffened cross section may be braced with conventional wide flange framing to prevent lateral and torsional displacement.*
4. *The influence of skewed braces on beam rotation has not been studied and may not be as effective as perpendicular bracing members in mitigating lateral and torsional displacement of moment beams.*

5. *Since bracing of the frame beam against lateral displacement of its flanges also appears effective in limiting the torsional demand on the column, bracing should be considered for conditions with deep (i.e. torsionally flexible) columns.*

6. *Bracing should be considered when performance beyond minimum requirements is desired and when the effects of lateral torsional buckling on architectural elements must be minimized.*

7. *The proximity of hinge bracing to the plastic zone of the beam suggests that seismic critical welds might be appropriate for brace attachment.*

8. *Where the top flange is braced by a concrete slab, sufficient shear studs should be provided to resist the brace force (see 2 above).*

FEMA 350 allows that when plastic hinges occur away from the column face and "Where the beam supports a slab and is in direct contact with the slab along its span length, supplemental bracing need not be provided." The FEMA 350 commentary cites "limited testing" (refer to Gilton et al (2000)) and refers to FEMA 355D, but does not offer analytical justification. The FEMA 350 recommendation appears to consider the stability of the assembly at a plastic rotation of 0.03 radians with no significant strength degradation. At higher rotations, however, tests Gilton et al (2000b) show that improved performance (less strength degradation) is possible with bracing located just beyond the plastic hinge region (i.e. farther from the column face).

The 6 percent recommendation given here is consistent with requirements of the upcoming 2002 AISC Seismic Provisions for RBS connections. Tests and analysis by Richards and Uang (unpublished) appear to indicate that brace forces increase the larger the distance between the bracing point and the plastic hinge. Richards and Uang's work also shows that any gap and/or any slippage of the connection of the lateral bracing to the beam will significantly increase the force in the brace.

AISC is currently developing hinge brace design procedures that will consider both stiffness and strength of the brace. Note that the intermittent bracing between hinges is often designed for 2 percent of the beam flange force at hinge yielding which is significantly less than the recommended 6 percent level for the location near the plastic hinge. (Code requirements for intermittent lateral bracing between hinges must be met regardless of whether hinges are braced.)

Also, beam hinging can be accompanied by significant distortion of the beam bottom flange due to lateral torsional buckling. Bracing of the beam may prevent or reduce architectural damage associated with this distortion, for example to window walls, precast panels, ceilings, etc.

12. Weld Interface (FEMA 350 Section 3.3.2.1)

The SEAOC Seismology Committee maintains (and the FEMA documents acknowledge) that the exact influence of certain field conditions at the welded beam flange-to-column flange joint is still not entirely predictable. Tests used by SAC (described by Dexter et al, 2002) to prequalify connection details did not necessarily duplicate or capture the full range (or likely combinations) of:

- Material and workmanship flaws.
- Weld and base metal toughness.
- Stress concentrations.
- Variable column material.
- Column flange thickness.
- Shear forces at the column face.
- Axial tension in the column flange (although most tests induced substantial flexural tension).

Refer to the SEAOC Blue Book (SEAOC, 1999) section C703.2 for further discussion.

SAC addressed some of these issues with various analytical studies or component tests. The cyclic tests of full-size beam-to-column connections, however, focused on overall behavior and performance. Consequently, tests by SAC and others on connections with flanges yielding (or near yielding) at the column face may not have captured all of the above-mentioned variables within their expected ranges or combinations. Therefore, to say that the tests do not support actual designs is a reasonable but conservative argument. For the prequalified connections, the tested combinations of materials and member sizes did successfully avoid pre-Northridge failure modes. Nevertheless, because the test matrix was not complete with respect to the parameters listed above (nor could it have been), engineers should consider whether untested combinations within the prequalified ranges might be similar to pre-Northridge details and therefore vulnerable to brittle behavior. (FEMA 350 Section 1 and FEMA 355D Sections 2 and 7.2 discuss the fundamental characteristics of pre-Northridge connections).

FEMA 350 and 355D do not provide a design procedure that specifically addresses flaws, etc. at the beam flange welded joint. Instead, FEMA 350 prequalification procedures were largely based on repeated successful performance of full-size beam-column assemblies, as well as analytical results, a wide range of component tests, and considerable judgement. One could argue that this global approach is more appropriate for prequalification of new connection types. Rather than focus on the theoretical prediction of local stress and strain at one sensitive location, or on small component tests investigating the variability of parameters, SAC sponsored full-size tests involving connection details expressly designed to reduce demands at critical locations within the connection.

Finally, while some parameters were not exhaustively studied, the large scope of testing that was performed should be recognized. Engineers and building officials should consider that no alternative structural system, in steel or any other material, has benefited from systematic testing and analysis similar to that performed since 1994 for steel moment-resisting frames.

13. Base Material Properties (FEMA 350 Sections 3.3.2.2 and 3.3.2.3)

*** 1b * This FEMA 350 recommendation is a significant change in previous practice. In addition, the SEAOC Seismology Committee recommends the following:**

- 1. The ASTM A992 specification should be amended to require Charpy V-Notch tests to confirm the values required by FEMA 350 Section 3.3.2.2 for all frame members. (Supplement SX3 is insufficient.) Mill certification with CVN test results should accompany each piece. ASTM A572 for plates should comply with supplement S5, which requires detailed requirements to be specified. Engineers are strongly advised to understand applicable ASTM specifications.*
- 2. More stringent toughness requirements might be necessary for service temperatures lower than 50 °F.*

13.1. Toughness

Changes in production techniques might modify the quality of steel. Review of FEMA 355A is strongly recommended. Chapter 1 of FEMA 355A provides an overview of the steel making processes while the remaining chapters provide information concerning material properties.

FEMA 350 Section 3.3.2.2 recommends that frame members should have Charpy V-Notch (CVN) toughness of at least 20 ft-lb at 70 °F and that it should be specified for members with flanges 1-1/2 inch or thicker and plates 2 inches or thicker. AISC's Supplement No. 2 (AISC, 2000) incorporates this requirement, as anticipated in the FEMA 350 commentary.

Supplement SX3 to the ASTM A992 specification appears to cover this requirement, but it applies only to Group 4 and 5 shapes. Engineers will need to amend A992 to cover the Group 3 shapes with thick flanges. The FEMA 350 recommendation should apply to all frame members, not just those listed in the supplement. Also, a mill certification with CVN values should accompany each piece. ASTM A673, referenced in ASTM A992, requires only one set of three tests for every 15 tons.

The base metal CVN toughness requirement of 20 ft-lbs at 70°F is intended for connections at service temperatures above room temperature (+50°F). Lower service temperatures may require modification to this requirement. The FEMA 350 commentary indicates that no specific tests were conducted to establish this value. Rather, it "was chosen because it is usually achieved by modern steels and because steels meeting this criterion have been used in connections which have performed successfully."

The FEMA 350 commentary also notes that some tested assemblies "demonstrated base metal fractures at weld access holes and at other discontinuities such as at the ends of cover plates. In at least some of these tests, the fractures initiated in zones of low notch toughness. Tests have not been conducted to determine if higher base metal notch toughness would have reduced the incidence of such fractures." As shown in FEMA 355A Figure 2-4, there can be significant differences in toughness in different directions of applied stress.

FEMA 350 Section 3.3.2.3 discusses the phenomenon of low toughness in the K-area often associated with rotary straightening. The FEMA 350 commentary notes, "Because rolling mill practice is frequently changed, it is prudent to assume that all rolled sections are rotary-straightened." The overview of this subject given in FEMA 355A, Chapter 7, is highly recommended. It also notes in Chapter 8, that the toughness of ASTM A913 steel is not significantly different in the K-area than at the center of the web.

More specification information on this topic is presented in FEMA 353, Section 2.1.1, Supplemental Requirements for Structural Steel.

13.2. Yield Strength/Ductility

As noted in FEMA 355A Section 4.3.1, "the webs of rolled sections normally have higher yield strengths than the flanges, due to greater hot working of the thinner web material during the rolling process." Tests sponsored by SAC showed that the ratio of flange to web dynamic yield strengths were typically below 1.0 (as low as 0.95), but one set of tests averaged 1.06.

In the same SAC study, FEMA 355A reports that the ratio of laboratory dynamic yield strength to mill test report values (flange and web material tests) were as low as 0.87.

Better ductility is associated with lower yield-to-tensile strength ratios. ASTM A992 steel is required to have a yield-to-tensile ratio less than or equal to 0.85. Project-specific testing can be used to verify yield strength, ductility, and fracture toughness. For more discussion, review of Chapter 7 of the SEAOC Blue Book (SEAOC, 1999) is recommended.

14. Weld Metal Toughness

(FEMA 350 Section 3.3.2.5, with March 16, 2001 Errata)

*** 1b * This FEMA 350 recommendation is a significant change in previous practice. In addition, the SEAOC Seismology Committee recommends the following:**

1. *Electrodes for critical welds should have an AWS welding classification for CVN of 20 ft-lbs at -20°F. The electrodes should produce weld metal with CVN toughness of at least 20 ft-lbs at 0°F and 40 ft-lbs at 70°F.*
2. *Careful review of AWS 5.29 (for use with E70TG-K2 electrodes) is strongly advised. The engineer must specify toughness requirements.*
3. *More stringent toughness requirements might be necessary for service temperatures lower than 50°F.*

The FEMA 350 fracture toughness requirements for weld metal with service temperatures greater than 50°F are based on Barsom (2000) (also discussed in Barsom, 2002). Barsom notes that only limited testing was carried out in the SAC program such that “the development of fracture-toughness, within the context of a fracture control plan was not possible.” He also states that “future technical developments and an improved understanding of the factors that are integral parts of the fracture control plan for buildings, subjected to seismic loads and deformations, may modify, augment or replace the methodology and/or the proposed requirements.” Johnson et al. (2001) state, “...some consideration should be given to evaluation of the fracture toughness of weld metal using a test such as the Crack Tip Orientation Displacement Test (CTOD).”

Barsom, Barsom (2000), concludes his report by proposing FEMA 350 minimum CVN requirements for service temperatures above 50°F and stating, “This CVN requirement should preclude weld-metal fracture toughness from being a contributing factor to the fracture performance of welded moment frame connections in seismic applications. Further improvements in the fracture performance of welded moment frame connections must be achieved by changes in design detailing, fabrication and inspection. Further research is needed to define the CVN requirements for connections exposed to temperatures below +50°F.” Johnson et al. (2001) also note that further research is needed for low temperature conditions. More stringent toughness requirements should be considered on projects where service temperatures are lower than 50 °F.

The International Institute of Welding, Joint Welding Group Final Draft dated October 2, 2001 recommends CVN values of 47 joules (35 ft-lbs) to 100 joules (74 ft-lbs) at service temperature for both weld metal and parent metal, for a low risk of fracture. A final draft of the IIW report is expected in 2002.

