WANT SOME WISDOM from a well-known welding wizard?

The second edition of Design Guide 21: *Welded Connections—A Primer for Engineers* was recently released. And what better way to learn about it than a chat with its author, Duane Miller.

The below conversation between AISC's Margaret Matthew and Miller provides

Design Guide 21 includes discussion of specialized situations such as welding on architecturally exposed structural steel (AESS), field welding and welding on existing structures.

Finally, while it may not seem all that signi cant, the addition of an index will, I hope, prove to be user-friendly. When the reader needs to know about porosity, preheat or an RBS connection, the index will direct them to the portions of the guide that discuss that topic.

MM: You were right, that was a long answer—but a good one. Next question: This new edition includes an expanded chapter on seismic welding issues. Why are welded connections subjected to seismic loading expected to behave differently than either statically or cyclically loaded welded connections?

DM: For our reader's bene t, it was AISC's idea to expand the chapter on seismic welding issues (Chapter 11), as well as to include the new chapter on fracture-resistant welded connections (Chapter 13). Both chapters required a lot of additional work, but I hope the efforts will increase the usefulness of the guide.

I'll assume there is a base knowledge amongst the readers of *Modern Steel* as to what constitutes "static" loading. The "cyclic" loading conditions discussed in AISC *Specification* Appendix 3 (and discussed in Chapter 12 of the updated guide) deal with low-stress-range, high-cycle-loading situations. Appendix 3 deals with situations where the number of cycles of loading is expected to exceed 20,000. There is an implicit assumption that peak stresses are elastic when Appendix 3 is used. In contrast, seismic loading involves high-stress-range, low-cycle applications.

The seismic design criteria contained in the AISC *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341) and AISC *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (ANSI/AISC 358) assumes inelastic deformations will be associated with a design-level earthquake. As a result, we have three basic loading conditions—static, cyclic and seismic—all with different design, detailing, fabrication, erection and inspection criterion.

MM: Why was the Northridge earthquake in 1994 one of the most signi cant earthquakes of the past century with regard to the wealth of engineering data obtained? And more speci cally, the wealth of knowledge gained with regard to welded connections?

DM: You're quoting from the design guide, but I'm not the author of those words; that is a quotation from Ron Hamburger. And obviously, I agree with those comments. One person on the peer review panel for the design guide took major exception to the inclusion of a discussion of the Northridge Earthquake, an event that took place over 20 years ago. The section was retained, however, for the reasons Ron stated.

Seismic design has always relied on analysis, laboratory experimentation and actual post-earthquake eld observations. In the case of the Northridge earthquake, the unexpected damage to the welded connections in moment frames resulted in millions of dollars of research on the focused topic of welded connections. The complexity of the behavior of the moment connections, and the myriad contributing factors to the observed behavior, required systematic research that was in some ways unprecedented. A unique aspect of the ndings from Northridge was that they were quickly incorporated into consensus documents such as the *Seismic Provisions*.

MM: There is also the new chapter on fracture-resistant welded connections (Chapter 13). For most building structures, the *Specification* indicates that the probability of fracture is low. Under what conditions does a design engineer need to worry about fracture as a limit state in welded connections?

DM: For most building applications, the limit state of fracture is not a principal concern because it is not typically the controlling limit state. There are many reasons why this is the case: The steel in most buildings in service is relatively warm; service loads create strain rates that are essentially static; the number of full design stress cycles is typically low; codes limit the severity of stress raisers; buildings normally have signi cant redundancy; and the typical fracture toughness of steels used in building construction is adequate even when fracture toughness levels are not speci ed. Thus far, I have not responded to your question but have reinforced the premise behind your question—that the risk of fracture in buildings is relatively low.

When does the design engineer need to be concerned about fracture? When the listed conditions are not true: when the service temperature of the steel may be cold; when dynamic loads are anticipated to be applied to structures (e.g., seismic or blast); or when the structure will be cyclically loaded (e.g., crane rail supports). Under such conditions, the possibility of fracture increases. For structures with less redundancy, the consequences of fracture become more pronounced. Finally, there are a few notable examples where the fracture toughness of commonly used steels is lower than normal. When such situations are encountered, considering the limit state of fracture is more important.

MM: What is included in the updated chapter on special welding applications (now Chapter 14)?

DM: This chapter deals with unusual welding-related applica-