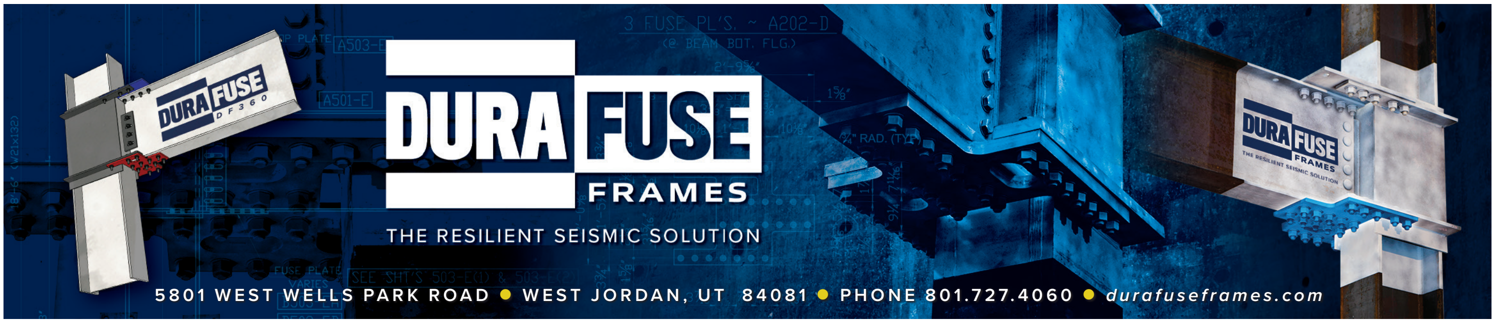

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TECHNICAL BULLETIN 20

Improving Functional Recovery by Using Replaceable Fuses

Paul Richards, PhD, PE[1]

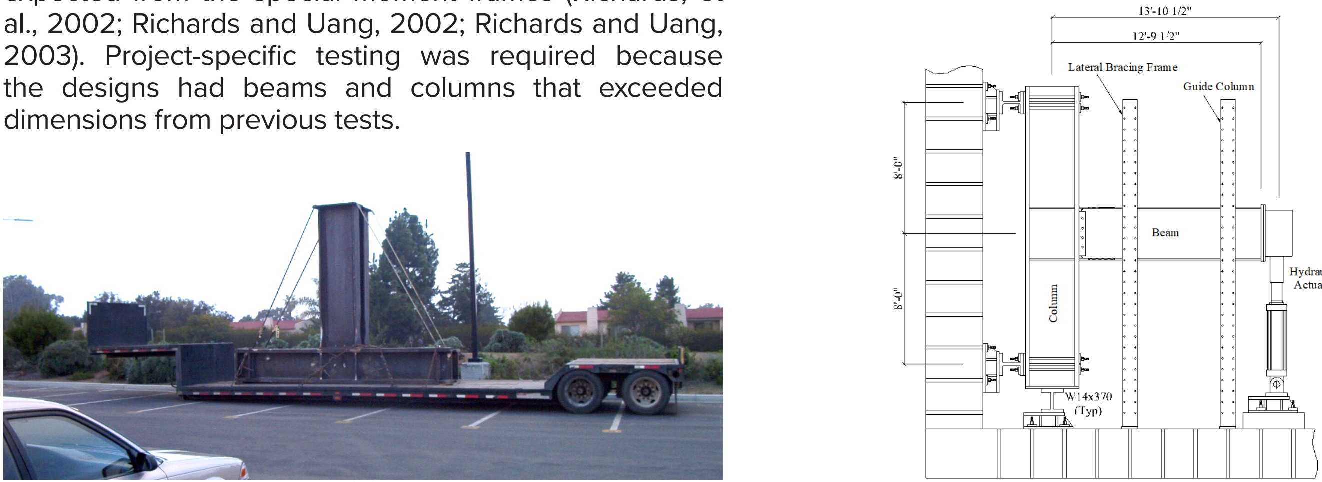
Abstract: Current building codes rely on high ductility systems to resist earthquakes safely. Buildings designed in this manner may be impractical to repair following a severe earthquake. Efforts are being made to improve building codes to address re-occupancy and functional recovery. One option is to design more structures as Risk Category IV. This would be costly and may backfire for some systems that already have low periods and are in the acceleration sensitive region of the response spectra. A complimentary or alternative option is to improve the repairability of structures so they can be returned to service after severe earthquakes. Various replaceable fuse concepts have been explored. The most practical concepts are already being used in practice. The most repairable concept for braced frames is BRBFs. The most repairable concept for moment frames is DuraFuse Frames (DFF). FEMA P-58 analysis confirms that DFF moment frames are a cost-effective path to functional recovery.

This technical bulletin summarizes the content from a presentation given by Dr. Richards at the SEAU Continuing Education Conference on March 3, 2021.

Background

After the Northridge Earthquake, there was considerable concern about the safety of steel special moment frame (SMF) buildings. As a graduate student at the University of California, San Diego (UCSD), I participated in several project-specific testing programs to verify that particular SMF connections could provide the ductility that was expected from the special moment frames (Richards, et al., 2002; Richards and Uang, 2002; Richards and Uang, 2003). Project-specific testing was required because the designs had beams and columns that exceeded dimensions from previous tests.

The tests were conducted per the procedures that are now found in AISC 341 Chapter K (AISC, 2016). Fig. 1(a) shows two specimens that had RBS connections and deep built-up columns. Fig. 1(b) shows the test set-up. A hydraulic actuator, attached to the beam via a corbel, applied the standard loading protocol. The RBS specimens experienced inelastic local buckling around 0.03 rad that resulted in strength degradation as deformations



(a)

(b)

Fig. 1 *Special moment frame testing: (a) specimens, (b) test set-up. Images from Richards, et al. (2002)*