FEMA 350 Section 3.3.2.5, together with the errata dated March 16, 2001, calls for CVN values of 20 ft-lbs at 0°F (not -20°F, corrected in the errata) and 40 ft-lbs at 70°F. FEMA 350 does not make the important distinction between filler metal and weld metal, however. CVN values for filler metal refer to the AWS welding classification in the applicable AWS electrode specification. Weld metal, however, refers to the as-welded condition, including diffusion of filler metal with base metal. It is the weld metal, not the filler metal, that must have the specified CVN properties. Most of the work done by SAC (including Barsom, 2000; Johnson et al., 2001; Ricles, et al., 2000) used E70T6 or E70TG-K2 electrodes with CVN toughness of 20 ft-lbs at -20°F as determined by AWS classification. These reports appear to form the basis of the FEMA 350 recommendation of weld metal with CVN toughness of 20 ft-lbs at 0°F and 40 ft-lbs at 70°F. Electrodes used for critical welds should therefore have an AWS welding classification for CVN of 20 ft-lbs at -20°F and should produce weld metal with CVN toughness of at least 20 ft-lbs at 0°F and 40 ft-lbs at 70°F. More specification information on this topic is presented in FEMA 353, Section 2.4.1.1, Toughness, Strength and Elongation and Appendix A, WELD METAL / WELDING PROCEDURE SPECIFICATION TOUGHNESS VERIFICATION TEST.

Requirements for E70TG-K2 electrodes are found in AWS 5.29, which has no toughness requirements. If AWS 5.29 is used, the engineer must specify toughness requirements as an agreement between the supplier and the purchaser. Several tests have used Lincoln's E70TG-K2 product, known as NR-311Ni. Lincoln certifies this product to meet the AWS requirement of 20 ft-lbs at -20°F.

15. Modified Weld Access Hole
(FEMA 350 Section 3.3.2.7)

*** 1a * This FEMA 350 recommendation is a significant change in previous practice.**

While the SEAOC Seismology Committee supports the use of the modified access hole described in FEMA 350 Section 3.3.2.7 and Figure 3-5, use of this detail without permission from the patent holder may be in violation of a U.S. patent.

The general weld access hole specified in AWS D1.1 and AISC, LRFD (1998), is intended to provide access for welding operations and reduce residual stress concentrations. The access hole recommended in FEMA 350, often referred to as the modified or improved access hole, is wider and longer than the

conventional AWS or AISC minimum access hole and has more stringent surface finish requirements. It provides greater clearance to facilitate bottom flange welding but more significantly, the improved configuration and finish requirements prove beneficial to performance.

The modified access hole recommended by FEMA derives from work by El-Tawil et al (1998) which led to testing and finite element analysis by Ricles et al. (2000). Ricles et al. considered nine weld access hole configurations on unreinforced (i.e. typical pre-Northridge) connections. Except for the case with no access hole at all, the profile shown in FEMA 350 Figure 3-5 was found to have the lowest PEEQ index (the ratio of effective strain to yield strain).

A high PEEQ index indicates greater potential for fracture under cyclic conditions. Tests indicated that unreinforced connections with notch tough weld materials frequently exhibited strain concentrations and consequent low cycle fatigue failure of the beam flange at the toe of the weld access hole. The SAC research found that the modified configuration, along with more stringent finish requirements, reduced the effects of low cycle fatigue. (Refer to Part B, Section 16 for more discussion of low cycle fatigue.)

In addition to reducing stress concentrations, the modified weld access hole may reduce shear on beam flanges. This was not a conclusion reported by Ricles et al., however, the vector diagrams that accompany the finite element analysis in that report show that the principal stress direction differs between standard and modified access holes. This difference may indicate that the shear on beam flanges is reduced for the modified weld access hole detail.

16. Low Cycle Fatigue
(FEMA 350 section 3.3.2.7)

*** 1a * This FEMA 350 recommendation is a significant change in previous practice.**

The FEMA 350 commentary cites low cycle fatigue as the cause of failure in some laboratory tests. Barsom (2000) discusses fracture "caused by the initiation and propagation of fatigue cracks" in tests of unreinforced connections performed by Goel, (1999) According to Barsom, the fatigue cracks initiated at the beam web-to-flange intersection at the weld access hole, the valleys of the flame cut weld access hole surface, the weld toe, and weld intersections.

Ricles et al. (2000) also discuss low cycle fatigue. They developed a method to predict crack initiation and extension over the life cycle of a beam-column connection using finite element analysis. The welded interface, the weld access hole, and web welds were identified as critical areas. The method was verified by tests of specimens with welded unreinforced beam flanges.

Partridge et al. (2000) reached similar conclusions regarding the importance of low cycle fatigue. They found that low cycle fatigue failure will occur in either the weld metal, the column face, or the beam web or flange at the weld access hole. The critical location depends on stress/strain concentration factors and on the cyclic response of the weld and base metals. As shown by Ricles et al. (2000), the modified access hole (see Part B, Section 15) tends to reduce stress and strain concentrations, increasing the capacity of connections to accommodate low cycle fatigue.

The modified access hole improves the performance of WUF-B, WUF-W, and RBS connections. FEMA 350 requires the modified access hole for the WUF-B and WUF-W connections, but leaves it optional for the RBS.

17. Welding Quality and Inspector Certification
(FEMA 350 Section 3.3.2.8)

*** 1a * This FEMA 350 recommendation is a significant change in previous practice.**

The FEMA 350 prequalified connections require not only specific member sizes and connection geometry but also appropriate quality assurance. FEMA 350, referring to FEMA 353, calls for more rigorous QA than was typically employed in the past. In particular, FEMA 353, Section 6.3.2, allows only AWS certified

welding inspectors (CWI) or Senior CWI to inspect welds in Demand Categories A&B (most beam-to-column welds). However, current AWS certified welding inspectors are not yet trained to ensure compliance with the FEMA 350 and 353. Furthermore, there is no similar certification program for inspection of high strength bolting, which is integral to some prequalified connections. Engineers need to ensure that qualified inspectors perform the necessary QA tasks.

18. Basis of Connection Prequalification
(FEMA 350 Section 3.4)

*** 1a * This FEMA 350 recommendation is a significant change in previous practice.**

The commentary to FEMA 350 Section 3.4 gives SAC's four criteria for connection prequalification:

"The following criteria were applied to connections listed as prequalified:

1. *There is sufficient experimental and analytical data on the connection performance to establish the likely yield mechanisms and failure modes for the connection.*
2. *Rational models for predicting the resistance associated with each mechanism and failure mode have been developed.*
3. *Given the material properties and geometry of the connection, a rational procedure can be used to estimate which mode and mechanism controls the behavior and the deformation capacity (that is, interstory drift angle) that can be attained from the controlling conditions.*
4. *Given the models and procedures, the existing data base is adequate to permit assessment of the statistical reliability of the connection."*

The SEAOC Seismology Committee concurs that these four criteria, though qualitative, are necessary and reasonably sufficient. Similar acceptance criteria are used by jurisdictional agencies to evaluate and prequalify steel moment frame connection designs other than those presented in FEMA 350.

An understanding of the parameters that influence connection performance is fundamental to the engineering of a seismic force resisting system. Even for prequalified connections, engineers are strongly advised to review original test reports to verify that the chosen connection will provide performance and reliability appropriate to the larger project. FEMA 355D, which was prepared to support the prequalification criteria in FEMA 350, is recommended as a first reference. It summarizes and discusses the SAC Phase 2 testing.

FEMA 350 lists seven connection types as prequalified for use as fully restrained connections in special moment frames. As described below, the SEAOC Seismology Committee, based on the investigations of its FEMA 350 Task Group, finds that the following connection types should not be considered prequalified for special moment frames for the full range of parameters allowed by FEMA 350:

- Welded Unreinforced Flanges - Welded Web (WUF-W).
- Free Flange (FF).
- Welded Flange Plate (WFP).
- Bolted Flange Plate (BFP).

The rationale for this finding is presented below for each connection type. The remaining three connection types have not yet been reviewed in sufficient depth to reach conclusions.

19. Prequalified Fully Restrained Connections
(FEMA 350 Sections 3.5 and 3.6)

FEMA 350 prequalifies nine connection types for use in moment resisting frames. Seven of these are prequalified for use in SMF's and are considered fully restrained by FEMA 350. This SEAOC Seismology Committee document addresses the question of prequalification for four of them: Welded Unreinforced Flange—Welded Web (WUF-W), Free Flange (FF), Welded Flange Plate (WFP), and Bolted Flange Plate (BFP). For WUF-W, BFP, and WFP, the independent detailed analyses of test data on which the Task Group relied are described in Appendices C, D, and E.

Prequalification of Reduced Beam Section (RBS), Bolted Unstiffened End Plate (BUPE), and Bolted Stiffened End Plate (BSEP) is discussed, however, the comments are not at this time based upon a detailed analysis and may be considered preliminary pending further study.

As noted below, it is the SEAOC Seismology Committee's position that some connection designs deemed prequalified by FEMA 350 should still be qualified by specific tests. In some cases, a small number of additional tests of critical conditions might justify prequalification in the future.

19.1. Welded Unreinforced Flanges—Welded Web (WUF-W)
(FEMA 350 Section 3.5.2)

*** 3 *** *The SEAOC Seismology Committee's position or conclusions vary from FEMA 350. The SEAOC Seismology Committee recommends:*

1. *This connection type should not be used as prequalified for SMF systems. Design of SMF systems with this connection type should be based on existing or new test results in accordance with FEMA 350 Section 3.9 or section 9 and Appendix S of the AISC Seismic Provisions (AISC, 1997 & 2000).*
2. *For OMF systems, the following are recommended:*
 - A. *Beam sizes should be such that $b/2t_f$ and h/t_w are not significantly less than for a W36x150 ($b/2t_f = 6.4$, $h/t_w = 52.0$). Reasonable recommended values are $b/2t_f = 5.9$ and $h/t_w = 49.0$.*
 - B. *Panel zone strength (i.e. thickness) should be greater than required by FEMA 350 Section 3.3.3.2. A reasonable recommended panel zone total thickness is 1.4 times the thickness required by FEMA 350 Section 3.3.3.2.*
 - C. *Use the inelastic drift limits given in Part B, Section 20.*
3. *When checking column-beam moment ratios, for example per equation 9-3 of the AISC Seismic Provisions (1997), the beam moments M_{pb} should be increased. A reasonable recommended factor for this increase is 1.4.*

The WUF-W is similar to typical pre-Northridge connections, but its ductility has been improved by the use of notch-tough welding electrodes, a welded web-to-column connection, and a modified weld access hole. Electrode toughness is discussed above in section 14. The modified access hole configuration is shown in FEMA 350 Figure 5 and discussed above in section 15. It is elongated relative to the configuration shown in the ASD "Specification for Structural Steel Buildings," Figure C-J1.2.a (in AISC, 1989). It also includes surface finish requirements. The elongated hole tends to reduce stress concentrations as well as the amount of shear carried by the beam's flanges. The stiffer welded web connection absorbs more of the shear force than a bolted shear plate and also transfers considerable moment to the face of the column.

