Considerations Regarding the Repair & Retrofit of Existing Welded Moment Frame Buildings

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1. Introduction

The purpose of this paper is to discuss considerations regarding the assessment of existing welded moment frame buildings their repair and retrofit. After the January 17, 1994 Northridge Earthquake, these buildings were found to be vulnerable due to damage of their joints and may be recognized as life hazardous as a result of partial collapse during a strong earthquake.

We understand that other organizations, including SEAOSC's Existing Building Committee, have taken appropriate measures to develop guidelines which is to be commended.

The emphasis of the ordinances and guidelines are based on current codes including ASCE/SEI 41 and the state of the art. *This document is not intended to critique them but to provide additional considerations to assist structural engineers with the decisions to come up with effective retrofit solutions.*

These considerations are based upon the experiences of the writers before and following the January 17, 1994 Northridge Earthquake, involvement in subsequent repairs and work in committees (Dhalwala, 34 years, Maranian, 25 years).

This paper discusses several issues that, in the opinion of the writers, despite the good work carried out over the last 25 years or so, including that carried out by SAC, AISC and AWS, are not adequately addressed in current codes and standards relating to steel structures.

2. Scope

The discussions in this paper are intended to apply to all buildings using welded moment frames in one or more directions.

Building types should include the following:

i) Single story buildings with flexible or rigid diaphragms including cantilevered columns substantially restrained at the foundation.
ii) Multi-story buildings.
iii) Buildings with a combined system of moment frames and braced frames or shear walls.
iv) Buildings with welded moment frame connection.
v) Several of the discussions in this document may also apply to other lateral resisting systems (e.g. Eccentric Braced Frames (EBFs), Concentric Braced Frames (CBFs), etc.).

3. Discussion

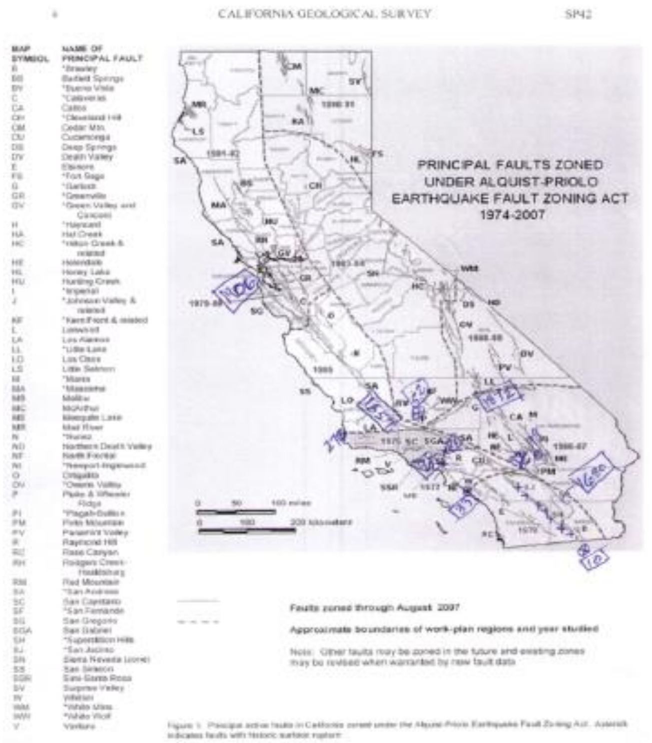
<u>Background</u>: Although the failures of pre-Northridge steel moment frame connections have been well documented and extensive research carried out, particularly by SAC (FEMA 351, 352, 353, 355D & 355E), with relevant codes and standards updated, in the opinion of the writers, current documents do not adequately address all of the underlying problems. The failure of the joints is essentially a fracture mechanics problem, frankly, a subject not well understood by most practicing engineers. The subject is also not well addressed in current building codes and standards in the opinions of the writers. Fractures have repeatedly occurred in many types of structures, over the past century and despite efforts to improve procedures, continue to occur. Maranian (2009), published by ASCE, describes several case histories over the last 90 years or so, discusses fracture mechanics theory, steel material, welding procedures and issues. It should also be noted that cracks in steel moment frame joints, including lamellar tearing, occurred in a 52 story building in Los Angeles in the 1970s [Kaminetzky (1991)] prior to the 1994 Northridge Earthquake.