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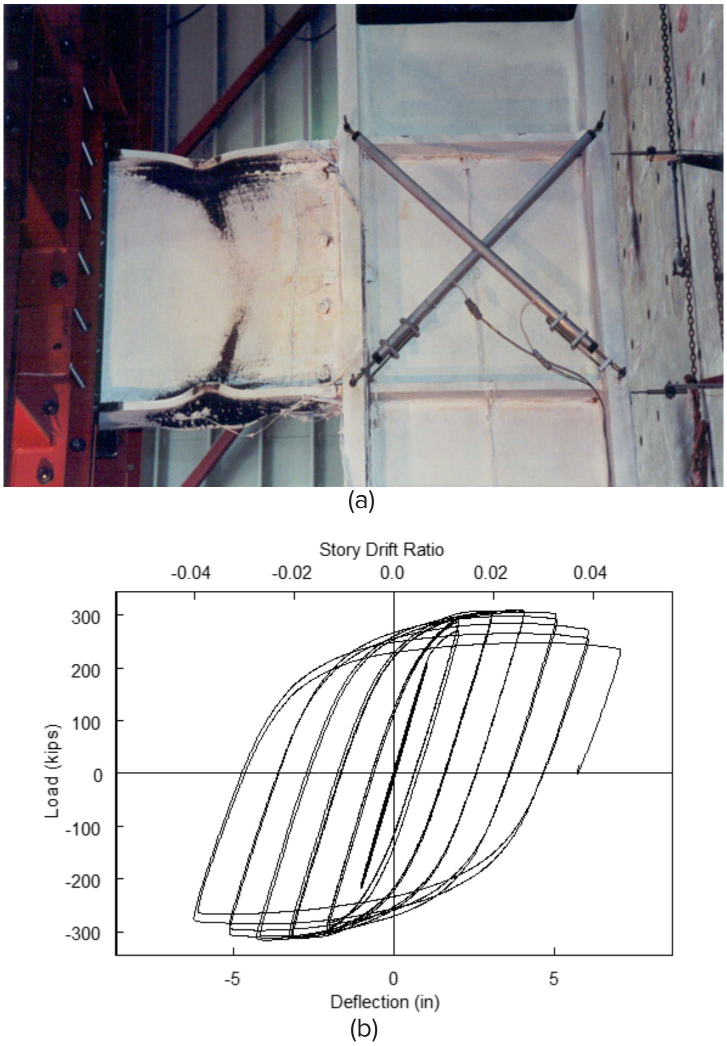

(a)

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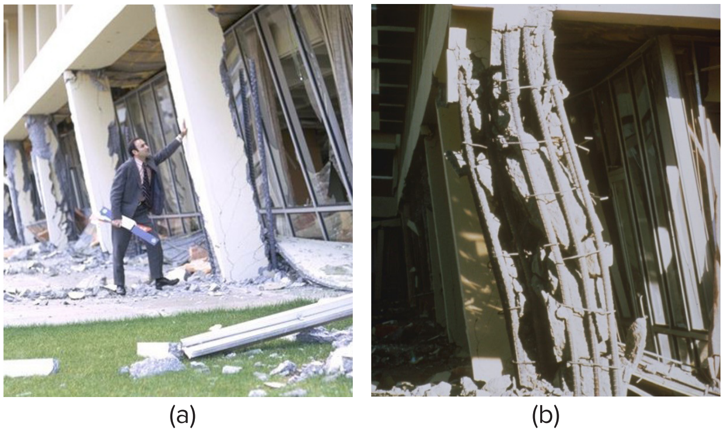
Fig. 2 *Special moment frame testing: (a) connection at 4% drift, (b) hysteretic behavior. Images from Richards, et al., 2002, and Richards and Uang, 2002*

increased. The objective of the tests was to complete two cycles at 0.04 rad prior to excessive strength degradation or material fatigue. Fig 2(a) shows a specimen at 0.04 rad and the hysteretic plot in Fig. 2(b) shows the connection did not have excessive strength lost prior to completing 0.04 rad cycles.

All the moment frame tests I helped with at UCSD confirmed that properly detailed SMF connections have sufficient ductility. However, that ductility comes at a price. Connections that look like Fig. 2(a) may be impractical to repair following a severe earthquake.

Our codes have improved building safety over the past thirty years but have not addressed functional recovery. Fig. 3 shows the St. Olive View Medical Center after the 1971 San Fernando Earthquake. While we provide much more transverse reinforcement for our columns now, we still design special concrete moment frames with an R factor of 8 such that frames will experience inelastic drifts which may be impractical to repair. Thus, our buildings are safer than the one show in Fig. 3 but not necessarily more repairable.



(a)

(b)

Fig. 3 *St. Olive View Medical Center after 1971 San Fernando earthquake: (a) residual drift, (b) poor column confinement. Photo credit: NOAA/NGDC, U.S. Geological Survey.*

The effects of our “safe-but-not-repairable” approach to design will be like what was observed following the 2011 Christchurch earthquake. While few buildings collapsed in Christchurch, 70% of the buildings in the Christchurch business district had to be demolished because they were not practical to repair (Spencer, 2016).

Two criteria will dictate the need for building repair after an earthquake. The first criteria, and the one that will likely govern in most cases, is residual drift. Buildings that are out-of-plumb following a severe earthquake will need to be repaired or demolished. Residual drift is simple to quantify but challenging to fix. If structural steel or rebar has yielded throughout the building, residual drifts will be locked in and may be difficult to unlock. The second criteria that may dictate structural repair is used-up ductility capacity. Some buildings may have minimal residual drift; however, if they have experienced multiple excursions to large drifts, the ductile elements in the structure may be close to fatigue. The remaining ductility capacity in elements will be both difficult to quantify and difficult to address if found lacking.

There is a growing realization that “safe” is not good enough for a lot of our infrastructure and that many of our buildings will not be repairable. In November, 2020, David Bonowitz gave a webinar sponsored by the Earthquake Engineering Research (EERI) and the Utah Resiliency Council on functional recovery. Much of the content from that presentation can be found in a white paper that Bonowitz authored with EERI titled *Functional Recovery: A Conceptual Framework with Policy Options*. The paper encourages the use of specific terms like “re-occupancy” and “functional recovery” when discussing building performance. Functional recovery is defined as “a post-earthquake state in which capacity is sufficiently maintained or restored to support pre-earthquake functionality ”(EERI, 2019).

The white paper outlines some potential policy options to address functional recovery in building codes:

> *Possible parallel strategies for buildings and lifeline infrastructure…could include: [requiring] certain government-funded construction projects (at all levels) to use building code provisions for the highest risk category, Risk Category IV, even where the building use would not be considered “essential” by the current code… [and/or requiring] certain private*

> *construction projects normally assigned to lower risk categories, Risk Category II or III, to use building code provisions for Risk Category IV.*

While I am an advocate of functional recovery and support the efforts represented by the paper, there are a few points to keep in mind when contemplating policies like increasing the number of buildings that are designed as Risk Category IV.

Consequences of Risk Category IV Design and Alternative Approaches to Improved Functional Recovery

Fig. 4 illustrates the controls for structural design that are defined in current building codes. The control "levers" that are defined by the code are ductility, strength, and stiffness (Fig. 4). In most ductile steel systems, the strength and the stiffness are coupled such that one cannot be changed appreciably without affecting the other, which is why the "strength" and "stiffness levers" in Fig. 4 are linked.

In current seismic design (ASCE, 2016), we adjust the ductility lever based on the seismic design category, and then determine required setting for strength and or stiffness based on safety considerations. When we put the ductility lever up to maximum, we know that our buildings are going to be damaged which may make functional recovery difficult.