The FEMA 350 design procedure for this connection consists only of a calculation of required panel zone strength and a check for continuity plates. Detailing requirements for this connection are otherwise prescriptive.

Appendix C describes the WUF-W tests performed by SAC researchers and presents the findings of an independent analysis of reported results.

Substantial testing of this connection type has been completed for only one beam size. It is the SEAOC Seismology Committee position that the available test results do not justify SMF prequalification of all beam sizes "W36 and shallower." With reference to the FEMA 350 prequalification criteria (FEMA 350 Section 3.4, discussed above in section 18), rational models are not yet in place to predict each potential failure mode or a controlling mechanism. Additional testing with a wider range of member sizes might justify prequalification in the future.

Without prequalification, connection designs for SMF systems should be based on existing or new test results in accordance with appropriate sections of FEMA 350 or the AISC Seismic Provisions.

As noted in Conclusion 2 of Appendix C, the performance of W36x150 test specimens relied on substantial flange and web buckling. Therefore, similar width thickness ratios are recommended when this connection type is used as prequalified for OMF systems.

As noted in Conclusion 3 of Appendix C, the test specimens had doubler plates substantially thicker than those required by FEMA 350 Section 3.3.3.2. Specimens with thinner doubler plates might have experienced significant, and possibly detrimental, panel zone yielding. Therefore, similar panel zone relative strengths are recommended when this connection type is used as prequalified for OMF systems.

As noted in Conclusion 5 of Appendix C, this connection type is still likely to impose high demands on the column and joint. Therefore, if this connection type is used, a factored (increased) beam moment is recommended for checking column-beam moment ratios using, for example, the AISC Seismic Provisions. The value of 1.4 is based on the test results considered by SAC and discussed in Appendix C.

19.2. Free Flange Connection (FF) (FEMA 350 Section 3.5.3)

* 3 * *The SEAOC Seismology Committee's position or conclusions vary from FEMA 350. The SEAOC Seismology Committee recommends:*

1. *This connection type should not be used as prequalified for SMF or OMF systems. Design of SMF and OMF systems with this connection type should be based on existing or new test results in accordance with FEMA 350 Section 3.9 or section 9 and Appendix S of the AISC Seismic Provisions (1997 & 2000).*

2. *Replace FEMA 350, Section, 3.5.3.1 Equation 3-11 with the following:*

$$T_{st} = \frac{M_f - R_y \cdot F_{yb} \cdot t_{fb} \cdot b_{fb} \cdot (d - t_{fb})}{(d_b - 4t_{fb} - 2t_{fb} - d_b / 4)}$$

This change restores principles of mechanics by properly accounting for distance between the assumed T/C couple.

In typical pre-Northridge connections, substantial portions of the beam shear force are transferred to the column through the beam flanges (see Section 3 above). The Free Flange connection was developed as an attempt to reduce the stiffness and restraint of the beam flanges, thereby reducing local strains in the beam flanges, limiting the amount of beam shear they carry, and reducing the potential for beam flange fracture.

For SMF systems, FEMA 350 Table 3-4 limits the beam depth to W30 members and the beam flange thickness to 3/4". Referring to Table 3-6 in FEMA 355D, the prequalification is apparently based on a total of seven tests. Five of these tests were performed at the University of Michigan (Choi et al.): one with a W24x68 beam, two with W30x99 beams, and two with W30x124 beams. Gilton et al. (2000b) performed one test with a W36x150 beam. Venti and Engelhardt (2000) tested a two-sided specimen with a composite floor slab. FEMA 355D Section 7.3 recognizes that the FF connection had limited testing in the SAC Phase 2 program.

In four of the specimens, inelastic panel zones contributed significantly to the response. Had the panel zones not yielded (because of higher strength steel, perhaps), these specimens might not have achieved the rotations that they did. Plastic rotations in three of the specimens were less than 3%. In two of the tests (Michigan 8.2 and 9.2), the beam yield stresses were relatively low (as low as 40.8 ksi in the beam flange). Also, the W24x68 and W30x99 members are non-compact with grade 50 steel and are therefore not allowed for SMF systems.

It is the SEAOC Seismology Committee position that the available test results do not justify either SMF or OMF prequalification for this connection type. With reference to the FEMA 350 prequalification criteria (FEMA 350 Section 3.4, discussed above in section 18), there is not sufficient data to establish yield mechanisms or to assess statistical reliability. The connection appears to have merit, but until additional testing is complete, its design should be based on existing or new test results in accordance with appropriate sections of FEMA 350 or the AISC Seismic Provisions.

With regard to the design procedure in FEMA 350 Section 3.5.3.1, Equation 3-11, which determines the beam flange tension applied normal to the column flange, does not appear to satisfy the principles of mechanics. A recommended alternative is given above. In an unpublished paper, S. Goel, B. Stojadinovic and J. Choi, Goel et al (2001 draft) present a design procedure different from the one in FEMA 355D. They address the flawed FEMA 350 Equation 3-11 and suggest other revisions to reconcile finite element analyses with test results. Their revisions account for realistic strain hardening and modify the assumed moment arm between the tension-compression couple in the beam web, reducing the moment demand on the shear plate.

Finally, Step 8 of the design procedure advises that the weld group attaching the shear plate to the beam web should be designed "based on the principles of mechanics." Using elastic properties of the weld group can result in very large fillet welds. Using plastic properties for the weld group, though more liberal, can still result in fillet welds larger than the beam web thickness. This requires further study.

19.3. Welded Flange Plate (WFP) (FEMA 350 Section 3.5.4)

*** 3 *** *The SEAOC Seismology Committee's position or conclusions vary from FEMA 350. The SEAOC Seismology Committee recommends:*

1. *This connection type should not be used as prequalified for SMF systems. Design of SMF systems with this connection type should be based on existing or new test results in accordance with FEMA 350 Section 3.9 or section 9 and Appendix S of the AISC Seismic Provisions (1997 & 2000).*
2. *For OMF systems, the following are recommended:*
 - A. *In place of FEMA 350 Equation 3-13, use the following:*

$$t_p = \frac{M_f}{R_y \cdot F_{yp} \cdot b_p \cdot \left(d_b + \frac{(t_{plb} + t_{plf})}{2} \right)}$$

B In place of FEMA 350 Equation 3-14, use the following:

$$I_w \cdot t_w = \frac{M_f}{0.707 \cdot F_w \cdot d_b}$$

As printed in FEMA 350, Equation 3-14 omitted the term d_b .

C The quantity $(M_f - M_w)$ may be substituted for M_f in equations 3-13 and 3-14, as modified above. The value M_w , the beam web flexural capacity, determined from an elastic stress distribution, at the face of the column may be taken as follows:

$$M_w = \frac{M_w \cdot I_{web}}{I_{(flange_plates+web)}}$$

Where:

$$I_{web} = \frac{(d_b - 2k - 2")^3 \cdot t_w}{12}$$

$$I_{(flange_plates+web)} = \frac{2 \cdot b_{fp} \cdot t_{fp}^3}{12} + 2 \cdot t_{fp} \cdot b_{fp} \cdot (d_b + t_{fp}) + I_{web}$$

d_b = depth of beam

t_w = thickness of beam web

k = distance from outer face of flange to web of toe of fillet

b_{fp} = width of flange plate

t_{fp} = thickness of flange plate

This method of estimating M_w assumes a CJP welded connection (beam web to column flange) as recommended below.

D. FEMA 350 Section 3.5.4.1, per the March 16, 2001 Errata, the text under "Step 6" should conclude as follows:

$$"... d_b + \frac{(t_{plt} + t_{plb})}{2} \text{ for } d_b - t_{fb}."$$

E. Use the inelastic drift limits given in Part B, Section 20.

3. Use a complete joint penetration groove weld (CJP), with shear plate dimensions as described in FEMA 350 Figure 3-11, to attach the beam web to the column flange. Using the FEMA 350 recommended details for WUF-W as a basis, it is recommended that the CJP weld (QA/QC category BH/T), using run-off tabs and backing bars, be applied for full length of shear plate plus 1/2 to 1 inch at each end. After welding, weld tabs and backing bars should be removed and the ends of welds ground smooth with a smooth transition to base metal.

4. Use of beam sections significantly more compact than the tested sections should be avoided until appropriate tests demonstrate that the more compact sections can achieve qualifying rotation through significant flange and web local buckling.

This connection involves flange plates attached to the column with CJP welds and to the beam flange with fillet welds. The beam flange does not attach directly to the column. As described in FEMA 355D, the principal yield mechanisms are yielding of the panel zone and flexural yielding of the beam. Typical failure modes include tensile fracture of the flange plate or CJP weld, fracture of the fillet weld between the beam and plate, and local and lateral torsional buckling of the beam. Tests showed a tendency for stress concentrations to occur away from the column, at the ends of the fillet welds that attach the plate to the beam.

FEMA 350 Table 3-5 limits SMF beams to "W36 and shallower" with a maximum flange thickness of 1 inch. These limits allow members substantially larger than those tested. One test (UCB-RC09), Kim, et al (2000) designed to provide balanced yielding of the beam, panel zone, and flange plate, did display significant panel zone yielding. However, the design procedure puts no upper limit on panel zone strength to control the relative yielding of the beam and panel zone. (By contrast, Step 3 of the design procedure for Bolted Flange Plate connections recommends upper limits on panel zone strength. See FEMA 350 Section 3.6.3.1.)

Appendix D describes the WFP tests performed by SAC researchers and presents the findings of an independent analysis of reported results.

FEMA 355D Table 3-19 lists five tests, all using W30x99 beams. This section is non-compact for grade 50 steel and is not permitted in SMF systems. Though this section is only slightly outside of the compactness requirements, since qualifying significant rotation was achieved through beam flange and web buckling, the ability of heavier sections to provide this mode of rotation remains unknown based upon the SAC tests.

All five specimens showed degradation after peak load was achieved. In four tests, the degraded moment capacity at 4% total story drift was only 30 to 60 percent of the nominal plastic capacity. These results fail the criteria of the AISC Seismic Provisions, but might satisfy FEMA 350. (Refer to Section 23 of this document for further discussion.) The one test specimen (UCB-RC09) whose panel zone was weaker than FEMA 350 Section 3.3.3.2 would require retained 83 percent of its nominal plastic capacity.

It is the SEAOC Seismology Committee position that the available test results do not justify prequalification for use in SMF systems. With reference to the FEMA 350 prequalification criteria (FEMA 350 Section 3.4, discussed above in section 18), rational models are not yet in place to predict each potential failure mode and the number of relevant available tests is not yet sufficient to assess statistical reliability.