i) <u>Seismicity</u>: With regard to Southern California, including the Los Angeles Basin, magnitude 7 (M7) strike-slip earthquakes, emanating from the Salton Sea have a return period of 250-280 years. A M8+ earthquake occurred in 1680 (See Figure 3.1).

Based upon deterministic assessment, with regard to the M6+ thrust-fault earthquakes which are due to the "Big Bend" in the fault line, there return period is estimated to occur every 21 years, +/- three years (See Figure 3.2). Thrust faults can have significant vertical accelerations compared with strike-slip, which is not reflected in the current Code.

Furthermore, near field (NF) and far field (FF) can have very different responses as shown in Figures 3.3 and 3.4, derived from Gioncu, Mateescu et al published in

Mazzolani (2000). Due to very high velocity pulses,



M7+ Earthquakes marked in text boxes
Return period at Salton Sea 250-280 years
M8+ occurred in 1680
OVERDUE

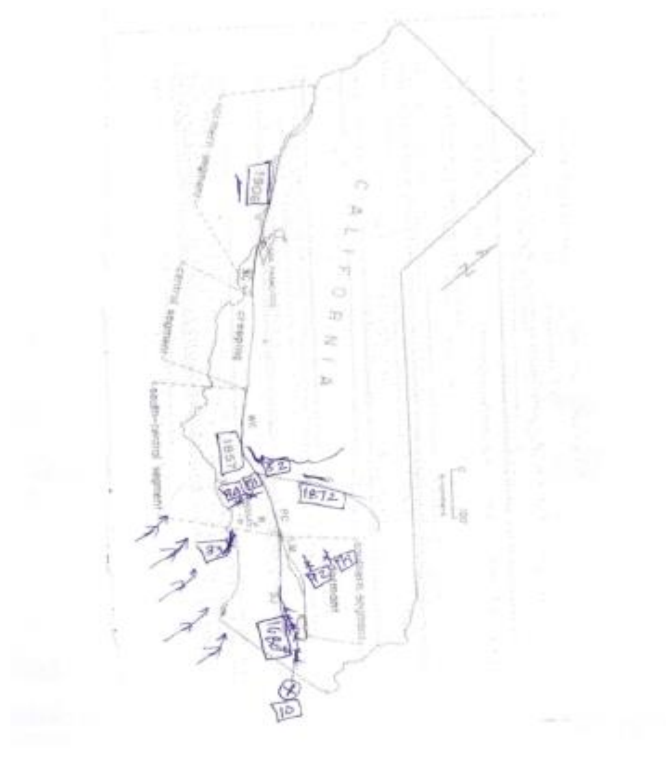
San Andreas Fault M8 + Earthquakes
Figure 3.1

significant vertical axial demands can occur particularly at the bottom stories. The consequential high yield strain demands at significant strain rates, causing the potential for fracture at connections and subsequent collapse, is significantly greater for NF conditions than for FF seismic events. Also, whereas strike-slip earthquakes are primarily dominated by horizontal forces, thrust faulting, can involve both significant horizontal and vertical accelerations, enhancing triaxiality demands on the connections appreciably reducing the ability of the material to perform in a ductile manner.