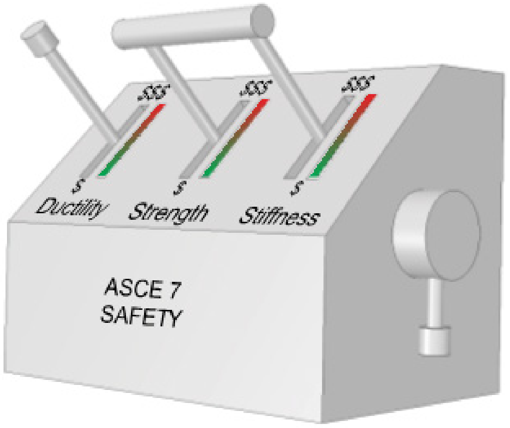


Fig. 4 *The controls for structural design as defined in building codes*

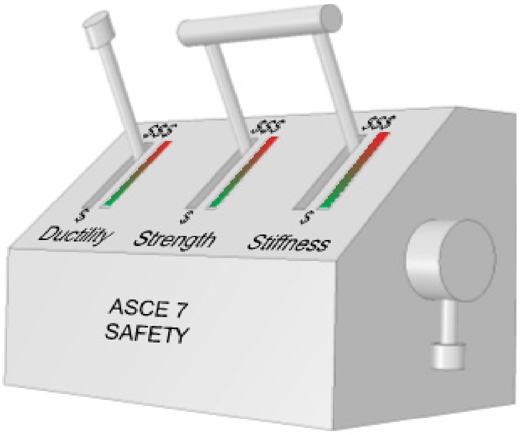


Fig. 5 *Control settings for Risk Category IV design*

For Risk Category IV buildings, we increase strength and stiffness in an attempt to reduce the damage that will make functional recovery difficult. This is analogous to also pushing the strength and stiffness levers on the controls to the top (Fig. 5).

In the following sub-sections, I will suggest reasons why this approach might not be the most economical way to address functional recovery.

Un-used ductility

In some respects, designing structural systems with high ductility capacity and then imposing a 1 or 1.5% drift limit is like engineering a sports car that can go 240 mph and then putting in a speed limiter that prevents it from going above 60 mph. It can be done, but it does not make economic sense. We are paying for ductility that we prefer not to use since ductility means structural damage, and structural damage means slow, expensive, and impractical functional recovery. In current design for Risk Category IV structures, the high ductility is used as an insurance policy that is not expected to be called on.

I am not suggesting that we use low ductility systems for important structures; however, I do believe there is a way to leverage ductility and mitigate functional recovery problems without pushing the strength and stiffness levers to maximum. There may be more cost-effective methods for addressing functional recovery than combining strict drift limits with high-ductility systems (Fig. 5).

Issues in the Acceleration Sensitive Portion of the Response Spectra

One of the reasons for imposing higher strength requirements and drift limits on Risk Category IV structures is that, in the velocity sensitive region of the spectra, higher stiffness and strength means lower ductility demands on the structure, and lower ductility demands imply less damage and easier functional recovery. The situation is different in the acceleration sensitive region of the response spectra (shorter period structures on or near the plateau of the design spectra). In this region, ductility demands can be significant, even with very high strength (Chopra, 2012).

Designing buildings to Risk Category IV does not necessarily guarantee better functional recovery. Fig. 6 shows results from non-linear response history analysis (RHA) of two elasto-plastic systems with the same mass subjected to the same earthquake time history (1994 Northridge Canoga Park). The first system had a stiffness of 50 k/in and a strength of 250 kips. The second system had twice the stiffness and strength. The results show that while the stiffer and stronger system had reduced maximum drift, the residual drift was the same in both cases (4 inches). The residual drift is a strong indicator of functional recovery cost (time and money) and, by this measure, the stronger and stiffer design showed no benefit in this case.

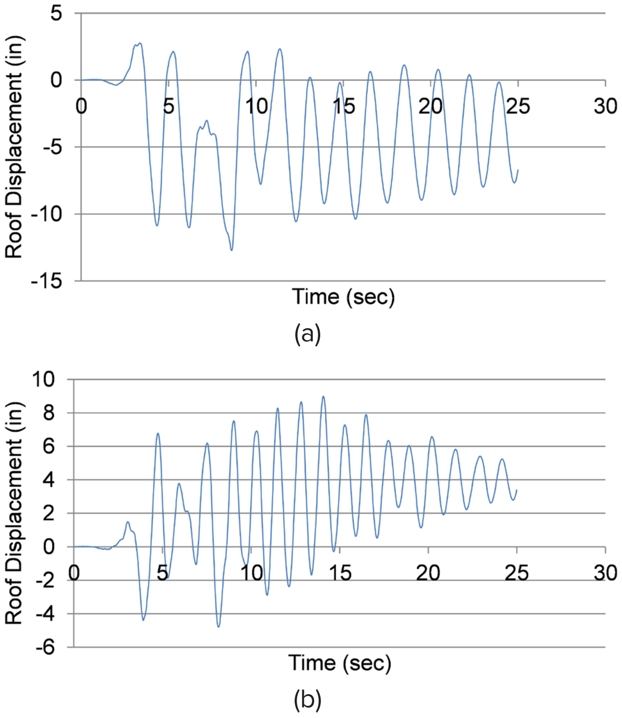


Fig. 6 *Nonlinear RHA results for two systems with same mass but different strength and stiffness: (a) baseline system (T=1.78 sec), (b) system with double strength and stiffness (T=1.26 sec)*

Because of the sensitivity of building response to the particulars of the ground motion, the only thing that is guaranteed with Risk Category IV design is that the structure is going to be more expensive to build initially.

Difficulties in Accurately Estimating Inelastic Drifts

The last thing to keep in mind, as we consider policy recommendations based on drift limits, is that we do not have accurate design tools for determining maximum drifts in buildings. Equation 12.8-15 in ASCE 7-16 (ASCE, 2016) that we commonly used for estimating drifts, is known to be inaccurate and can be off by a factor of two or more as compared to results from non-linear RHA (Karavasilis, et al., 2007; Medina and Krawinkler, 2005; Richards and Thompson, 2009; Uang and Maarouf, 1994). Equation 12.8-15 is often non-conservative for lower-period structures and overly conservative for higher-period structures. The accuracy of Equation 12.8-15 is dramatically different from the other things that we do in structural engineering, yet we generally speak of maximum drifts as something we can reasonably control and predict.

The inability to predict drift in design is one of the reasons why steel moment frame connections have to be able to withstand story drifts of 4% (AISC, 2016) even though we are never allowed to design buildings for more than 2.5% (ASCE, 2016). It is questionable to base a functional recovery strategy on a parameter that we cannot predict accurately in design.