Without prequalification, connection designs for SMF systems should be based on test results in accordance with appropriate sections of FEMA 350 or the AISC Seismic Provisions. Existing test results may be used, but as noted above, the SAC test results did not meet the AISC criteria.

Further, it is the SEAOC Seismology Committee position that certain of the FEMA 350 design equations should be replaced with the more logical alternatives given above. In FEMA 350 Equation 3-13, use of the value M_{yf} appears illogical and inconsistent with the flange plate weld design. Also, analysis of SAC tests suggests that the web connection contributes moment transfer to column and should be accounted for in the connection design. For this reason, the quantity $(M_f - M_w)$ may be substituted for M_f in the modified equations 3-13 and 3-14 recommended above. The value M_w is the beam web flexural capacity determined from an elastic stress distribution at the face of the column. Since the available test data suggest that the contribution of the web connection to moment resistance may be significant, it is reasonable to account for that contribution when sizing the flange plates.

FEMA 350 Equation 3-14 is incorrect and has not been addressed in the Errata dated March 16, 2001. In its incorrect form, the equation's units are inconsistent. The moment in the numerator should be divided by the beam depth to yield an approximate flange force.

Figure 3-11 provides details of web connection involving a prescriptive shear plate, partial joint penetration groove welds and fillet welds. The SAC project connection test reference document, Kim et al (2000), does not indicate welding details used in the tested connections. However, the commentary in FEMA 350 Section 3.5.4 states that complete joint penetration groove welds (CJP) were used in the tested connections. Therefore, a CJP groove weld between the beam web and the column is recommended for consistency with the commentary and other connection types with proven enhanced performance. FEMA 350 does not provide specific information about CJP welds for this connection.

19.4. Bolted Flange Plate (BFP)
(FEMA 350 Section 3.6.3)

*** 3 *** *The SEAOC Seismology Committee's position or conclusions vary from FEMA 350. The SEAOC Seismology Committee recommends:*

1. *This connection type should not be used as prequalified for SMF systems. Design of SMF systems with this connection type should be based on existing or new test results in accordance with FEMA 350 Section 3.9 or section 9 and Appendix S of the AISC Seismic Provisions (1997 & 2000).*

2. *If oversized holes are provided per FEMA 350 Figure 3-17, the connection should be considered Partially Restrained. Significant deformation can occur before the onset of beam yielding.*

3. *For OMF systems, the following are recommended:*

A. *The FEMA 350 Section 3.6.3.1 design equations may be modified appropriately. Reasonable alternatives, consistent with available test results, are as follows:*

$$M_{fail,bolts} = 2 \cdot N \cdot A_b \cdot (F_{vbolt}) \cdot d_b \quad (\text{Alternative 3-43})$$

$$M_{fail,FP} = 0.85 \cdot F_{upl} \cdot (b_p - 2(d_{bthole} + 0.062) \cdot t_{pl} \cdot (d_b + t_{pl})) \quad (\text{Alternative 3-45})$$

In lieu of FEMA 350 Equation 3-47, satisfy the following at the row of bolts farthest from the column:

$$\left(\frac{Z_b - 2(d_{bthole} + 0.062) \cdot t_{fb} \cdot (d_b - t_{fb})}{Z_b} \right) \geq 0.75$$

B. *Use the inelastic drift limits given in Part B, Section 20.*

4. *Design of bolted flange plate connections should include verification of bolts against slip under service load conditions. Check bolt slip for UBC 97 Equation (12-13) (ASD - Seismic or Wind):*

$$N = M_{fasd} / ((d + t_{pl}) \cdot 2 \cdot B_{sf})$$

Where B_{sf} = Bolt slip critical allowable load
Note: 1/3rd increase is permitted.

M_{fasd} = Moment at face of column due to UBC 97 Code ASD combination (12 - 13)

FEMA 355D Section 5.3.2 cites pre-Northridge use of the BFP connection as well as early testing by Popov and Pinkney (1969) and Harriott and Astaneh (1990). This connection offers simplicity in construction and is relatively economical.

FEMA 355D Section 5.3.2 notes that BFP connections "...may have significant energy dissipation and rotation capacity...or they may have very limited energy dissipation with small inelastic rotations..." It further states, "Net section fracture of the beam or flange-plate or fracture of the flange weld appear to be common modes of failure. These modes of failure can be brittle with limited inelastic deformation capacity unless they are delayed while plastic rotation occurs at other locations."

FEMA 355D Section 5.3.2 continues, "Yield mechanisms include flexural yielding of the beam, tensile yield of the flange plate and shear yielding of the panel zone of the column." FEMA 350 summarizes the preferred failure mode: "the best inelastic behavior is achieved with balanced yielding in all of the three preferred mechanisms: beam flexure, cover plate extension and compression, and panel zone yielding."

FEMA 355D Section 7.3 recognizes that "...the models for predicting [bolted] connection performance and balancing connection behavior are not as well defined as the models used for welded-flange connections, and they also are more complex. Further research into the seismic performance of bolted connections is desirable in fully understanding the yield mechanisms and failure modes of these connections as well as balancing the connection performance to achieve maximum ductility from the connections."

Appendix E of this document describes the BFP tests performed by SAC researchers and presents the findings of an independent analysis of reported results.

FEMA 355D Table 5.5 lists 8 tests, six with W24x68 beams and two with W30x99 beams. These sections are non-compact for grade 50 steel and are therefore not allowed for use in SMF systems.

Panel zone yielding contributed substantially to the total rotation achieved in most tests. No doubler plates were provided in seven of the eight specimens. In the other specimen, a doubler plate was added during a second stage of testing. Had the test specimens been designed for the panel zone requirements of FEMA 350 Section 3.3.3.2, doubler plates would have been required. With doubler plates it is unlikely that any significant panel zone rotation would have taken place. Without significant panel zone rotations, some of the specimens might not have met the criteria of FEMA 350 Table 3.15.

Good performance of BFP connections requires nearly simultaneous yielding of the panel zone, the connection plate, and the beam. With variable steel properties, however— F_y of the column panel zone may range from 50 to 65 ksi, for example—significant inelastic contributions from each mechanism can not be assured. Beam material properties are also important. The beam steel in the test specimens had good yield to ultimate tensile stress ratios (0.74 and 0.79). Ratios closer to the allowable limit of 0.85 would be less likely to provide good performance at the net section through the bolt holes and could fracture at lower drifts than those achieved in the tests.

The oversized holes required by FEMA 350 raise two concerns. First, gravity load conditions, moderate earthquakes and perhaps wind load might produce permanent frame displacement. This seems especially important because bolt-slip occurred in the tests at less than 40 percent of peak load. A service load check of bolts against slip is recommended. Second, construction tolerances are such that the bolts might not engage evenly, leading to early failure of some bolts. Full bearing values have been used for bolts in oversized holes, which is inconsistent with AISC ASD and LRFD specifications. The justification for use of oversized holes appears based upon only a limited number of tests (Refer to Appendix E, Summary Item 5).

Simultaneous satisfaction of the design equations in FEMA 350 Section 3.6.3.1 is rarely possible, with net section fracture nearly always controlling. The formulas for evaluating shear failure of the bolts (FEMA 350 Equation 3-43) and net section fracture of the flange plate (Equation 3-45) and beam flange (Equation 3-47) are not consistent with principles of mechanics since force is constant between the

column face and the first line of bolts. For this reason, the modifications recommended above omit the increase in moment capacity to face of column (length ratios L_{TF1} , L_{TF2} , and L_{TF3}). The beam flange net section fracture criterion is further modified to allow the ratio $Z_{net} F_u / Z_{gross} F_y$ to be greater than 0.75 instead of 0.85. The 0.75 value is based on the use of OMFs with limited yield strain and tests by Schneider and Teeraparbwong (1999). It appears that bolt friction might marginally reduce net section force. Also, steel currently produced to meet ASTM 992 typically has F_y / F_u ratios less than 0.85. Project specific testing to establish F_y / F_u ratios is recommended.

The commentary to FEMA 350 Section 3.7 classifies connections as Partially Restrained "if the deformation of the connection itself will increase the calculated drift of the frame by more than 10%." In the tests, the drift due to bolt-slip in oversized holes was approximately 0.5%, which is about 13% of the 4% story drift requirement. Thus the BFP connection does not appear to meet the FEMA 350 criteria for Fully Restrained Connections.

Based on these findings and concerns, it is the SEAOC Seismology Committee position that the available test results do not justify prequalification of BFP connections for SMF systems. With reference to the FEMA 350 prequalification criteria (FEMA 350 Section 3.4, discussed above in section 18), rational models are not yet in place to predict each potential failure mode or a controlling mechanism.

Without prequalification, connection designs for SMF systems should be based on existing or new test results in accordance with appropriate sections of FEMA 350 or the AISC Seismic Provisions.

Despite this SEAOC Seismology Committee position, the work by Schneider and Teeraparbwong (1999) is encouraging and suggests value in further testing of this connection type. Additional research should address potential brittle failure modes in the beam flange at the last line of bolts (that is, furthest from the column), at the welded attachment of the flange plate to the column, and in the flange plate at the line of bolts adjacent to the column. The following suggestions along these lines might also be considered:

- A small reduced section in front of the flange plates (that is, toward the beam midspan).
- Elongated holes that allow ductile stress flow in front of the flange plate.
- Plates welded to the beam flanges (under the top flange and above the bottom flange).

BFP connections may be suitable for OMF systems in which inelastic story drifts are limited (to be consistent with qualification test rotation). In these conditions, the modifications given above to the FEMA 350 design equations may be appropriate.

19.5 Bolted Unstiffened End Plate (BU EP) and Bolted Stiffened End Plate (BSEP) (FEMA 350 Sections 3.6.1 and 3.6.2)

*** 1b * This FEMA 350 recommendation is a significant change in previous practice. In addition, the SEAOC Seismology Committee recommends the following:**

1. *This connection type should not be used as prequalified for SMF systems. Design of SMF systems with this connection type should be based on existing or new test results in accordance with FEMA 350 Section 3.9 or section 9 and Appendix S of the AISC Seismic Provisions (1997 & 2000).*
2. *For OMFs, use the inelastic drift limits given in Part B, Section 20.*
3. *These connections rely on panel zone yielding. Engineers are advised to examine applicable BSEP test results to determine the degree to which design conditions match the tested conditions, including panel zone, on which the empirical FEMA 350 design equations are based.*

4. *Engineers should check the net section of the column at the bolt line against applicable AISC provisions.*

5. *It is suggested that final designs, using what appear to be curve formulae given in FEMA 350 be independently verified using principles of mechanics.*

The SEAOC Seismology Committee has not yet performed an independent analysis of BUPEP or BSEP connection tests. The recommendations above should be considered preliminary.