Identifying geographical areas where there is a potential for NF events along soil seismic characteristics including soil period is warranted.

ii) <u>Potential Issues at Frames and their connections</u>: Issues associated with steel moment frame connections and most other steel lateral resisting systems include, but not limited to, the following:

- Material properties including non-metallic inclusions
- Welding issues including low fracture tough welds, possible poor welding procedures and possible hydrogen embrittlement.
- Size effects
- Low toughness of welds
- Through thickness
- Defects
- Plane strain and tri-axial stress conditions
- Restraint to weld shrinkage
- Low-cycle fatigue
- Stress concentrations
- Strain concentrations
- Local buckling
- Heat affected zones
- Low Temperature

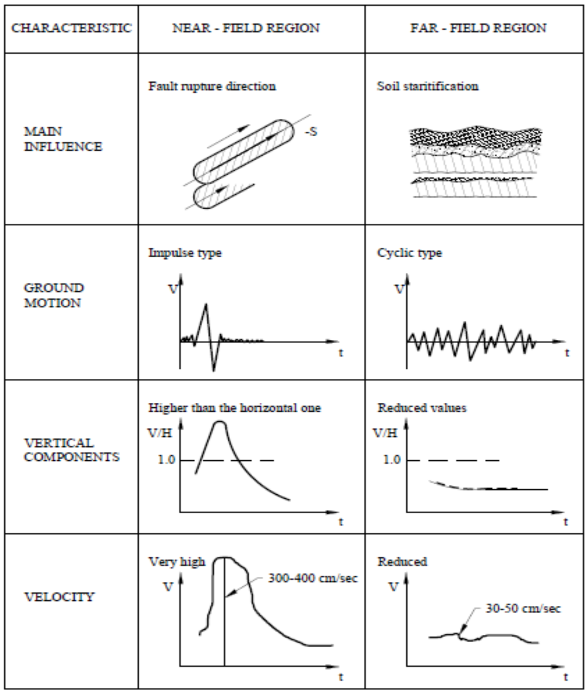


- Estimated return period of thrust faulting
- After 1971 Earthquake = 300 yrs
- Deterministic assessment
- M6+ estimated to occur every 21+/-3 yrs
- Northridge EQ occurred after 23 years
- **OVERDUE**

M6 + Earthquakes due to the Big Bend
Figure 3.2

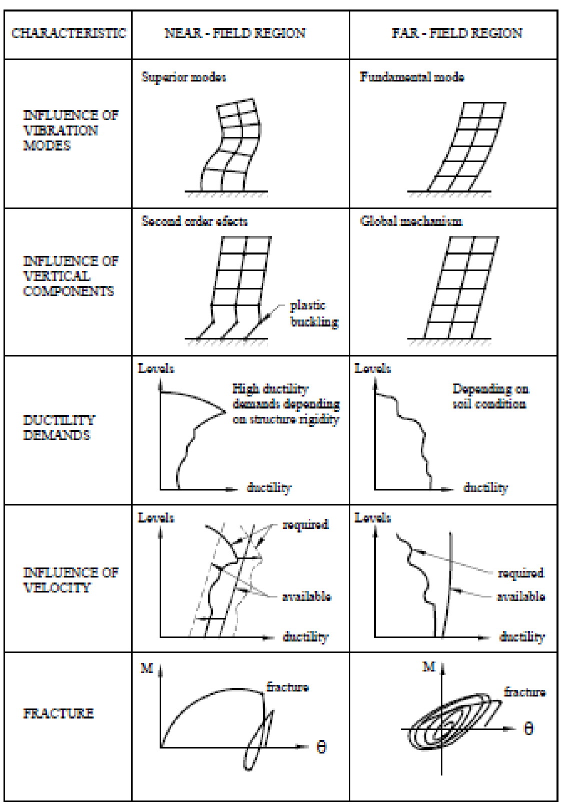
Assessment of several of these issues, associated with existing buildings, particularly with establishing material properties, is very difficult. Application of new welds can themselves pose issues including restraint to weld shrinkage, establishing proper welding procedures to avoid unacceptable defects, hardness and low fracture toughness. It is even possible that new welds, not done correctly, could cause further problems.