Fig. 7 *The repairability lever is a way to control functional recovery*

Alternative Approach to Functional Recovery

Recognizing that improving functional recovery is not as simple as just increasing strength and stiffness, and recognizing the cost implications of such a policy, I think we should discuss an alternative approach. The remainder of this bulletin discusses another “lever” that is available to help control our building functional recovery (Fig 7.). That lever is repairability.

Most structural engineers have not given much thought to repairability because it is not mentioned in our building codes. The reason repairability is not mentioned in our current codes is because it has nothing to do with safety, which is the focus of our current codes.

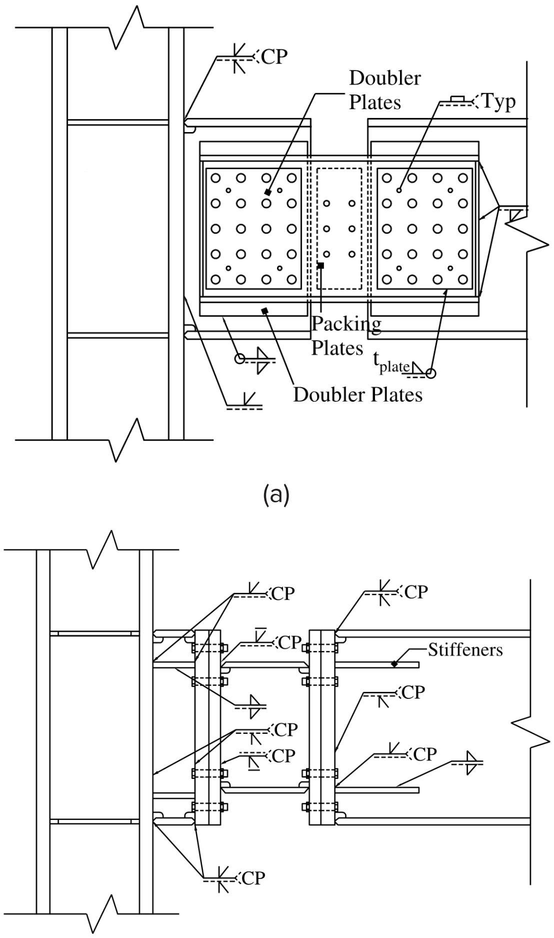
In the sections that follow, I will discuss some examples of repairable systems from the literature and some examples of repairable systems that are being used in practice. The final section of the bulletin discusses how using repairable systems may be a more economical approach to functional recovery than increasing strength and stiffness requirements.

Repairability Concepts from the Literature

The development of repairable structural systems has been an active area of research for the past twenty years. This section will discuss some concepts that work in the laboratory but have not seen widespread use in the U.S.

Moment Frames with Replaceable Connections

The first concept that will be discussed is moment frames with replaceable connections as reported by Shen, et al. (2011). They performed experimental testing on moment frame connections that introduced replaceable elements near the ends of the beams. One detail they tested had two channels that sandwiched the beam web at the slice [Fig. 8(a)]. Another detail incorporated a link beam with end plates [Fig. 8(b)]. In both cases, the replaceable elements were proportioned to have about one-third of the flexural capacity of the beam to ensure inelastic



(a)

(b)

Fig. 8 *Moment frame connections with replaceable elements near the beam ends: (a) two channels that sandwich the beam at the splice, (b) link beam with end plates. Figures from Shen, et al. (2011)*

behavior would be confined to the replaceable element and to make the connections reasonable.

Both details were tested in the laboratory following the procedures outlined in AISC 341 Chapter K (AISC, 2016). Both details worked as intended and prevented damage to the beam during testing. Fig. 9(a) shows the double channel detail at large drift and there is clearly no beam damage. The connection completed cycles at 7% drift [Fig. 9(b)] which is well beyond the minimum required for qualification (AISC, 2016). The other detail, with a link beam, also completed cycles beyond 4% drift (Fig. 10).

These concepts, as tested, had lower strengths and stiffness than typical moment frames. The flexural capacity of the replaceable elements was only 33-40% of the flexural capacity of the beam.

Despite successful demonstration of the concepts, these details have not seen use in U.S. practice. One drawback of these details is that they result in more expensive moment frames than current practice. The link elements introduce flexibility to the moment frame that would result in heavier required beams and columns to meet drift requirements. The detail with channels requires doubler plates on the beam webs and the detail with an end-plated

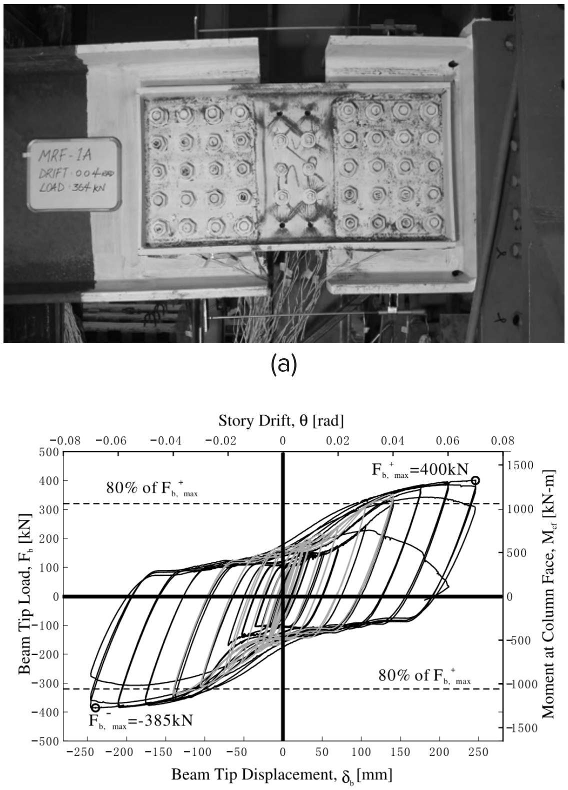


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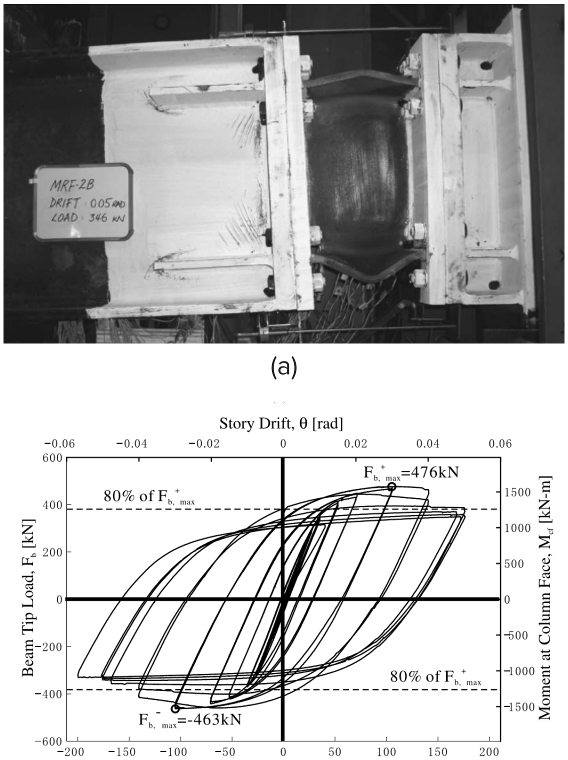
Fig. 9 *Experimental testing of moment frame connections with replaceable channels: (a) connection at large inelastic drift, (b) hysteretic behavior. Figures from Shen, et al. (2011)*