These connections rely on panel zone yielding for substantial portions of their energy dissipation capacity. If member sizes preclude panel zone yielding, the overall performance of the assembly might be different from what tests have predicted. (See above for similar discussions of this topic for other connection types.) Owing to this reliance on panel zone yielding to achieve SMF qualification rotation, it is the recommendation of the SEAOC Seismology Committee to not consider these connections as prequalified SMF connections. With appropriately adjusted inelastic drift limits, they may be used as OMF's.

The FEMA 350 design procedures do not mention a net section check in the columns at the bolt line, which may be controlling. Net section fracture should be avoided because of its abrupt nature.

The FEMA 350 design procedure for BSEP connections uses empirical formulae based on curve fitting. The procedure is therefore not as transparent as a rational method based on principles of statics and structural mechanics. Therefore, use of principles of mechanics to verify final designs based on FEMA 350 equations is recommended. Also, some consideration should be given as to whether the actual design is similar to the tested conditions (SAC prequalification tests and others as available).

19.6 Reduced Beam Section (RBS) (FEMA 350 Section 3.5.5)

*** 1a * *This FEMA 350 recommendation is a significant change in previous practice.***

The SEAOC Seismology Committee has not yet performed an independent analysis of RBS connection tests. Over 70 relevant tests of RBS connections were performed by the SAC Phase 2 project or considered by the writers of FEMA 355D and FEMA 350. A thorough independent review will therefore be difficult and time-consuming. However, the large body of mostly successful test data indicates that the RBS connection is likely one of the most reliable connection types prequalified by FEMA 350.

Preliminary comments concerning the use of RBS connections are:

1. Moore et al. (1999) presents RBS design procedures that some engineers may have used prior to publication of FEMA 350. Engineers should now recognize that there are some differences between FEMA 350 and Moore et al. Differences include the following:
 - A. Evaluation of girder shear at the joint since FEMA 350 includes the C_{pr} factor, that accounts for the peak connection strength, in the determination of M_{pr} Moore et al does not include the C_{pr} factor. The C_{pr} factor results in a higher shear value when using FEMA 350 than determined using Moore et al's recommendations.
 - B. The lower beam shear, determined in Moore et al's Steel Tips, also results in lower column moment demands than that determined by FEMA 350.
 - C. Moore et al refers to FEMA 267A for the Strong Column weak beam ratio, and Panel Zone evaluation. These have been revised in FEMA 350 from the recommendations given in FEMA 267A.

- D. Moore et al requires use of continuity plates equal to thickness of beam flanges. FEMA 350 provides a continuity plate design method and continuity plates may not be required.
2. The FEMA 350 design procedures do not include a check of the beam at the reduced section under gravity or wind loads. In most situations this is not a controlling condition. However, heavily loaded beams can overstress the hinge or shift the hinge away from the center of the reduced section. Light buildings with large projected areas that increase wind loads might control design of the reduced section.
 3. Refer to FEMA 350 Section 3.5.5.1, Step 2, item d): The FEMA 350 Errata of March 16, 2001 correctly remove the term C_{pr} . The sentence should read, "If $M_f < R_y Z_b F_y$ the design is acceptable."
 4. While RBS connections might not require lateral torsional bracing at the hinge in order to meet acceptance criteria for SMF systems, lateral bracing is expected to improve overall performance of the connection and behavior of the assembly as it approaches its rotational capacity. This may be warranted for projects that require a better than code minimum level of performance.
 5. Connection qualification tests demonstrate that significant lateral distortion of the lower flange can occur. If the distortion is large enough, it might be harmful to the building enclosure or to adjacent nonstructural components. Bracing is expected to control this distortion. (See Section 11.)
 6. Based on evaluation of other connection types, attachment of the beam web to the column with a complete penetration weld is expected to improve rotational capacity. The welded attachment should be considered on projects that warrant a higher than code minimum level of performance.
 7. The RBS connection is prequalified with either the AWS/AISC standard weld access hole or the modified weld access hole shown in FEMA 350 Figure 3-5. It is likely that the modified weld access hole will improve this connection's performance.

20. Application to IMF and OMF Systems
(FEMA 350 Sections 3.5, 3.6, 3.7, 3.9.2, and 4.6.2)

*** 1b * This FEMA 350 recommendation is a significant change in previous practice. In addition, the SEAOC Seismology Committee recommends the following:**

1. Allowable story drifts for OMFs should be significantly smaller than allowable story drifts for SMFs. Limits equal to one half of the SMF allowable appear appropriate, based upon qualifying interstory drift angles.

A greater drift may be permitted if it can be demonstrated from tests, conforming to AISC Seismic 97 App. S. In any case, drift should not exceed that permitted by the Building Code.

FEMA 350 Table 3-15 gives minimum qualifying total interstory drift angles. The required capacities of an OMF system are half of those required for SMFs. Given identical performance objectives, this suggests that a properly designed OMF will have the strength and stiffness necessary to withstand half the drift experienced by an SMF. Unpublished studies by Hale (1999), however, showed that interstory drifts and plastic rotation demands on the joints are nearly the same for SMFs and OMFs designed by the 2000 IBC and the 1997 UBC. The primary reason is that drift typically controls frame design. Even with R values of 4 and 8 (for the OMF and SMF, respectively), drift controls the design in both cases, and frame member sizes are roughly the same for both systems. The R factor for the OMF is not low enough to provide sufficient member stiffness to produce corresponding reductions in connection rotation demand. The

result is that OMF connections should experience approximately the same connection rotation as SMF connections. Therefore, the different qualification criteria in FEMA 350 do not appear rational. Code-based design might result in more connection damage in an OMF than in an SMF, and the OMF damage might exceed expectations of both the code and FEMA 350.

FEMA 350 design requirements for the OMF are approximately equivalent to the IMF requirements in AISC's Supplement No. 2 (AISC, 2000). Both require qualification tests with the same rotational capacity. FEMA 350 also has an acceptance criterion for ultimate drift angle capacity. Since definitions change, engineers should verify that the intended connection and system performance match those of the governing code. Supplement No. 2 redefined the IMF to be similar to the prior OMF, with tested connections and the joint inelastic rotation requirement of 0.01 radians. The OMF was revised to have a prescriptive connection with no requirements for qualification testing. OMF use in Supplement No. 2 is restricted to use with light framed construction with dead loads not exceeding 15 psf for walls, floors, and roofs.

For further discussion, see Appendix B.

21. Welding Parameters and Categories (FEMA 350 Sections 3.5 and 3.6)

*** 1b *** *This FEMA 350 recommendation is a significant change in previous practice. In addition, the SEAOC Seismology Committee recommends the following: For SMF systems, use QC/QA category BH/T for the weld(s) connecting the shear plate to the column (e.g. WUF-B, F.F, WFP, RBS, BFP) and welds directly connecting beam web to column (eg. WUF-W). Where the shear plate is also welded to the web, in an SMF system, the QC/QA category for the shear plate-to-web welds should be BH/L*

Welding parameters specified in FEMA 350 for Prequalified Connections vary from connection to connection. In some cases, FEMA 350 Sections 3.3.2.4, 3.3.2.5, and 3.3.2.6 are referenced. In other cases (such as the WFP), only 3.3.2.4 is referenced.

As explained in FEMA 350 Section 3.3.2.8, QC/QA procedures (given in FEMA 353) vary according to the weld's seismic demand, consequence, and primary loading direction. The Prequalified Bolted Fully Restrained Connections appear to have consistent welding category requirements. However, the FEMA 353 welding categories for web shear plates in Prequalified Welded Fully Restrained Connections vary from BL/T (medium demand, low consequence) to BH/T (medium demand, high consequence). FEMA 350 does not explain the differences.

22. Connection Details at the Roof (FEMA 350 Sections 3.5, 3.6, and 3.7)

*** 2 *** *The SEAOC Seismology Committee recommends additional considerations (revisions) as follows: Acceptable performance may be reasonably expected from either of two details at the top of a frame column:*

1. *Extend the column beyond the beam top of steel by at least three inches.*
2. *Use a cap plate on the column, vertically aligned with the beam top flange. The attachment of the cap plate to the column should be sufficient to develop the beam flange force M_p .*

FEMA 350 suggests no details for prequalified connections at the roof or uppermost floor of a frame. Reasonable recommendations, none of which have been tested, are given above. Future editions of the AISC Seismic Provisions are expected to address these conditions.

23. Testing Procedures and Acceptance Criteria
(FEMA 350 Sections 3.9.1 and 3.9.2)

- * 1b *** *This FEMA 350 recommendation is a significant change in previous practice. In addition, the SEAOC Seismology Committee recommends the following: Acceptance Criteria in the AISC Seismic Provisions are different in some respects from those in FEMA 350. Where better than minimum code level performance level is desired, engineers should consider requiring a minimum ultimate drift capacity, θ_U , as recommended in FEMA 350.*

FEMA 350 Table 3-13 defines θ_{SD} as the rotation “at which either failure of the connection occurs or the strength of the connection degrades to less than the nominal plastic capacity, whichever is less.” Table 3-13 also defines the ultimate drift angle θ_U as the rotation “at which the connection damage is so severe that continued ability to remain stable under gravity loading is uncertain.” The term “failure” is not defined in the context of Table 3-13, but the note under Table 3-14 uses the same term in setting a degradation limit: “Failure shall be deemed to occur when the peak loading in a cycle falls to 20% of that obtained at maximum load or, if the assembly has degraded, to a state at which stability under gravity load becomes uncertain.” It is the SEAOC Seismology Committee’s understanding that the definition following Table 3-14 applies only to θ_U and not to θ_{SD} . Rather, the “strength degradation” limit represents the onset of degradation, so no degradation should have occurred before the required rotation is achieved.

With respect to acceptance criteria, FEMA 350 Table 3-15 sets required capacities for OMF and SMF systems in terms of both θ_{SD} and θ_U . FEMA 355D reports that several tests of prequalified connections achieved θ_U values significantly less than the required SMF capacity of 0.06 radians. In some cases, the measured values were limited by testing apparatus, not by failure of the connection. Also, some tests of prequalified connections did not satisfy the maximum degradation limit, their strengths falling to less than 20% of those obtained at maximum load.

Other standards and reference documents have used different acceptance criteria. Older tests may have been performed to obsolete criteria, and the engineer might have to translate older test results into the newer terminology.

The 1997 AISC Seismic Provisions required an inelastic rotation capacity of 0.03 radians. In Supplement No. 2 (2000), that provision has been translated into a requirement for interstory drift angle capacity of 0.04 radians. This assumes a typical value of elastic rotation equal to 0.01 radians but this will vary with connection configuration. As for degradation, the 1997 AISC Seismic Provisions (section 9.2b) require that a certain beam strength be retained when the qualifying drift angle is achieved. For beams that hinge adjacent to the column face, the flexural strength at the column face must equal the nominal plastic moment of the beam. For RBS connections or those exhibiting beam local buckling, the strength at the column must be at least 80 percent of the beam’s nominal plastic moment. AISC does not specify an ultimate “post-degradation” drift capacity similar to FEMA 350’s value of 0.06 radians.

