Ductility in MODERATION

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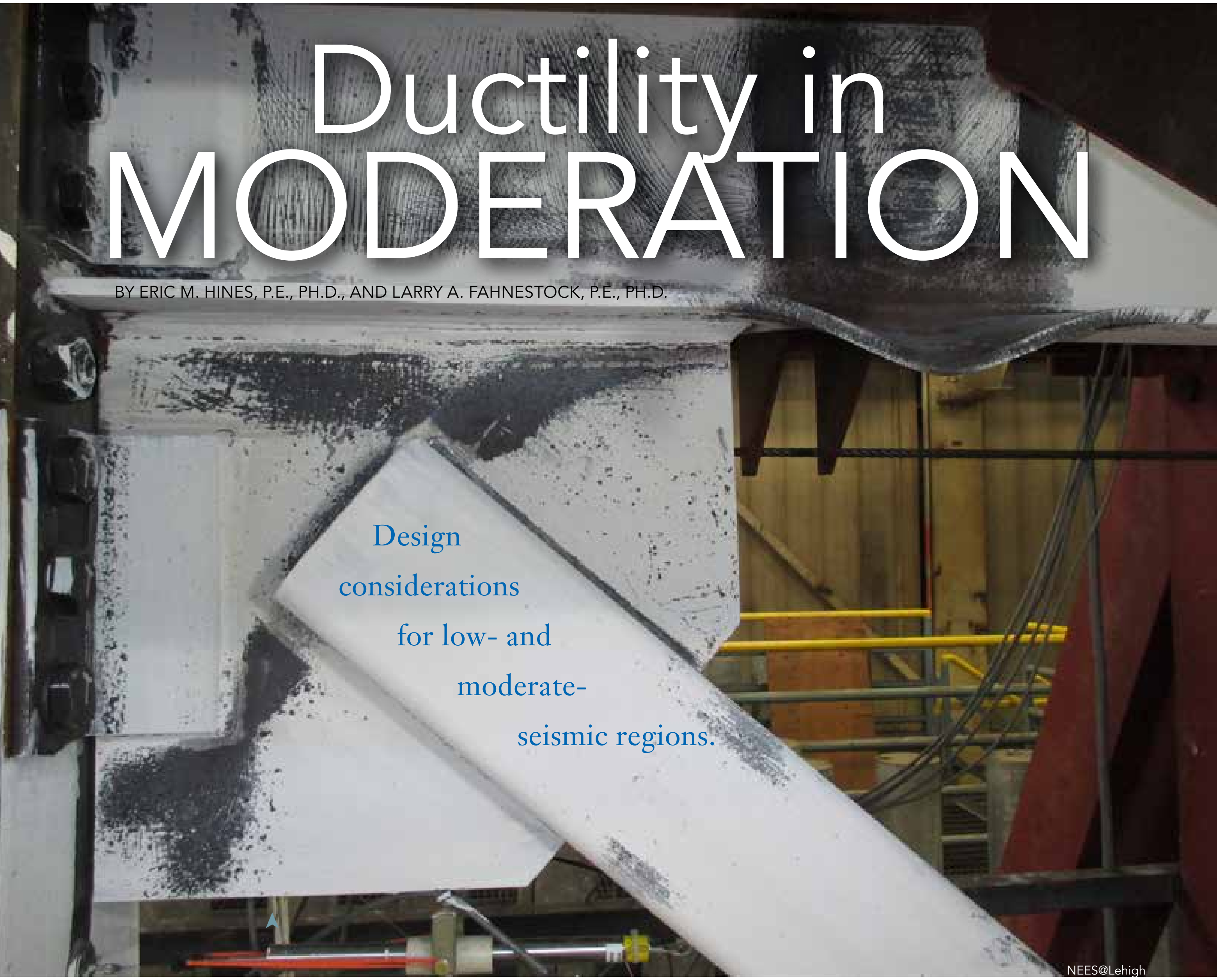
Design considerations for low- and moderate-seismic regions.



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SEISMIC DESIGN OF STEEL STRUCTURES in the U.S. emphasizes the development of ductile elements designed to experience inelastic behavior during a seismic event.

The system containing these elements must be "capacity designed" with enough strength in the non-yielding elements to ensure that the yielding elements can sustain significant inelastic deformation. Thus, the expense of achieving high-seismic performance resides both within the detailing of the ductile elements themselves and the strengthening and detailing of the surrounding system to remain elastic.