Therefore, some degree of the practical aspects of effecting repairs and retrofit, along with economic considerations, needs to be carefully considered.



Characteristics of the near-field and far-field ground motion

NF & FF Ground Motions
Ref: Mateescu et. Al
Figure 3.3



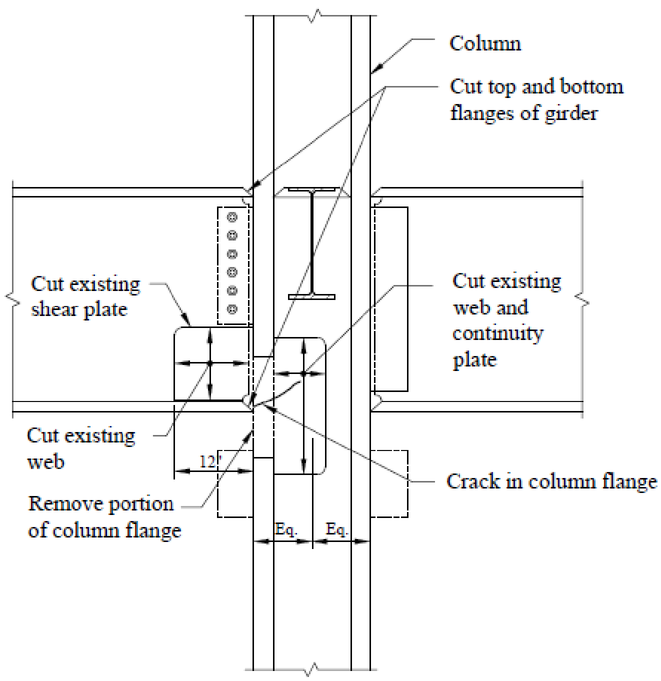
Characteristics of the structural behaviour

NF & FF Structural Responce
Ref: Mateescu et. Al
Figure 3.4

4. General Building/Structure Information

The general information that, as a minimum should be obtained, includes the following:

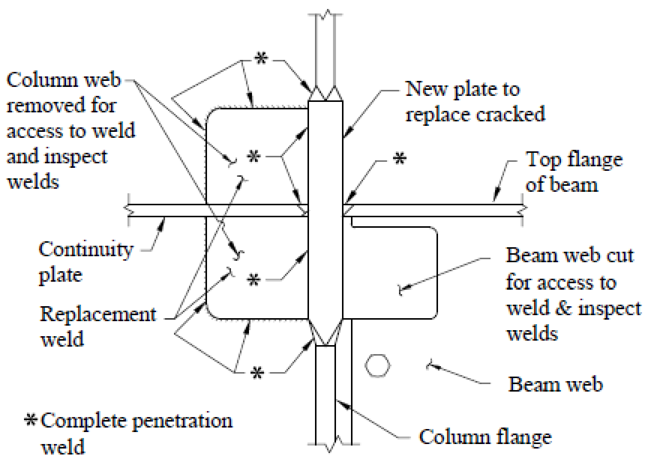
i) Year when built.
ii) Number of stories.
iii) Identify irregularity.
iv) Number of bays in each direction pertinent to redundancy.
v) Beam members identify AISC Group and Material Specification.
vi) Column members identify AISC Group and Material Specification.
vii) Establish soil foundation conditions, e.g. soft soils, bedrock, spread footings, that may uplift piles which do not uplift.
viii) Columns that are fully or only partially fixed at the foundations.
ix) Was the building inspected following major earthquake(s) (i.e. City of Los Angeles, Santa Monica)?
x) If inspected, was the building repaired? If repaired what was the extent of the damage and how repaired. For example, some may only have involved restoring CJP welds for beam flanges to column flanges, others may have involved extensive repairs due to cracks through columns (See Figures 4.1 and 4.2).