For reference, section C703.4 of the 1999 Blue Book (SEAOC, 1999) recommends determining a test specimen’s capacity as “the maximum deformation at which two cycles are completed and the strength remains above both of the following levels.

- 85 percent of the specimen design strength, considering measured rather than nominal yield strength of the materials, but ignoring strain hardening effects.
- 70 percent of the peak tested specimen strength.”

The Blue Book does not recommend an ultimate drift capacity.

24. Prequalification Testing Criteria
(FEMA 350 Section 3.10)

Notwithstanding the recommendations of FEMA 350 Sections 3.9 and 3.10, a jurisdictional authority may develop its own procedures and acceptance criteria for evaluation and qualification of a given connection or frame design. Examples include:

1. Los Angeles County Technical Advisory Panel (LACO-TAP), Department of Public Works, in accordance with County of Los Angeles Current Position on Design and Construction of Welded Moment Resisting Frame Systems CP-2, dated August 14, 1996.
2. ICBO Evaluation Service, Inc., in accordance with ICBO ES Acceptance Criteria for Qualification of Steel Moment Frame Connection Systems (AC 129-R1-0797) and AISC Seismic Provisions for Structural Steel Buildings (1997).
3. City of Los Angeles Engineering Research Section, which invokes the qualification procedures contained in FEMA 267, FEMA 267A, County of Los Angeles Current Position on Design and Construction of Welded Moment Resisting Frame Systems CP-2, and AISC Seismic Provisions for Structural Steel Buildings (1997).

25. Immediate Occupancy Performance Level Damage
(FEMA 350 Section 4.2.2)

** 1b * This FEMA 350 recommendation is a significant change in previous practice. In addition, the SEAOC Seismology Committee recommends the following: Owners, building officials, and engineers are advised to evaluate the Immediate Occupancy performance implied by FEMA 350 and to define performance objectives that suit particular projects.*

Immediate Occupancy is defined in different ways by different documents and by different parts of FEMA 350. Absent consistent criteria, Immediate Occupancy performance should be defined on a building-specific basis, recognizing the general intent of various guidelines documents, including FEMA 350. FEMA 350 Appendix A provides generalized and detailed evaluation procedures that may be helpful in this regard.

FEMA 350 Table 4-2 suggests that a building can perform at the Immediate Occupancy level even with 10% of its frame connections "fractured." (It is reasonable to interpret this to mean fractures of beam flanges or beam flange welds only. In the Northridge earthquake, more serious fractures of the shear connection or through the column flange occurred very rarely in buildings with damage rates under 10%.)

FEMA 350 Table 4-12, however, limits the drift angle of prequalified connection types to 0.015 or 0.020 radians for Immediate Occupancy performance. Since FEMA 350 assumes essentially elastic response up to a drift angle of 0.01 radians, Table 4-12 implies plastic drifts or plastic joint rotations of only 0.005 and 0.010 radians. At these plastic rotation levels, properly designed and constructed SMF connections should have no flange fractures at all. It thus appears that the approximate percentage of fractures in Table 4-2 is based only on analytical lateral stability studies, not on FEMA 350's own design criteria.

Further, the damage associated with Immediate Occupancy in Table 4-2 conflicts with Section 4.2.2.2.2, which states that "Damage is anticipated to be so slight that it would not be necessary to inspect the building for damage following the earthquake, and such little damage as may be present would not require repair." Full-blown connection fractures result in substantial loss of connection strength and stiffness (50% or more). Most engineers would not consider ignoring such a loss in 10% of a frame's connections. The design objective described in Section 4.2.2.2. should not be construed as a dismissal of the need for post-earthquake inspection, damage assessment, and repair.

Performance objectives for steel moment-resisting frames have been described by other documents as well. The SEAOC Vision 2000 Committee (California Office of Emergency Services, 1995) and Appendix G of the Blue Book (SEAOC, 1999) say that Operational performance might involve "Minor local yielding at a few places; no observable fractures; minor buckling or observable permanent distortion of members." They say that permanent drift should be "Negligible" for Operational performance and "Less than 0.5 percent" for Life Safety performance. FEMA 350 Table 4-2 allows a permanent drift "Less than 1 percent" for Immediate Occupancy. (From a structural perspective, the Operational and Immediate Occupancy objectives require the same structural response, but Operational performance requires that nonstructural components remain functional as well.)

As a comparison, for the seismic rehabilitation of existing buildings, FEMA 356 (2000) states that Immediate Occupancy performance should involve at most: "Minor local yielding at a few places. No fractures. minor buckling or observable permanent distortion of members." It also states that permanent drift should be "negligible" (meaning something less than 0.1%) at that performance level. For Life Safety performance, permanent drift is kept under 1% in FEMA 356.

PART C AREAS REQUIRING FURTHER RESEARCH

FEMA 355D Section 7.3 lists the following issues as unresolved, requiring "additional research to develop fully rational design guidelines:"

- Reliability of details with minimal testing, in particular Free Flange and Weld Overlay details.
- Liberalized lateral bracing requirements for girders.
- Liberalized continuity plate requirements.
- Effects of panel zone yielding on connection performance.
- Yield mechanisms and failure modes of bolted connections.

In addition, the SEAOC Seismology Committee recommends further research on the twelve topics discussed briefly below. Of these, five (in no particular order) are considered to be of highest priority, based on expected usefulness and importance in understanding frame performance:

- As-constructed weld interface.
- Additional connection tests.
- Panel zones.
- Low cycle fatigue.
- Deep columns.

1. As-Constructed Weld Interface

The SEAOC Seismology Committee maintains, and the FEMA/SAC documents acknowledge, that the exact influence of certain field conditions at the welded beam flange-to-column flange joint is still not entirely predictable. Tests used by SAC to prequalify connection details did not necessarily duplicate or capture the full range (or likely combinations) of:

- Material and workmanship flaws.
- Weld and base metal toughness.
- Stress concentrations.
- Variable column materials.
- Column flange thickness.
- Shear forces at the column face.
- Axial tension in the column flange (although most tests induced substantial flexural tension).
- Etc. See Blue Book commentary section C703.2 (SEAOC, 1999).

Component testing, as opposed to tests of full beam-column assemblies, should be sufficient to address these conditions.

2. Connection Types

Further testing and development is indicated for the following:

- Welded Unreinforced Flange—Welded Web (WUF-W). See Part B, Section 19.1, above.
- Free Flange (FF). See Part B, Section 19.2, above.
- Bolted connections, particularly the Bolted Flange Plate (BFP). See Part B, Section 19.4, above.
- Connections using Weld Overlays.
- Connections in which columns yielding might occur.

3. Panel Zone Performance

Research should attempt to define the bounds between weak and strong panel zones for different connection types.

4. Lateral Bracing near the Plastic Hinge

For connection types that move the beam plastic hinge away from the column face, research should develop, and confirm by testing, a theoretical basis for bracing requirements near the anticipated hinge location. Both strength and stiffness requirements to address lateral torsional and local buckling are needed, as are methods to determine a maximum allowable distance from the hinge to the brace.

5. Damage States by Performance Level

The question of how much frame damage is acceptable for Immediate Occupancy, Life Safety, or Collapse Prevention deserves more attention. The Performance Based Engineering subcommittee of the SEAOC Seismology Committee expects to address this question.

6. Low Cycle Fatigue

Research should attempt to define predictable relationships between local buckling, low cycle fatigue, and eventual fracture. This topic relates to braced frame systems as well as moment frames.

7. Columns Deeper than W14

Further testing of deep columns is recommended. The effects of stiffeners and doubler plates on panel zone buckling and the effects of column flange restraints on twisting should be studied. Effects of low toughness in the K-area are important as well.

8. Column Moment Magnification

The actual forces in columns subject to frame action should be studied in order to develop reliable but realistic moment magnification factors for design. A better understanding of ductility capacity in heavy column sections may require additional testing. Also, a column crack study, using SAC data, should be conducted to understand the relationship of column cracks and the potential for column moment magnification, along with other variables that might propagate column cracking.

9. Connection Details at the Roof

Conditions at the roof or uppermost floor of a frame column have not been tested. Practical options such as extension of the column, use of a column cap plate, and allowing the column to yield at the roof level should be studied.

10. Fracture Toughness at Service Temperatures

FEMA 350's toughness requirements appear adequate for most common conditions, but there remains a lack of data and understanding regarding the parameters that affect fracture control. Further testing should attempt to define useful fracture control plans, with toughness requirements dependent on service temperature, flange thickness, flaw size, etc.

11. Column and Beam Flange Thickness

A parametric study, including testing, should address the effects on connection performance of residual stresses and variation in column and beam flange thickness over the range of member sizes likely to be used.

12. Base Metal Properties

Regular testing of steel by AISC or other appropriate organizations is recommended. Testing should monitor material properties, particularly where mill practices change or are different between mills. Furthermore, testing is recommended to determine if higher base metal notch toughness can contribute to reduction of fractures at stress concentrations.

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APPENDIX A

The SEAOC Seismology Committee's Role

SEAOC technical committees have developed seismic design criteria, written building code commentary, and recommended building code provisions since at least 1960. SEAOC was one of three SAC Joint Venture partners, and prominent SEAOC members contributed to the SAC effort, although the SEAOC Seismology Committee was only indirectly involved.

As noted in FEMA 350 Section 1.2, "Development of [the FEMA 350] recommended criteria was not subjected to a formal consensus review and approval process, nor was formal review or approval obtained from SEAOC's technical committees." The FEMA recommendations are neither codes nor consensus standards. They are intended to serve as resource documents for code development.

To facilitate the appropriate use of FEMA 350 by engineers and building officials, the SEAOC Seismology Committee formed a task group charged with the review, assessment, and commentary on FEMA 350. That task group is responsible for the Commentary and Recommendations presented here and for their near-term development.

APPENDIX B

Application to Ordinary Moment Frame Systems

Prepared by Tom Hale for the SEAOC Seismology Committee

This appendix addresses the system that FEMA 350, the 1997 UBC, the 2000 IBC, and the 2001 IBC Supplement all call the Ordinary Moment Frame (OMF). AISC's Supplement No. 2 (AISC, 2000) and the 2000 NEHRP Provisions (FEMA 368) both include a system with similar design requirements, which they call an Intermediate Moment Frame (IMF). In this appendix, the FEMA 350 OMF and the AISC IMF are considered essentially the same and are referred to by the older designation: OMF.