A large share of the seismic research in the U.S. has focused on developing the detailing needed to achieve ductility. For decades, the cost of this detailing has been perceived to be less than the cost of designing a stronger system both in high-seismic regions as well as moderate-seismic regions. But as seismic detailing requirements have grown more sophisticated and stringent since the 1990s, engineers in areas of moderate seismicity have observed that ductile detailing of elements within a capacity-designed system can be prohibitively expensive. Consequently, the use of the $R = 3$ provision for steel struc-

▲ Dudley Square Police Station in Boston.

▲ OCBF brace buckling testing.

◀ Ordinary concentrically braced frame (OCBF) testing at Lehigh University.

▼ R = 3 brace buckling testing.

▼ R = 3 chevron braced frame testing.

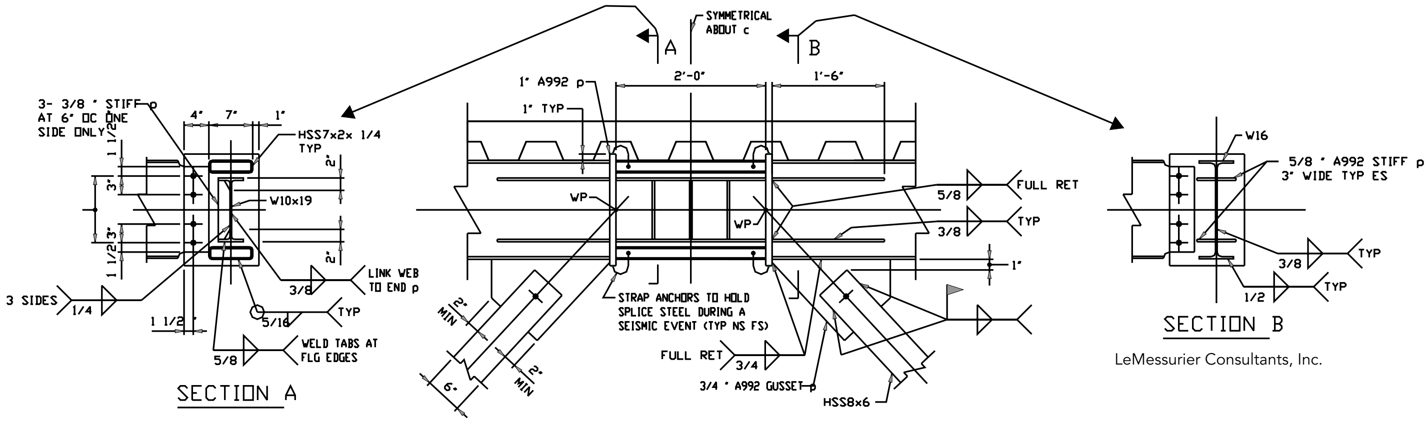
tures, which allows for seismic force reduction without ductile detailing, has expanded significantly. Within the last 10 years it has become clear that if engineers practicing in moderate-seismic regions wish to employ a consistent seismic design philosophy, a new approach to seismic research is required.

Whereas past seismic research focused on developing details to achieve a given level of ductility has placed cost in a secondary role, this new research must place priority on reducing cost while recognizing that significant performance may be achieved with moderate ductility levels. A deeper understanding of moderate-ductility systems will allow for the development of a new seismic design philosophy based on system reserve capacity.

Robust Flexibility

In moderate-seismic regions, the concept of reserve capacity can complement the concept of ductility in a manner that offers flexibility for structural designers to develop robust systems for complex structures. Low-ductility steel concentrically braced frame (CBF) structures comprise a significant portion of the national building stock, yet their inelastic seismic response is not well understood. While these structures have brittle brace elements and connections, they can achieve system ductility through contributions from the braced frame gusset plate connections and the gravity framing. The resulting "reserve" moment frame system can prevent sidesway collapse even when the primary lateral force resisting system (LFRS) is significantly damaged. In this context, ductility is viewed not as deformation capacity while maintaining full lateral strength, but rather deformation capacity while maintaining a reduced level of lateral strength, provided by the reserve system after degradation of the primary CBF.