Access windows required to facilitate repair and inspection of column and girder
Figure 4.1

xi) If not inspected, has the building experienced strong seismic motions? (e.g. Simi Valley, Whittier, Northridge, Chino Hills, etc.)

xii) Wind cyclic events, particularly for tall buildings.
xiii) Type of Welding Process, electrodes and toughness of weldment.
xiv) Remodels affecting the building's seismic resisting system.



Repair Moment Frame Connection following the Northridge Earthquake
Figure 4.2

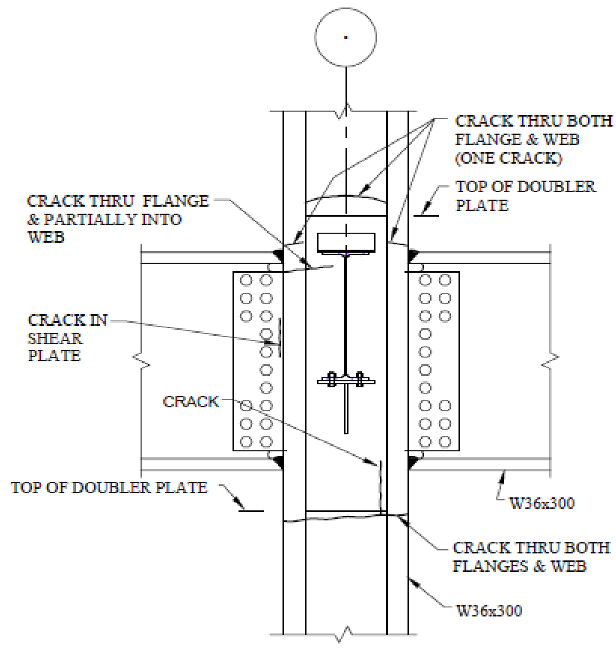
5. Preliminary Assessment

It is important to gain an understanding of the potential issues and failures that can occur. Each building will have its own set of as built conditions that need to be well established by the Structural Engineer. Lack of real knowledge about the existing structure (e.g. material properties, defects, etc.) can appreciably reduce the confidence in retrofit solutions being effective.

i) <u>Additional Information to be established</u>:
In addition to items listed in Section 4 above, items that should be established from existing drawings, include but not limited to, the following:

a) Connection details are with or without continuity plates. If continuity plates occur are they properly aligned with flanges?
b) Moment connections to major axis of column.
c) Moment connections to minor axis of column. If occur, do the beam flanges align with the continuity plates.
d) Discontinuous columns; If so, what are the details and how connected?
e) Identify types of welds used, i.e. CJP with back-up bar, fillet welds or partial penetration (PP) welds for continuity plates, doubler plates.
f) Other connection types. There are some buildings which used other connection types such as bolted flange plates.

Figure 5.1, referred to later regarding cracks found after the 1994 Northridge Earthquake, indicates a typical Pre-Northridge connection with complete penetration welds with back up plates at the beam to column flange connection and beam web bolted to a shear plate.