The IBC 2000 and IBC 2001 Supplement currently allow the use of OMF systems in Seismic Design Category D to a height of 35 feet. In Seismic Design Category E, OMFs are allowed except in multistory buildings where dead loads exceed 15 psf for floors, roofs and walls. The dead load limit in SDC E was intended to allow at least light frame construction in regions of high seismicity. It may appear as though light framing would mitigate concerns for poor moment frame performance in regions of high seismicity. In practice, however, the reduced dead load merely leads to lighter beam and column sizes, and the number of bays of moment resisting framing remains the same as in typical buildings with heavier concrete deck and steel floor framing. The lighter beam and column sizes experience about the same plastic rotation demands as conventional sizes, but the lighter members have larger width/thickness ratios, which are not desirable for developing reliable plastic hinges.

The minimum qualifying total interstory drift for OMF systems, given in FEMA 350 Table 3-15, are supported by a SAC-sponsored report by Yun and Foutch (2000). The objective of the Yun and Foutch report was to address weak column-strong beam (WCSB) systems with no plastic hinging in the beams. Their report was based on an example 3-story OMF in Seismic Design Category (SDC) D only. They did not address the OMF in SDC E, where the roof, wall and floor dead load in multistory buildings may not exceed 15 psf. Had the lighter building been considered, OMF beams and columns with much larger width/thickness ratios (and probably non-compact sections) would have been studied.

The OMF example in Yun and Foutch was designed to meet the requirements of the 1997 NEHRP Recommended Provisions, also known as FEMA 302 (BSSC, 1997). The seismic force resisting system consisted of three-bay frames with W14x311 exterior columns, W14x342 interior columns, W27x161 roof beams, and W33x354 or W33x318 floor beams. An independent review (Hale, 1999) revealed that this was a conservative design that does not reflect the optimized OMF member sizes used in practice. In the Yun and Foutch example, the floor beam sizes were selected to assure a weak column-strong beam system. Total floor/roof masses used to determine seismic forces averaged approximately 120 psf, where 90 psf is more typical for structural steel buildings.

The Yun and Foutch example, designed to NEHRP criteria, was controlled by drift, not strength. Member demand/capacity ratios (using LRFD) ranged from 0.2 to 0.6. Drifts determined from the seismic lateral static forces using the calculated fundamental period were from 60 to 70 percent of the maximum allowable drifts. Yun and Foutch concluded (in section 5.7 of their report): "The overall strength of the building was much greater than required for this site. Thus, even though hinges formed in the columns, the demands were so small that the buildings performed well." A more optimal design would likely have led to different conclusions.

The median first story drift from a nonlinear dynamic analysis of the 3-story OMF without doubler plates was 2.5 percent. The 84th percentile drift was 3.5 percent, and the 95th percentile drift was 5.0 percent. (The plastic rotations occurred principally in the panel zones. However, further analyses were made with strengthened panel zones and beams to force plastic hinging in the columns.) Assuming an approximate equivalency between total connection rotation and interstory drift angle, this compares with the minimum required drifts for OMF systems given in FEMA 350 Table 3-15: 0.02 radians at the point of strength degradation and 0.03 at the ultimate state (connection failure). The IBC connection inelastic rotation requirement is 0.01 radians, which is consistent with FEMA 350's assumption of 1 percent elastic drift. Thus, the rotation capacities required by FEMA 350 do not appear adequate unless plastic hinging in the column—to protect the beam-to-column connection—is assured.

Since the SAC Phase 2 tests did not include column plastic hinging, the effect of that hinging on the connection is not known. For low rise buildings where column axial loads are a small portion of the column capacity (10%-20% of P_n), sway mechanisms involving column hinging might be acceptable, but with limits on column width-thickness ratios. Small width-thickness ratios are necessary to suppress local buckling that will subsequently cause premature fracture in the early stages of plastic hinging. The width-thickness limits in the 1997 AISC Seismic Provisions referenced in section 10.4b are presumed sufficient to prevent premature fracture.

Yun and Foutch have shown by analysis that WCSB systems are viable when column axial loads are small, column width-thickness ratios and system height limits are controlled, and column panel zones are strong enough to develop the yield moment of the framing beams. Under these conditions, plastic hinging will occur principally in the columns and not in the panel zone or beams. This will protect the IMF and OMF beam-to-column connections. Without these requirements for WCSB proportioning, strong column-weak beam behavior could occur, with substantial plastic hinging in the beam or panel zone, and unattainable ductility demands on the beam-column connection.

Traditionally, IMF and OMF systems have had few ductility requirements and were "catch-all" categories for frame designs that did not meet the SMF detailing and ductility requirements. Analytical studies cited by SAC and by the SEAOC Seismology Committee (Hale, 1999) have demonstrated that OMF and IMF systems designed by current building codes might have plastic hinging in the beams or columns. Therefore, if connections with low ductility capacities are to be used, more rational system requirements are needed for the OMF and IMF.

APPENDIX C

Interim Review of Welded Unreinforced Flange—Welded Web (WUF-W) Connections

July 9, 2001

Prepared by Peter Maranian and Robi Kern for the SEAOC Seismology Committee

Peter Maranian and Robi Kern, on behalf of the SEAOC Seismology Committee, conducted a detailed analysis of the SAC connection test reports for WUF-W connection. Copies of this report may be obtained from by contacting the SEAOC Seismology Committee.

Testing was conducted in two phases on W 36x150 beams. The first set, the "T" series set, was a preliminary study. This set of tests was used to develop connection details for the second set, the "C" series set, which represents the final connection as included in FEMA 350. The following summary of findings and conclusions is extracted from that report:

Summary of "T" Series Tests

1. These tests show the development of the FEMA 350 recommendations for WUF-W connections.
2. Four out of the five on WUF-W tests (Specimens T1, T2, T5 and T6) achieved the 4 percent drift requirement. Test T4 was a WUF-B specimen. Test specimen T3 failed at less than 3 percent drift. Specimen T6 achieved 6 percent drift without failure.
3. Specimens T5 and T6 had the beam web attached to the column flange with a complete penetration welds.
4. The panel zones for Specimens T1 and T2 contributed significantly to the plastic rotation. They did not have doubler plates. However, had they been designed in accordance with FEMA 350 Section 3.3.3.2, they would have required doubler plates which would have significantly reduced the panel zone contribution to plastic rotation.
5. Specimens T5 had a small panel zone contribution but performed well. Welding the beam web to the column flange with a complete penetration groove weld enhanced its performance.
6. All specimens displayed some cracking during early cycles (even at less than 2 percent drift).
7. Failures in four out of the five specimens occurred at or near the welded interface. Specimen T6 did not fail but had cracks at the fusion line in the bottom flange weld.
8. No cracking was found in the weld access hole region prior to final fracture.

Summary of "C" Series Tests

1. All five tests had details similar to the FEMA 350 recommendations.
2. All five tests achieved the 4 percent drift requirement.
3. Panel Zone contributions were small. However, it should be noted that the doubler plates used are more than twice the thickness required per FEMA (3-7), Section 3.3.3.2. If doubler plates, with theoretical thickness, designed in accordance with FEMA 350 were used, based upon the peak test loads, average panel zone shear stress were estimated above 37 ksi (Specimen C1) and as high as 44.1 ksi (Specimen C4). At these levels of shear stress, significant panel zone yielding appears likely.

4. All specimens displayed some cracking, typically at the ends of the web groove weld. However, use of run off tabs at the beam web reduced the tendency for the crack to propagate.
5. Significant local buckling of beam flanges and, in some cases beam webs, occurred, which was the main cause of degradation.
6. No cracking was found in the weld access hole region prior to final fracture.
7. The stresses at the flange weld interface were very high even when including the contribution of the shear plate. Average stresses were as high as 88 ksi. This also resulted in significant column stresses close to or even above yield (Specimens C1, C2, C5).

Summary of Other Items of Report

1. The report by Ricles et al has some excellent discussion on the issues associated with this type of connection including continuity plates, panel zone weld access hole geometry, beam web attachment details.
2. The report provides a detailed discussion on low cycle fatigue. Furthermore, it develops a new method for low cycle fatigue analysis using non-linear finite element analysis to predict crack initiation and extension and the life cycle of a beam-to-column connection. The low cycle fatigue analysis results carried out by Ricles et al were in good agreement with test results. The low cycle fatigue analysis also showed that connections with strong panel zone had better performance than connections with weak panel zones. The strong panel zone limits excessive shear distortion of the panel zone, which in turn reduces distortion in the vicinity of the flange/column weld and beam/web intersection at the access hole. This delays the propagation of beam web weld cracking.
3. The report has an extensive study on weld access hole geometry and size. It considers nine different weld access hole configurations. Finite element analysis was carried out to determine the ratio of peak plastic strain to yield strain. The least favorable was the standard access hole. The most favorable condition was with no access hole. The most favorable access hole configuration studied is that shown in FEMA 350 Figure 3-5.

Conclusions

1. Initial testing of five one sided connections led to attachment of the beam web to the column flange using complete penetration welds with a shear plate serving as a backing plate. Welding around the bolted shear plate with a fillet weld was found inadequate. Subsequently the C series consisting of five two sided tests using W36x150 beams were carried out and developed plastic rotations of 0.04, 0.05, 0.052 and 0.046. Although these tests performed well, cracking of the beam flange welds to the column flange occurred at lower drift (3 percent drift). Also, the beam web welds to the column flange cracked at lower drifts (3 percent drift) but these cracks did not propagate with the exception of test specimen C1.
2. Severe local buckling of the flanges and web occurred, which significantly contributed to the plastic rotation. Beams with lower b/t_f and d/t_w ratios may not exhibit sufficient local buckling and may not perform as well. This is due to the fact that delay of flange and web buckling may tend to maintain or increase demands on the welds, which had already commenced cracking at earlier stages.
3. The report by Ricles et al recommends a strong panel zone to limit excessive distortion and delay the propagation of the beam web weld cracking due to low cycle fatigue. However, it is important to note that the C series specimens used doubler plate sizes significantly greater than the

theoretical doubler plates required by FEMA 350, 3.3.3.2. Had doubler plate sizes been used that are comparable to the theoretical size, significant panel zone yielding would probably have occurred. The possible excessive distortion resulting from panel zone yielding may have led to propagation of cracking particularly in the web weld.

4. The report by Ricles et al gives a good indication of details associated with the web attachment. It also demonstrated the importance of run off tabs at the beam web to reduce the tendency for the crack to propagate.
5. These connections all displayed high peak test load to theoretical test load ratios (as much as 40 percent greater). Demands at the welded interface were very high possibly due to strain hardening and triaxial constraint. Demand on the column appears to be far greater with this connection than other connection types (e.g. RBS, BFP, WFP). This suggests use of a higher joint strength ratio to insure hinging occurs in the beam.
6. The report by Ricles et al highlights the phenomena of low cycle fatigue as an important issue.