Fundamentally, a reserve system is more flexible than a primary LFRS, hence reserve capacity activates after significant damage to the LFRS. This stiffness incompatibility between reserve system and primary LFRS differentiates reserve capacity conceptually from redundancy provided by extra LFRS elements. Although reserve capacity is not currently quantified in design, the R = 3 provision for steel structures in low- and moderate-seismic regions implicitly relies on reserve capacity for collapse prevention, even though the nature of this reserve capacity is not well-understood and can vary widely. Research



A shear link detail for Dudley Square Police Station.

to date has not thoroughly studied reserve capacity, so relying on it without proper understanding in moderate-seismic regions jeopardizes safety. Thus, there is an essential need for clarity and consistency in considering reserve capacity for seismic design and assessment in moderate-seismic regions.

The philosophy of system reserve capacity opens new possibilities for designing structures in moderate-seismic regions, with potential influence on assessing and retrofitting structures in high-seismic regions. This philosophy, which prioritizes cost reduction over achieving optimum levels of ductility, may also impact design in developing countries where ductile seismic details are not affordable or achievable within common practice. Reserve capacity ought to be seen as complementary to ductility. The level to which each philosophical approach is used on a given project ought to be determined by the structural designer in a manner that best suits the project.

Next, we'll discuss recent testing of low- and moderate-ductility systems, then focus on a recent police station project in Boston where the philosophies of ductility and reserve capacity were combined to achieve a high-performing, economical design for a complex system.

Full-Scale Testing

During the summer and fall of 2014, we tested two full-scale CBFs at Lehigh University's Advanced Technology for Large Structural Systems (ATLSS) laboratory in the NEES@Lehigh facility. This work was led by the University of Illinois at Urbana-Champaign (UIUC) and Tufts University as part of a Network for Earthquake Engineering Simulation Research (NEESR) project funded by the National Science Foundation (NSF). UIUC and Tufts are collaborating with a team from the École Polytechnique Montreal (EPM) on this work as part of an international program to investigate reserve capacity in low-ductility CBFs [Fahnestock et al. 2014]. This program also includes testing and analysis of top and seat angle connections for enhanced beam-to-column moment capacity [Nelson et al. 2014], testing of tubular brace reengagement with gusset plates after connection fracture [Davaran et al. 2014], and collapse analysis of 3-, 6- and 9-story low-ductility braced frame buildings with varying levels of reserve capacity [Hines et al. 2009, Sizemore et al. 2014]. The NEES@Lehigh tests included a two-story, R = 3, chevron brace configuration (Fig. 1) and a two-story, OCBF, split-X brace configuration.

These tests were designed to explore post-elastic behavior in low-ductility braced frames with a particular emphasis on brittle damage mechanisms. The full report on these tests can be found in Bradley [2015] and will soon be submitted for journal publication. These tests allow direct comparisons between detailing requirements for OCBF and R = 3 frames and well as direct comparisons between split-X and chevron configurations.

In general, the tests showed positive results for the buckling of OCBF braces designed to meet the moderately ductile member b/t requirements and connected for amplified seismic forces. For the OCBF, the upper story braces were HSS6×6×$\frac{3}{8}$ (b/t = 14.2), and the lower story braces were HSS6×6×$\frac{1}{2}$ (b/t = 9.90). Throughout the entire test, these braces did not experience local buckling, and they developed stable distributed plastic hinge behavior. The OCBF system survived as a moderately ductile system up to 1.5% story drift and an overstrength of approximately 3. Shortly beyond the 1.5% drift level, two fractures at the central split-X connection occurred in close succession and created a very weak two-story mechanism, leaving almost no reserve capacity. For the R = 3 system, the upper story braces were HSS8×8×$\frac{5}{16}$ (b/t = 24.5), and the lower story braces were HSS8×8×$\frac{3}{8}$ (b/t = 19.9). The R = 3 upper story braces buckled suddenly, with significant local buckling (Fig. 3), at a story drift of 0.3%, transitioning the system directly from an elastic behavior to a robust reserve capacity driven by frame action in the test unit.