OBSERVED DAMAGE AT 3RD FLOOR JOINT OF A 3 BAY FRAME IN A 11 STORY BUILDING

Northridge Earthquake
Figure 5.1

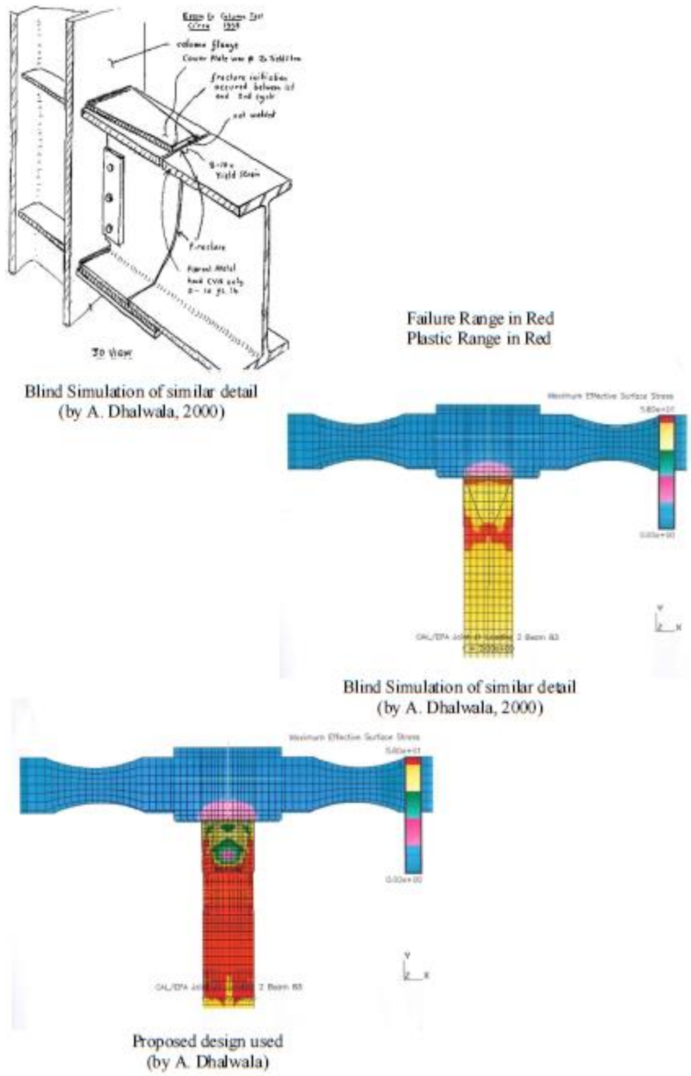
ii) <u>Types of Failures</u>:
Types of failures are well established in FEMA 351 and 352. General failure types that can occur include the following:

a) Beam flange cracking.
b) Heat affected zone cracking.
c) Lamellar tearing (ref. Farrar et al (1975), Farrar & Dolby (1972) & Lindley et al (2001).
d) Column flange through cracks.
e) Other brittle modes.
f) Ductile tearing of flanges after post yielding enhanced by low cycle fatigue and post local buckling of flanges and web. This intended failure mode of the connection was not observed after the 1994 Northridge Earthquake.

Understanding where the cracks were initiated can be very helpful. In many cases, the origins were at the back up where slag inclusions occurred. However, there was evidence of cracks occurring away from welds (see Figure 5.1). In a few cases, there were questions whether some cracks were existing possibly being caused by hydrogen embrittlement occurring at some interval of time after the original welding of the connection.

iii) <u>Connection Behavior</u>: Some of the behavioral phenomena , based upon decades of established knowledge and research and testing that has been carried out on steel framed moment connections, include the following:

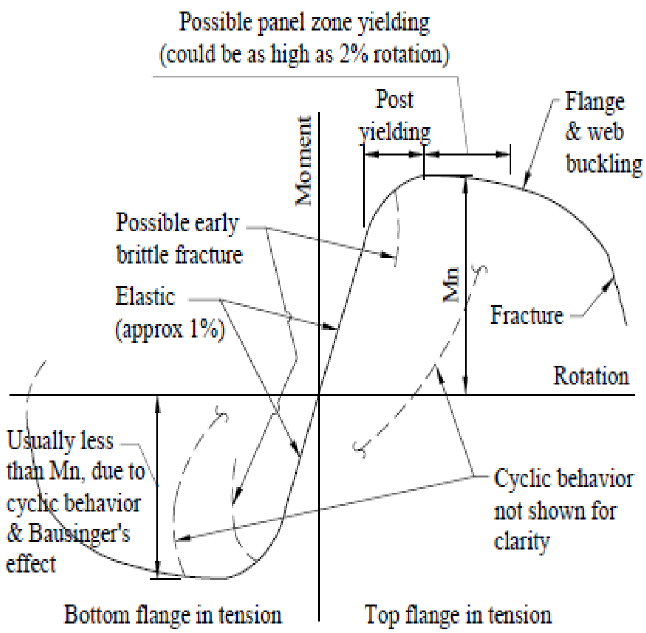
a) Tendency for stress/strain concentrations due in some part to an appreciable portion of the shear being taken by the beam flanges [(Richard et al 1995)]. This can be ascertained by nonlinear analysis of the joint.
b) Beam depth. The deeper the beam the higher the strains tend to be. This can be determined both from relatively simple analysis and again by non-linear joint analysis.
c) Span to beam depths. The smaller the span to beam depth the greater the shear which increases stress concentrations, which also can be ascertained by nonlinear analysis. Also, the shorter the span, the shorter is the length of the yield zones tending to increase strain demands to achieve rotational and drift demands.
d) Thickness of flanges. The thicker the flanges the greater the potential for brittle failure since plane strain conditions exist.
e) The thicker the flanges the greater the larger locked in stresses and strains are likely to be due to restraint to weld shrinkage. [(Ref. Masubuchi K., Miller (1993), Dong & Zhang [(1998), Tsai et al (2002)]. This can be ascertained by nonlinear joint analysis.
f) Regarding items d) and e), it is important that the triaxial conditions be considered. Figure 5.2 represents simulation by analysis of failure due to triaxiality in a beam to column connection that used cover plates along with an actual failure of a test specimen with cover plates which attained only about 1% rotation at failure (also see later). Even when representing the stresses in at the through thickness condition in two dimensions using Mohr's Circle, in most cases ductile yielding is unlikely to work as shown in Figure 7.1 referred to later [(Ref. Dowling (1999), Blodgett (1998), Maranian (2009)].
g) Flange width to flange thickness and beam depth to web thickness ratio affecting whether or not local buckling can occur. Local buckling may allow greater drift to occur. This can be ascertained by nonlinear joint analysis.
h) Local buckling in post yield conditions, if occurs, will be somewhat uncontrolled and has its limitations due to low cycle fatigue subjected to axial and bending. Again, this can be ascertained by nonlinear joint analysis.
i) Whether or not panel zone yielding and or buckling may occur since material strengths can vary. Panel zone yielding can help considerably with regard to drift. This is ascertained by simple analysis and by nonlinear joint analysis.



Cover Plate Failure due to Triaxialty
Figure 5.2

The general typical behavior, shown in Figure 5.3, first involves the elastic behavior followed by inelastic behavior which is modified by Bauschinger's effect. The hysteresis curve is also affected by whether or not the panel zone tends to yield. Generally, at the later cycles increase in local buckling of the beam flanges and web occurs which becomes uncontrolled likely causing lateral torsional buckling.

Based on our knowledge and experience, failures from the 1994 Northridge Earthquake, generally appeared to have occurred in the early stages prior to significant yielding and local buckling since no local buckling was observed. It should also be noted that, performance of existing connections can be affected by previous low cycle (seismic) and high cyclic (wind) fatigue events [(Ref. Partridge et al (2000), Nastar, (2010), Kanvinde et al (2018)].



Crued Representation of Moment Frame
Connection Behavior
Figure 5.3

6. Testing & Inspection

As a minimum, field inspection and testing should follow the recommendations and guidelines of FEMA 352. A clearly defined Quality Control (QC) and Quality Assurance (QA) should be established applicable to the building. While these will normally be based upon AWS D1.1, AWS D1.7, AWS D1.8, AISC 360 and AISC 341, some additional recommendations include the following:

i) Establishing a confidence regarding the level of defects/damage for all connections. This will require some Ultrasonic testing of complete penetration welds and/or Magnetic Particle testing of welds other than CJP welds. Also see below.