APPENDIX D

Interim Review of Welded Flange Plate (WFP) Connections

May, 2001

Prepared by Peter Maranian and Robi Kern for the SEAOC Seismology Committee

Peter Maranian and Robi Kern, on behalf of the SEAOC Seismology Committee, conducted a detailed analysis of the SAC connection test reports for WFP-W connection. Copies of this report may be obtained by contacting the SEAOC Seismology Committee.

The following summary of findings and conclusions is extracted from that report:

Summary

1. All five tests exhibited substantial beam flange and web local buckling causing appreciable degradation. Failure typically occurred through fracture of the flanges due to low cycle fatigue. In four of the specimens, the moment capacity at 4 percent drift was less than 80 percent of the nominal plastic capacity. The fifth specimen performed better because it had significant panel zone yielding (see 2 below).
2. The four specimens with doubler plates had only a small contribution to drift from the panel zone (about 0.25 percent). The fifth specimen (UCB RC09), built with no doubler plate, experienced substantial panel zone contribution to drift (about 2.5 percent).
3. The complete penetration welds at the welded interface did not fail. Average test stresses at the welded interface were high if web capacity is ignored (as much as 63.6 ksi, UCB RC08). If estimated beam web capacity is included, the average stress is probably at or less than yield (maximum 58.3 ksi). The moment plates were significantly thicker in the first three tests than determined from FEMA 350 (3-13). The last two tests very closely match the thickness values determined from FEMA 350 (3-13) (0.82" per Eqn 3-13 compared to 7/8" used).
4. The fillet welds connecting the cover plates to beam flanges did not fail. In all cases (except RC04, which was not evaluated because it had a dovetailed plate), the average weld force/inch, including the transverse weld, was greater than the maximum capacity of the weld determined from AISC LRFD assuming $\phi = 1.0$ and if the beam web capacity is ignored. If the estimated beam web capacity is included, the weld force/inch is less than the ultimate theoretical capacity of weld in only one test (UCB RC 7) and marginally higher than the ultimate theoretical weld capacity in the remaining tests (UCB RC 6, 8, and 9). The fillet weld sizes used were significantly smaller than determined from FEMA 350 Equation (3-14) - 3/4" per Equation 3-14 compared with 5/8" and 9/16" used. It should be noted that a 3/4 inch fillet weld would not be feasible for a W30x99 since the beam flange thickness is 11/16 inch.

Conclusions

1. The tests performed in a ductile manner typically with ductile tearing of the flanges. However, the test results for four out of the five specimens do not satisfy the requirement for ϕ_{SD} , in FEMA 350, Table 3-15 for SMF due to too much degradation at a drift of 4 percent. The one specimen that did satisfy the requirement for ϕ_{SD} appeared to be due to the panel zone contribution as a result of their not being a doubler plate. However, applying FEMA 350 Section 3.3.3.2 would have resulted in the need for a doubler plate for this specimen as used on the rest of the test specimens. With a doubler plate, the panel zone contribution to rotation would have been small (less than 0.25 percent).

2. The design formulae appear to need correction both for the flange plate and the weld connecting the flange plate to the beam flange. Recommendations, if used as an OMF, are given below.
3. The performance did not satisfy the requirements of 1, 2 and 4 given in FEMA 350, Section 3.4 for pre-qualification for SMF. The connection may be suited for use as an OMF, where the inelastic behavior is expected to be limited. Sizes should not exceed that tested in depth, weight and beam flange thickness.
4. None of the specimens displayed weld failures even though estimated average force/inch in the fillet welds connecting the flange plates to beam flanges were high. This is encouraging particularly as weld failures on tests of cover plated beams, and also previous tests by Noel and Uang, 1996 on beams with flange plates, occurred as described in the report by Whitaker et al, 2000.

APPENDIX E

Interim Review of Bolted Flange Plate (BFP) Connections

June 6, 2001

Prepared by Peter Maranian and Robi Kern for the SEAOC Seismology Committee

Peter Maranian and Robi Kern, on behalf of the SEAOC Seismology Committee, conducted a detailed analysis of the SAC connection test reports for BFP-W connection. Copies of this report may be obtained by contacting the SEAOC Seismology Committee.

The SAC test program included tests of eight specimens, however, complete test data for one test, BFP 08, was not made available and is not included in this evaluation. The following summary of findings and conclusions is extracted from that report:

Summary:

1. All test specimens had substantial panel zone yielding. The panel zone contributed as much as, or more than, 1.3 percent rotation.
2. Specimens BFP01, BFP02, BFP04, BFP06 and BFP07 would require doubler plates if designed per FEMA 350 clause 3.3.3.2. In the case of BFP03 and BFP05, the need for a doubler plate was marginal (theoretically 0.031 inches). Without a doubler plate, the average panel zone shear stress in BFP03 and BFP05 was about 28 ksi, indicating that panel zone yielding was only just attained. None of the specimens had a doubler plate with the exception of specimen BFP0 8, which had a doubler plate, added in the second retest.
3. Based upon the theoretical doubler plate thickness required (note that the practical thickness would be greater to an even 1/8 inch increment), it is unlikely that any significant panel zone yielding would have taken place.
4. The reports by Schneider and Teeraparbwong on BFP01 through BFP04 show that bolt slip occurred below the AISC ASD allowable slip critical values. BFP01, BFP02, and BFP03 slipped more than 30%, and BFP04 slipped 25% below the AISC ASD values. It is our understanding that the AISC slip critical values incorporate a factor of safety on the slip values, so that this result was surprising. The reports stated that the bolts were torqued to the specified pre tensioned requirements.
5. Oversized holes in the flange plate were used in BFP01 and BFP06. It also appears that oversized holes were used in BFP05 and BFP07 although this is not clear, as there are inconsistent statements in FEMA 355D. The contribution of bolt slip when oversized holes were used is significant. Based upon the report on BFP01, approximately 0.5 percent rotation occurred in the connections with oversized holes compared with 0.25 percent for standard holes for W24x members. For deeper beams, the bolt slip contribution would be less.
6. Significant flange and web local buckling took place in all specimens, which appeared to have contributed marginally to the total rotation. It should also be noted that the beam sizes selected, W24x68 and W30x99, do not satisfy the compact section requirements for Grade 50 member sizes.
7. Failure mechanisms were typically along the last bolt line in the beam flange, that is, net section failure. Analysis confirms high net section stresses. BFP01, BFP03, and BFP05 had net section stresses of approximately 72 ksi and BFP02, BFP04, BFP06, and BFP07 experienced stresses of

approximately 80 ksi. Except for BFP01, which failed at the welded interface (see below), these values are above the mill test ultimate tensile stress. Based upon the mill tests for the beams, the ratio of F_y/F_t was 0.76 except for BFP03 and BFP05, which was 0.79. Actual ratio of average gross stress to average net section stress was 0.75 for the W24x68 tests (BFP01, BFP02, BFP04, BFP06, BFP07) and 0.78 for the W30x99 tests (BFP03 and BFP05). Comparison of these ratios very closely match (0.76 (F_y/F_t) compared to 0.75 (f_g/f_n), 0.79 (F_y/F_t) compared to 0.78 (f_g/f_n). Thus our analysis, considering mill test values, only predicts a possibility of net section failure. Presumably, stress concentrations encouraged net section failure to occur in the tests. Had the ratio of F_y/F_t , for the beam, been greater, it is likely that failure at the net section would have occurred earlier. It should be noted that ASTM 992 permits $F_y/F_t < 0.85$, well above the ratio for these test specimens.

8. Exceptions to this mode of failure occurred in specimen BFP01, which failed in the heat affected zone of the column flange and BFP08, which developed a significant lateral torsional buckling zone such that the test was stopped. It should be noted that Specimen BFP01 had a relatively high average through-thickness stress (59.7ksi). Specimen BFP01 failed at the welded interface. The failure mode of test BFP01 is a reminder that fracture at the welded interface is still possible.
9. Peak loads were all less than predicted from the beam plastic moment including over strength and strain hardening. As shown in the reports, although girder hinging did occur, it was typically not substantially developed.
10. Except BFP01, the average bolt forces due to the peak test load were close to or higher than ultimate values given in the AISC LRFD manual with ϕ equal to one. BFP02 and BFP06 were 7 percent, and BFP07 was 9 percent above the AISC LRFD values.
11. Stresses between the top and bottom plate and column at the welded interface where as high as 49.3 ksi, significantly exceeding the specified yield. Mill test and/or coupon test information was not found in the report. The ratio of average net section stress was not less than 0.775.
12. If panel zone yielding does not take place then, the plastic rotations would have been far less. Approximate estimates for total drift at failure indicate the following:

BFP01	3.15 percent
BFP02	3.65 percent
BFP03	3.75 percent
BFP04	3.85 percent
BFP05	3.95 percent
BFP06	5.6 percent
BFP07	4.5 percent
BFP08	Unknown (await report)

Thus, it appears that 5 out of 7 tests would not have provided the 4 percent drift requirement in FEMA 350 for SMF's without panel zone yielding. It should be noted that use of standard holes would further reduce the values for BFP01 and BFP06 by approximately 0.25 percent (and subject to verification, BFP05 and BFP07).

Conclusions

1. It is clearly evident that reliance upon panel zone yielding contributing to the connection rotation is not justified if the panel zone is designed in accordance with FEMA 350 Section 3.3.3.2. Analyses of these tests reveal such designs produce panel zone that behave closer to an elastic condition than an inelastic condition.
2. If the panel zone requirements are made more liberal, there is also the problem that the column steel yield strength can vary from 50 ksi to 65 ksi. Thus, even with careful selection of column size and doubler plate thickness, the actual yield strength of the column and doubler plate could

exceed expected strengths and yielding of the connection and girder such that occur without allowing panel zone yielding. This condition would likely prevent attainment of 4 percent total drift. Please note that our concern for some degree of reliance on panel zone yielding which may otherwise not occur may also apply to other connection types.

3. The beam material properties reported in test reports had good yield to ultimate tensile stress ratios (0.76 and 0.79). Beam materials closer to the limit of 0.85 for F_y/F_t would more than likely exhibit less favorable behavior at the net section leading to net section fracture at lower drift values.
4. The performance of the connection with oversized holes is concerning since bolt-slip occurred in these tests at relatively low moment (less than 40 percent Peak moment). Thus permanent deformation caused by moderate earthquakes and wind is possible. Furthermore, there is concern that, in practice due to lack of ideal field fit, bolts in oversized holes may not always be placed correctly and some bolts may take load in bearing before others. This may result in failure of the bolts at less than expected rotation.
5. With regard to the design method given in FEMA 350, the formula for evaluating the flange bolts (equation 3-43) and the net section fracture of the flange plate (equation 3-45) are not consistent with the principles of mechanics. Also, this committee received reports from engineers attempting to apply the FEMA 350 bolt design method who were unable to find design solutions. This was confirmed by checking several connections from an actual frame design.
6. In our opinion the performance did not satisfy the requirements 2 and 3 given in Section 3.4 for pre-qualification.