This test program investigated two fundamental parameters that influence seismic response: system type and system configuration. System type is defined by the level of force reduction (R value) and the detailing and capacity design requirements. In this case, the system type distinction is primarily related to OCBF vs. R = 3 detailing. System configuration is defined by global frame geometry, in this case a split-X configuration versus a chevron configuration. These two tests clearly demonstrated the superiority of OCBF detailing over R = 3 detailing for achieving a moderate level of ductility through brace buckling and yielding. They also demonstrated the vulnerability of a split-X system to collapse if such a system were to form a two-story mechanism, as it did during testing. While the R = 3 system demonstrated poor ductile performance with respect to the bracing, the systems' tendency to form a single story mechanism deriving strength both from column continuity and possible "long-link EBF" mechanisms demonstrated the effectiveness of reserve capacity in maintaining system stability.

It is important to consider that the essential question of reserve capacity in these two systems hinged on whether they

▲ Throughout Dudley Square Police Station, uniformly sized HSS column sections were designed to be exposed to view.

formed a one-story or two-story mechanism in the post-fracture range of behavior. While the potential vulnerability of a split-X system is clear from the test, it is not possible to conclude from a single chevron test that chevron configurations can be expected, in general, to form only single story mechanisms. Designers wishing to rely on such single story mechanism behavior ought to consider exercising a form of capacity design where the story likely to form a mechanism can be identified. Having identified such a story, a designer ought to maintain an awareness of how column orientation, framing configurations, structural discontinuities and column splices would affect the behavior of the identified reserve system. If there is any chance for the formation of a brittle multi-story mechanism, it is critical for the designer to consider beam column connections that can provide adequate moment resisting capacity to form a reserve system. Such connections may be achieved economically with the use of top and/or seat angles, gusset plates and slab reinforcement as discussed by Stoakes and Fahnestock [2011] and Nelson et al. [2014].

Both of these test units represented a level of detailing consistent with the economic constraints faced by designers in moderate-seismic regions. The economy in these frames is achieved both in the compromise of ductile detailing and in the lack of a rigorous capacity design process. For collapse resistance at higher drifts, both systems would rely heavily on their reserve capacity provided by frame action and other possible post-buckling, post-fracture mechanisms. Such behavior may be considered acceptable for a large share of the building stock in moderate-seismic regions, but for essential structures, or for buildings where a higher performance objective is desired, it is reasonable to ask whether ductility can be achieved at a lower cost if the relationship between wind and seismic loads is considered carefully. Such was the case for the project we'll discuss next, where capacity design requirements were reduced by designing weak shear links to act as structural fuses and where reserve capacity concepts were invoked in locations where the architectural program complicated the direct use of ductile detailing.

Designing for Dudley Square

The new Area B-2 Police Station in Boston's Dudley Square neighborhood, designed by architect Leers Weinzapfel Associates and structural engineer LeMessurier Consultants, Inc., provided an opportunity for designers to consider the role of an essential facility as contemporary community building. For the steel structure, this meant framing a glass lobby, a perimeter clerestory and a cantilevered roof to emerge cleanly from a tight-fitting limestone ashlar façade bearing directly on the foundation with no horizontal relieving joints. Throughout the building, consistent HSS column sections were designed to be exposed to view. Architectural considerations related to the dense program within the building, the carefully crafted façade and the desire for future flexibility—combined with the structural imperative of maintaining operation after a hazardous earthquake event—led to the choice of developing the steel framing system independently of the masonry façade and partition system.

Although allowed by code for such a facility, R = 3 CBFs and OCBFs were not attractive candidates for the lateral system due to their inherently brittle behavior and unproven seismic performance. During design development, lack of opportunities for consistent bays of lateral framing in the long direction led to consideration of moment resisting frames (MRFs). However, building stiffness requirements, difficulties with detailing MRFs to perform in a ductile manner with the building's HSS

A sample shear link at Dudley Square.

The 8-in.-square HSS columns provided adequate capacity in most cases under load combinations.

columns and expenses related to the number of required moment connections spurred the design team to evaluate bracing in both directions. In order to provide the building with significant ductility capacity and stiffness, the design team looked into the possibility of using eccentrically braced frames (EBFs), more typically used in high-seismic regions.