(a)

(b)

Fig. 10 *Experimental testing of moment frame connections with replaceable beam link: (a) connection at large inelastic drift, (b) hysteretic behavior. Figures from Shen, et al. (2011)*

link requires ten CPJ welds per connection; both of which would increase fabrication costs. Another drawback of these types of repairable moment connections is that the beam would require shoring during repair.

Replaceable Shear Links

Several studies have investigated the use of replaceable shear links in eccentrically braced frames (EBFs) and other applications. In traditional EBFs, a segment of the beam is designed to yield and, after a severe earthquake, the entire beam may require repair. This would be complicated by the fact that the braces in EBFs typically have moment connections to the beam. Several researchers have investigated EBFs where the yielding link is designed as a distinct, replaceable element (Fig. 11).

In a recent paper, Bozkurt, et al. (2019) investigated the concept shown in Fig. 12. This type of configuration may be easier to repair when there is some residual drift remaining prior to the installation of the replaceable components. Large-scale testing demonstrated excellent hysteretic behavior for the links (Fig. 13) and that repairability in the lab was possible.

Despite successful demonstration of the concept, EBFs with replaceable links have not seen use in U.S. practice. One drawback of the system is the complicated detailing. EBF design and detailing is already viewed as somewhat complex and this adds another layer of complication. Another challenge is that in a building application, the slab-on-metal-deck would need to be removed and the beams shored to accommodate the replacement of the link. The greatest challenge for concepts like these, however, is they need to be competitive relative to other replaceable fuse options. As will be discussed in the next section, buckling restrained braced frames (BRBFs) provide all the benefits of replaceable EBFs while having less complicated detailing and replacement procedures.

Replaceable shear links have been used in contexts other than EBFs in the U.S. The tower of the east span of the San Francisco-Oakland bay bridge has replaceable shear links that couple the tower's vertical elements.

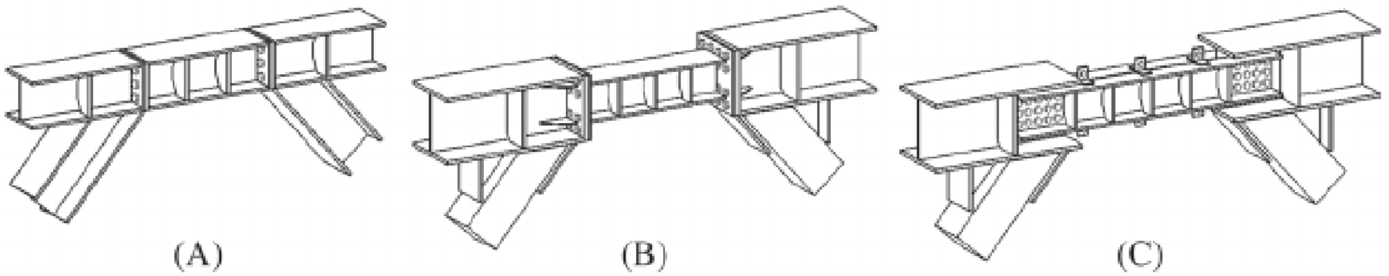


Fig. 11 *Replaceable EBF link concepts. Figure from Bozkurt, et al. (2019)*

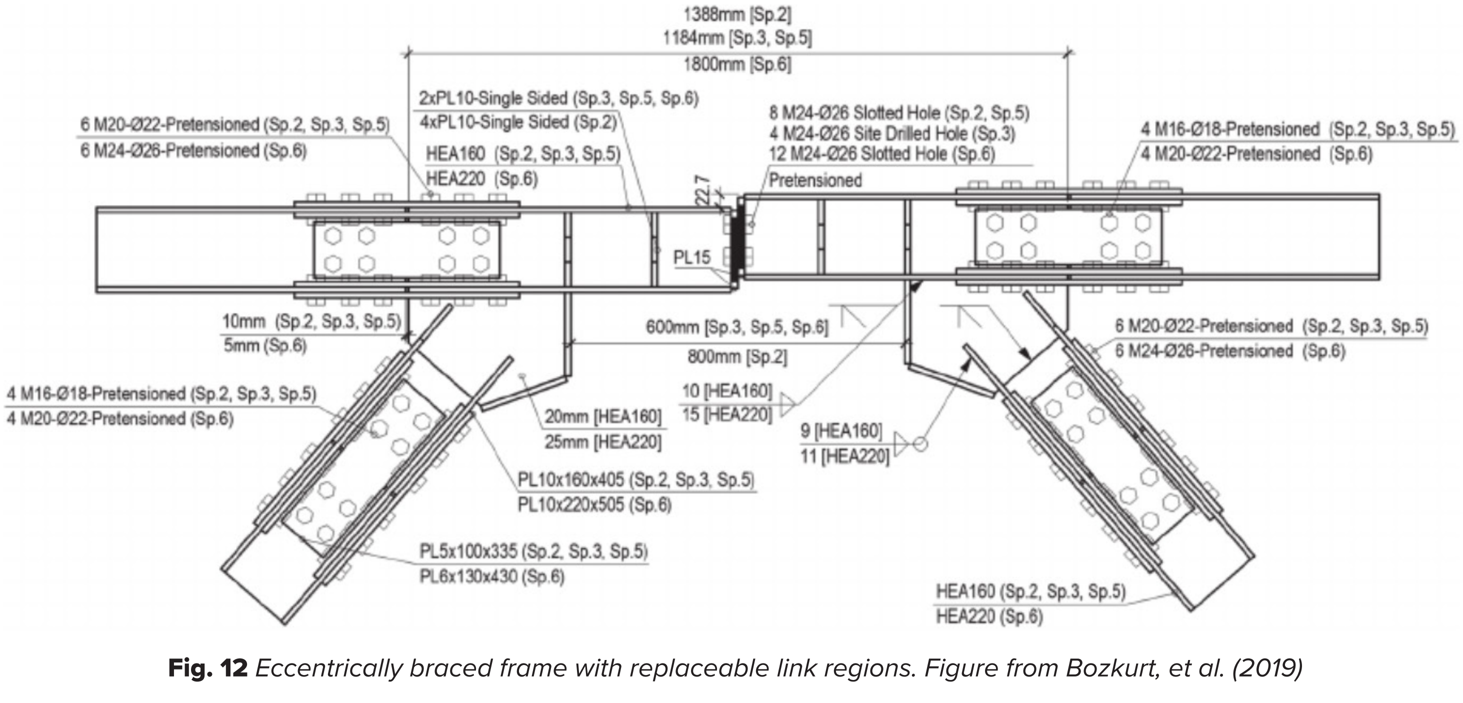


Fig. 12 *Eccentrically braced frame with replaceable link regions. Figure from Bozkurt, et al. (2019)*