For buildings in moderate-seismic regions that are expected to experience wind loads greater or equal to seismic loads, EBFs are commonly thought to be too expensive owing to their capacity design requirements for braces, beams and columns and field-welded erection details. Rethinking the typical EBF link details, however, allowed the designers to reduce capacity design requirements and maintain erection details consistent with a typical CBF. The result was special shop-fabricated link beams with W10×19 shear links that were proportioned to meet the elastic design requirements for the building, which were controlled by wind loads in the short direction and seismic loads in the long direction. Shear links ranged in length from 2 ft to 4 ft in accordance with architectural requirements and column capacity design limits. Horizontal HSS stiffeners on either side of each link provided both stiffness on the weak axis during erection and a surface for attaching the composite floor deck without disturbing the link itself. These stiffeners were designed as sacrificial elements, and their incidental strength was considered in the capacity design of the system. The link beams required no special measures for erection and allowed the use of relatively light braces with no special slenderness requirements.

In the absence of specific provisions for design of weak shear links in such an application, links were selected to resemble as closely as possible those tested by Okazaki and Engelhardt [2007]. These links were constructed from A992 steel in contrast to the links tested in the 1980s that were constructed from A36 steel. The test units themselves were wide-flange sections welded to end plates and then bolted to the test setup. The Dudley Square link details were designed to imitate the details of the actual test setup as closely as possible. The 33,000-sq.-ft, three-story structure was designed in conformance with the *Massachusetts State Building Code*. Per this code, it was assessed to have a fundamental period of T = 0.51 sec. Considering the number of CMU partition walls in the structure, a decision was made not to amplify the building period beyond its base value.

The seismic weight of the structure was calculated to be W = 4200 k. For Site Class C in Boston, and a seismic importance factor of 1.5, the LRFD seismic base shear was calculated to be V_E = 135 k. For exposure B and a wind importance factor of 1.15, the LRFD wind base shear was calculated to be V_W = 278 kips in the short direction and 130 kips in the long direction. Hence, while the lateral system design was controlled by seismic forces in the long direction, in the short direction the building's effective R-factor can be calculated as:

$$R_{eff} = R\left(\frac{1.0V_E}{1.6V_W}\right) = 7\left(\frac{135k}{278k}\right) = 3.4$$

This implies an additional level of safety due to the inherent strength of the EBF. Limits on possible bracing locations reduced the bracing in the building's long direction to one bay on the third floor and one bay on the second floor. Since the building's long direction was controlled by seismic forces, two moment frame bays were also designed for this direction. These moment frames plus the continuity of all of the buildings' columns were intentionally allowed to be more flexible than the EBF system in order to provide building redundancy in the form of reserve capacity, should the EBF bays become compromised.

The 8-in.-square HSS columns provided adequate capacity in most cases under load combinations, including link overstrength and gravity loads. However, four columns in the center of the structure, supporting 40-ft spans plus several bays of bracing, were designed as built-up sections from ¾-in. plate. Welds were completed on the column faces according to AESS standards, ground smooth and left exposed to view.

In order to match as closely as possible the test configuration from Okazaki and Engelhardt, W10×19 links were connected to end plates by two-sided fillet welds that are one-and-one-half times the size of the flange or web. Weld tabs were provided at the flange edges to "avoid introducing undercuts or weld defects at these edges" [Okazaki and Engelhardt 2007, p. 761]. End plates forming the transition between the W10 shear link and the W16 beam were specified as 1 in. thick. System performance was found to be excellent for a range of link sizes studies in the context of non-linear time history analyses [Hines and Jacob 2010, Jacob 2010].

The economical construction of this facility demonstrates that an EBF can be designed for a moderate-seismic region and still be economically competitive with more conventional braced frame systems. Expenses incurred via capacity design requirements can be mitigated by selecting the smallest possible links to withstand wind forces, and these link-beam assemblies can be fabricated as a single element in the shop. In the field,

Dudley Square demonstrated that an economically competitive EBF can be designed for a moderate-seismic region.

these built-up link beams and the braces can be erected in a manner similar to a typical CBF with no special detailing requirements. The extra fabrication effort required for the built-up link beams seems to be well worth the reliable safety benefits of providing a robust seismic force resisting system.

From a design point of view, the length of the link is closely related to inelastic drift requirements. The 2-ft links in this design were considered by the AISC *Seismic Provisions* to have an available link rotation angle of 0.08 radians, whereas the 4-ft links were considered to have only a 0.02 radian link rotation angle. Further testing of continuous link beams with longer links and flange yielding outside the link region could help to create more latitude for designers in moderate-seismic regions, where drift demands are expected to be significantly lower than in high-seismic regions. Results reported by Engelhardt and Popov [1992] for beams outside of links that were overloaded axially ($\pm 0.7P_y$) and in bending, intentionally to violate capacity design principles, still allowed links to achieve approximately 0.02 radians of plastic rotation. Since the test setup for this study did not include a slab, the links were framed into columns on one end, and the tests were designed to illustrate poor performance with flexible braces that allowed most of the moment to be taken by the beam. What was considered poor performance for high-seismic regions may yet imply superior performance when compared to low-ductility, low-reserve capacity CBF designs in moderate-seismic regions.

Looking Forward

In high-seismic regions, structural designers associate an expected level of seismic performance with the buildings that they design. Although the urgency associated with seismic design in moderate-seismic regions is understandably reduced, designers should still envision intended inelastic response targeted for acceptable performance when proportioning building systems [Hines and Fahnestock 2010].

In moderate-seismic regions, there is an increased tolerance of damage due to a large earthquake, so collapse prevention is the dominant performance objective. Ductility and reserve capacity are both viable approaches to achieving collapse prevention in moderate-seismic regions—and in both approaches, the relative strengths and deformation capacities of the system elements are critical considerations. Current research is developing a framework for employing reserve capacity in moderate-seismic design. ■

References

- Bradley, C.R. (2015) Experimental Investigation of Post-Elastic Failure Mechanisms in Low-Ductility Braced Frames and the Implications for Collapse Performance, Masters Thesis, Department of Civil and Environmental Engineering, Tufts University, August, 604 pp.
- Davaran, A., Beland, T., Fahnestock, L.A., Hines, E.M. and Tremblay, R. (2014) "Experimental Behavior of Low-Ductility Brace Connection Limit States," *Structures Congress*, ASCE, Boston, April 3–5.
- Engelhardt, M. and Popov, E. (1992) "Experimental Performance of Long Links in Eccentrically Braced Frames," J. Struct. Eng., 118(11), 3067-3088.
- Fahnestock, L.A., Hines, E.M., Tremblay, R., Bradley, C., Nelson, J., Beland, T., Davaran, A., Sizemore, J. (2014) "Reserve Capacity and Implications for Seismic Collapse Prevention for Low-Ductility Braced Frames in Moderate Seismic Regions," *Tenth National Conference on Earthquake Engineering*, EERI, Anchorage, AK, July 21–25.
- Hines, E.M., Appel, M.E. and Cheever, P.J. (2009) "Collapse Performance of Low-Ductility Chevron Braced Steel Frames in Moderate Seismic Regions," *AISC Engineering Journal*, 46(3), Third Quarter, pp. 149–180.
- Hines, E.M. and Fahnestock, L.A. (2010) "Design Philosophy for Steel Structures in Moderate Seismic Regions," *9th US National and 10th Canadian Conference on Earthquake Engineering*, EERI/CAEE, Toronto, Canada, July 25–29.
- Hines, E.M. and Jacob, C.C. (2010) "Eccentric Braced Frame System Performance," ASCE Structures Congress, Orlando, Florida, May 12–15.
- Jacob, C.C. (2010) "Creating an Eccentrically Braced Frame Model for Application in Seismic Performance Assessments," MS Thesis, Department of Civil and Environmental Engineering, Tufts University, 137 pp.
- Nelson, J., Davaran, A., Beland, T., Bradley, C.R., Hines, E.M., Tremblay, R., Fahnestock, L.A. (2014), "Cyclic Experimental Behavior of Angles and Applications for Connection Design and Modeling", *Proceedings of the 2014 ASCE Structures Congress*, Boston, MA, April 3–5.
- Okazaki, T. and Engelhardt, M.D. (2007) "Cyclic loading behavior of EBF links constructed of ASTM A992 steel," *Journal of Constructional Steel Research*, 63, pp. 751–765.
- Sizemore, J., Davaran, A., Fahnestock, L.A., Tremblay, R. and Hines, E.M. (2014) "Seismic Behavior of Low-Ductility Concentrically Braced Frames," *Structures Congress*, ASCE, Boston, April 3–5.
- Stoakes, C.D. and Fahnestock, L.A. (2011) "Cyclic Flexural Testing of Concentrically-Braced Frame Beam-Column Connections," *Journal of Structural Engineering*, ASCE, 137 (7): 739–747.