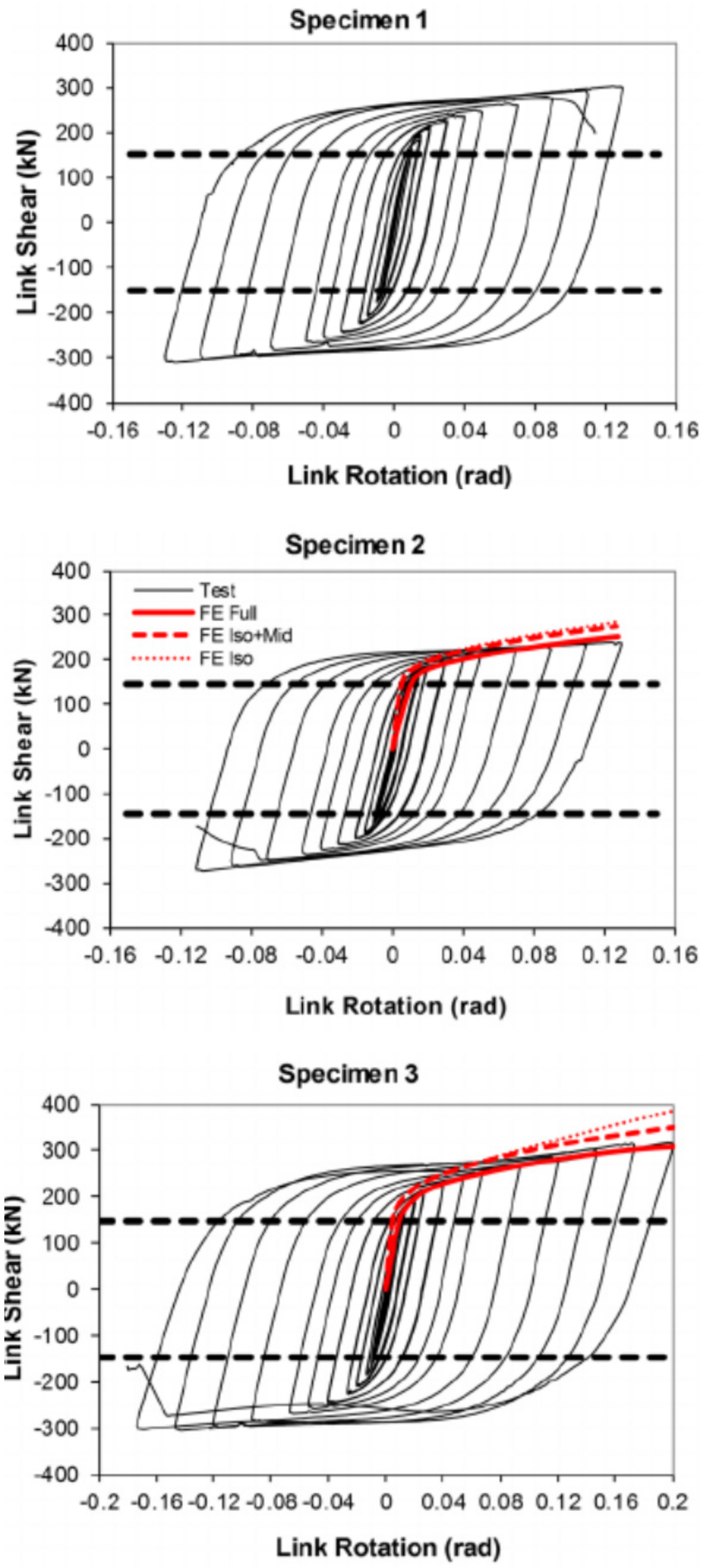


Fig. 13 *Hysteretic response of replaceable EBF link components. Figure from Bozkurt, et al. (2019)*

The ductility capacity of the shear links was investigated through experimental testing (Fig. 14) prior to construction of the bridge (McDaniel, et al., 2003). Cyclic testing demonstrated good hysteretic behavior.

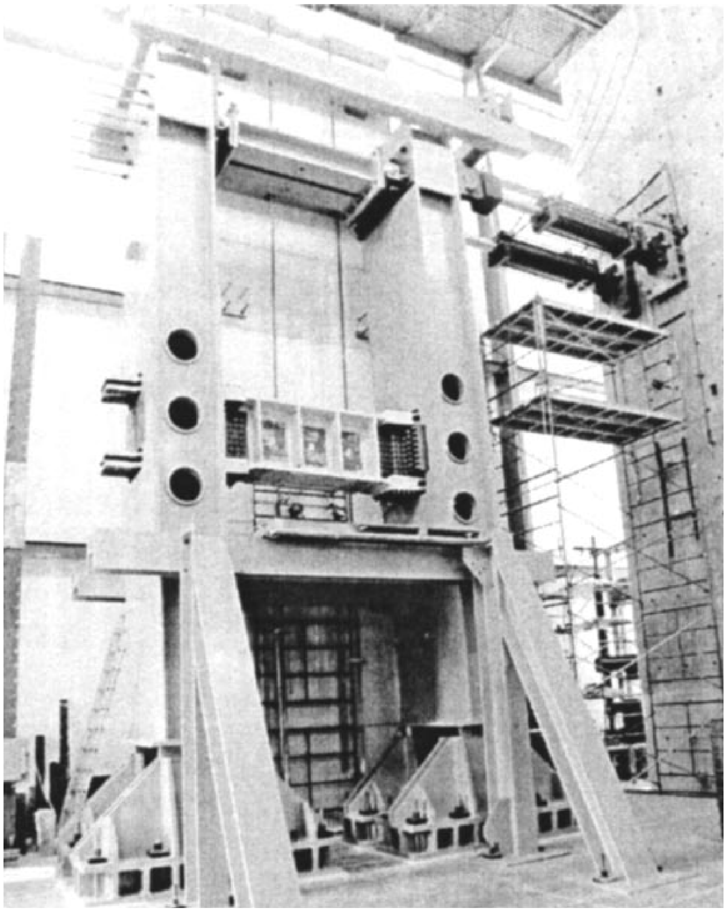


Fig. 14 *Experimental validation of replaceable shear links for bridge tower. Figure from McDaniel, et al. (2003)*

Replaceable Structural Fuses in Practice

In the last twenty years, replaceable structural fuses have made their way into U.S. practice in the form of steel buckling restrained braced frames (BRBFs) and special moment frames (SMFs) with replaceable fuses. Designing buildings that can be repaired is a cost effective strategy for functional recovery after severe earthquakes. Using the repairability "level" in design (Fig. 7) may be a more economical approach to functional recovery than pushing the strength and stiffness levers to maximum (Fig. 5).

Buckling Restrained Braced Frames

Buckling restrained braced frames are commonly used in high-seismic regions and have several features that give them enhanced repairability compared to other structural systems (Fig. 15). First, the inelasticity in a BRBF is concentrated into the BRBFs and the beams and columns remain elastic. Second, BRBFs are designed to carry gravity loads when the braces are absent, so braces could be replaced after a severe earthquake without shoring the beams. Third, gusset plates in BRBFs are designed to remain elastic and would not have to be replaced. Fourth, BRBs are often bolted in place, making removal and replacement relatively simple. And finally, instruments can be incorporated into BRBFs to measure displacements and provide data that could be used to justify not replacing the braces, if residuals drifts are not a problem.

Fig. 15 *BRBFs have several features that provide enhanced repairability*

While BRBFs have enhanced repairability, they also provide the best upfront economy for many buildings. The inherent stiffness in braced frames allows BRBFs to use less steel than moment frames. The high ductility of BRBFs (R=8) results in lower strength demands as compared to other steel braced frames, concrete systems, and masonry systems. The economy and repairability of BRBFs make them superior to EBFs with replaceable links.

Special Moment Frames with Replaceable Fuses

There are two prequalified moment frame systems that incorporate replaceable fuses and have been used in U.S. practice: Simson Yield-Link and DuraFuse Frames.

The Simpson Yield-Link connection, shown in Fig. 16, incorporates fuses element at the top and bottom of the beam. The fuse elements are proportioned to prevent beam yielding. Additional plates are used to sandwich the fuse elements to prevent buckling when they are in compression. Experimental testing has demonstrated this connection can accommodate large drifts without any yielding in the beam.

The DuraFuse Frames (DFF) connection incorporates a shear yielding fuse plate at the bottom flange level. Fig. 17 shows the overall geometry and Fig. 18 illustrates how the fuse yields under severe loading. Compared to the other moment frame concepts that have been discussed in this paper, the DFF connection has the advantage of being a fully restrained (FR) and full-strength connection, so frame weights are comparable-to or less-than traditional moment frames.

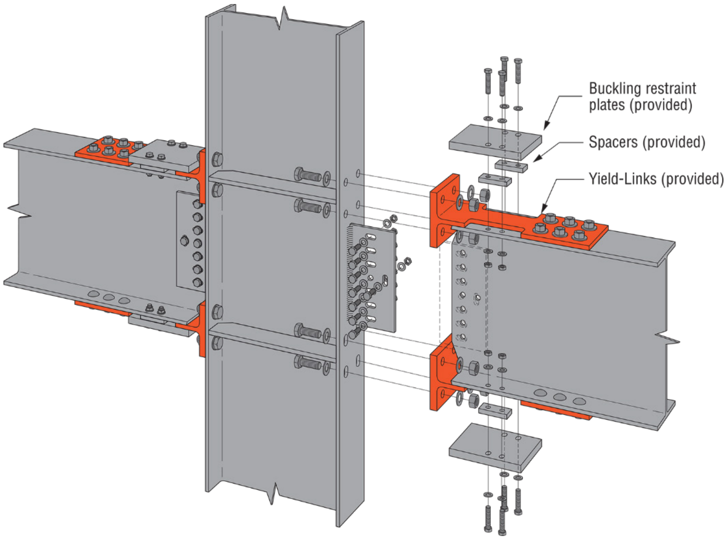


Fig. 16 *Simpson Yield Link moment connection. Figure from Simpson*

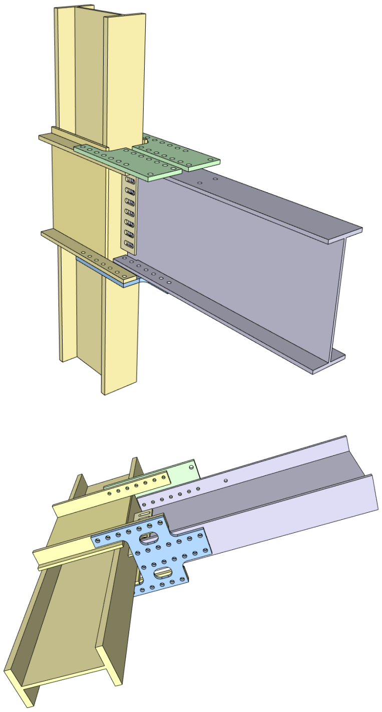


Fig. 17 *DuraFuse Frames moment connection*

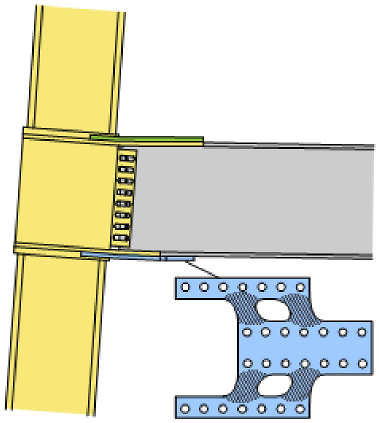
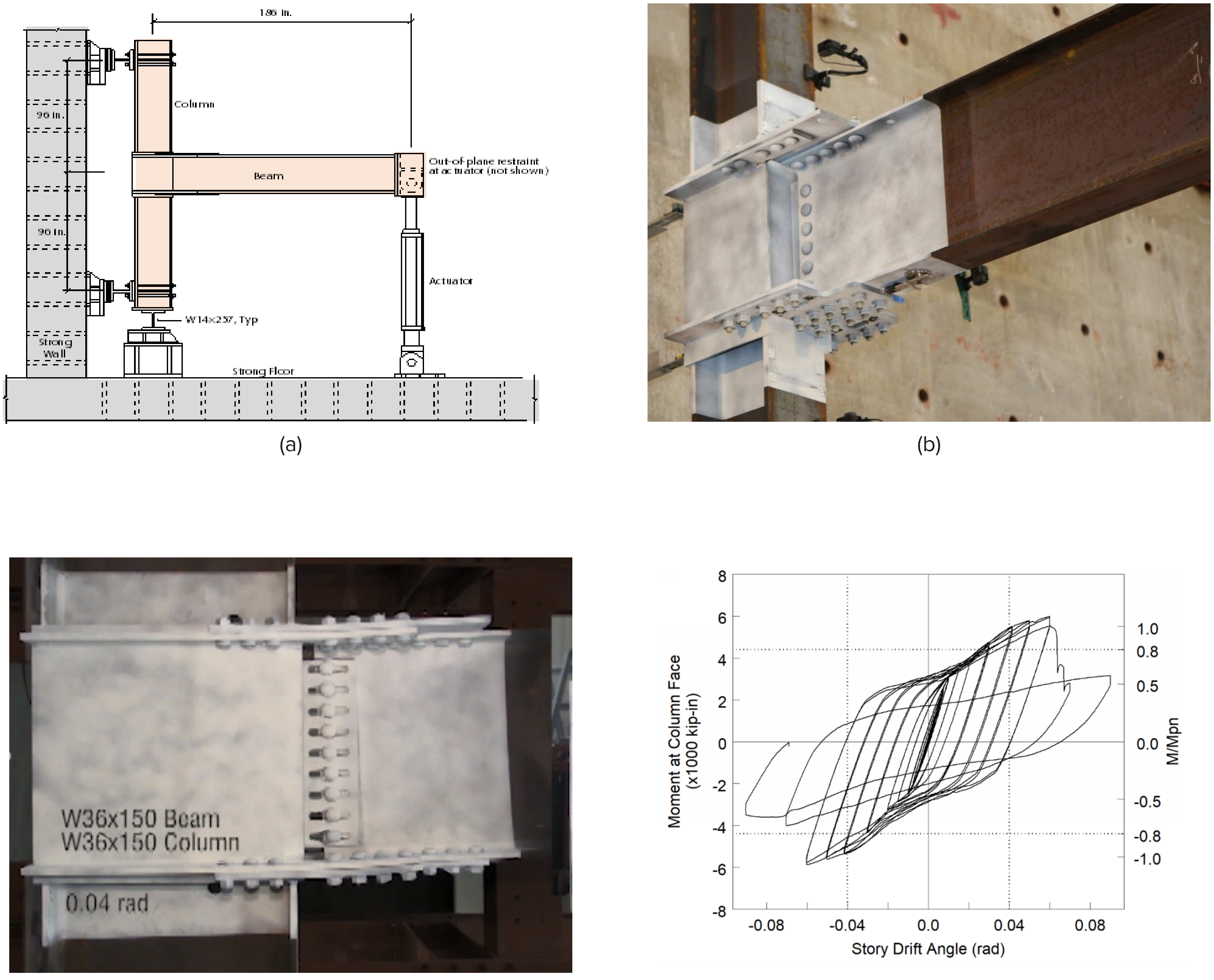


Fig. 18 *DuraFuse Frames fuse plate yielding under severe earthquake loading*

Experimental testing has been performed to prequalify the DFF connection for use. Full-scale testing was performed at UCSD for beams ranging from W21 to W40 (Reynolds and Uang, 2019). Hysteretic behavior was similar to other bolted moment frame connections and drifts of 6% were achieved in the tests [Fig. 19 (d)]. The experiments also demonstrated repairability of the connections. Tests were conducted where the fuse plate was replaced and the connection performance was identical after repair. Repair of the DFF connection is easier as compared to other concepts since no removal of the floor slab is required and the replaceable element is relatively light.

In addition to being more repairable, DFF connections provide the best upfront economy for many buildings. Since DFF connections preclude beam yielding, beam lateral bracing is reduced by about 75% (Reynolds and Richards, 2021). The simple fabrication of the connection (no CJP welds, continuity plates, or doubler plates) reduces shop costs. The field bolted assembly accelerates erection schedules.



(a) (b) (c) (d)

Fig. 19 *Images and results from DuraFuse Frames prequalification testing: (a) set-up for UCSD tests, (b) G-series specimen during testing, (c) B-series specimen at 4% drift, (d) typical hysteretic behavior*

Improved Functional Recovery with DuraFuse Frames

FEMA P58 is a probabilistic performance prediction method that was developed to quantify functional recovery costs (ATC, 2019). Studies have been conducted to compare the recovery costs of DuraFuse Frames systems as compared to traditional steel moment frames (Richards, 2020). DuraFuse Frames systems have dramatically lower functional recovery costs because: DFF systems are less likely to require repair, DFF systems are less expensive to repair when required (replacing a fuse plate instead of a beam), and DFF systems are less likely to have non-repairable residual drifts. FEMA P58 studies have demonstrated that expected annual losses from structural damage, residual drift, and collapse are 60 to 80% lower for DFF systems as compared to traditional moment frames.

Summary and Conclusions

Current building codes are focused on structural safety. Using R-factors in design is an economical way to achieve safe buildings; however, relying on ductile response means that buildings will be damaged during severe earthquakes. When damage occurs in structural elements like beams and walls, it may be impractical to repair buildings.

There is a growing realization that “safe” is not good enough and that buildings codes ought to address re-occupancy and functional recovery. Efforts are underway by EERI to provide guidance on potential policy options. One option is to design more structures as Risk Category III or IV. This approach will increase the cost of structures and is not guaranteed to reduce residual drifts in particular cases. Strengthening and stiffening structures that already have low periods will push them further into the acceleration-sensitive region of the response spectra and may backfire in some cases. Complete reliance on drift control is questionable since our methods for approximating inelastic drifts are crude.

An alternative, or complimentary, approach to functional recovery is to incorporate repairability into design. Repairability has not been given much attention in practice since codes are currently focused on safety. A variety of repairability concepts have been explored that introduce replaceable fuses into structures. Leveraging repairability may be a more economic path to functional recovery.

Some concepts that have been explored in the laboratory are unlikely to see widespread use in practice because there are superior options. Moment frames with replaceable beam link elements can accommodate large drifts, yet introduce expensive details and connection flexibility that results in overall increase in frame weight. Eccentrically braced frames (EBFs) with replaceable links would require floor removal for repair and are more complicated than other options.

Some repairability concepts that are already in use are buckling restrained braced frames (BFBFs) and moment frames with replaceable fuse plates. BRBFs provide excellent elastic stiffness and ductility, and inelastic behavior is confined to the braces which are relatively easy to replace. Simpson Yield Link and DuraFuse Frames (DFF) moment frames have replaceable fuse plates that prevent beam yielding. The DFF fuse plate is at the bottom flange level and can be replaced without removing the slab. All these systems can be repaired without shoring.

FEMA P-58 analyses have been conducted to demonstrate reduction in functional recovery costs by using DFF moment frames rather than traditional SMFs that have beam yielding. Since there are no upfront cost premiums to using BFBFs and DFF systems, they represent an economical approach to managing functional recovery.

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