ii) Establishing material strengths both Fy and Ft. A high Fy/Ft would imply lower ductility. Also, the possible variability of Fy and Ft in both in beams and columns can significantly affect performance. For example, a low Fy in the column compared with high Fy in the beam may appreciably affect the strong column to weak/beam relationship. Furthermore, it may allow panel zone yielding to take place which may improve performance. On the other hand, a high Fy in the column compared with low Fy in the beam may prevent panel zone yielding taking place. It should be noted that, circa early 1990's, beams were typically Fy = 42ksi and columns Fy = 55ksi.

iii) Fracture toughness tests on representative welds. Note, large variation in material toughness is an inherent material property that can be expected in the same weld.

iv) Fracture toughness tests utilizing the Charpy Vee Notch (CVN), (conventionally used in this industry) on columns carried out in the through thickness direction. It should be noted that the CVN test is not a direct measurement of fracture toughness. Other industries use tests such as the Crack Tip Orientation Test (CTOD) which directly measures fracture toughness [Ref. Barsom and Rolphe (1999)]. It should also be understood that large variation in fracture toughness can occur. Furthermore, it should be understood that the need for higher fracture toughness increases with thicker materials and triaxial conditions as discussed in 5 (iii)(f) above.

v) Tests to determine if lamellar tearing is an issue. If so this would imply that there are rolled in non-metallic inclusions in the material that could lead to through thickness failure. Applicable tests include the Cantilever Lamellar Tear Test.

vi) Establishing as-built details including the following:
- Copes
- Back-up bars used
- Reinforcing fillets
- Continuity weld details (e.g. partial penetration welds, fillet welds?)
- Doubler plate details and weld details
- Column splice details
- Lateral bracing details
- Quality of Fabrication – for example what process was used for cutting, copes access holes, etc.?
- How well copes were fabricated, e.g. what process was used when cutting? Did they get grinded to reduce stress concentrations?

These details are important to check for the potential for significant stress/strain concentrations.

vii) If repairs were carried out on damaged connections, what repair details were used? For example, some may only have involved restoring CJP beam flange to column flange welds, others may have involved extensive repairs due to cracks through columns.

viii) Possible deterioration due to corrosion.

ix) Visual inspection will only likely pick up significantly damaged connections, therefore it may not be sufficient on its own.

x) Ultrasonic testing and, in some cases, magnetic particle testing should be carried out on selected connections. See recommendations above.

xi) As a minimum, the number of connections selected for inspection should be based upon FEMA 352. Locations selected to carry out this testing may be based upon preliminary analysis to identify the potentially more critical connections.

xii) Inspection of existing foundations, particularly if damage is suspected. Nondestructive excavation would then need to be carried out.

7. **Analysis/Design**

i) Structural Analysis:

Analysis procedures, adopting ASCE/SEI 41 are currently generally adopted for existing buildings. The use of nonlinear three dimensional analysis is encouraged with a degree of pragmatism and the understanding that any analysis used still has its limitations. This document does not cover the procedures but the writers wish to add the following recommended items that need to be considered:

a) Parametric Study: Establishing beam and column depth of members, b/t ratios, d/tw ratios. These are important to establish, together with the items listed in 5(v), triaxial stress conditions, potential for local buckling, significant stress/strain conditions. Also, the potential for local buckling of flanges and webs.

b) Analysis: The analysis should include for the phenomena known as "Moment Magnification", occurring in multi-story buildings, where significantly higher column moments can occur due to higher mode effects than otherwise predicted by static procedures [(Ref. Paulay and Priestley (1992) and Bondy (1995)]. This should show up in non- linear time history analysis. It should be noted that columns may yield before yielding of girders. We wish to note that, according to our knowledge, none of the numerous beam/column moment connection tests carried out involved the column yielding prior to the beam.

c) Drag Forces: Identify if significant drag forces occur. These may greatly affect the performance of moment connections.

d) Service Temperature: The temperatures to which the frames will be exposed (exterior versus interior) needs to be included. Steel joints can exhibit brittle behavior at low (close to freezing) temperatures and are affected by strain rates (see below).

e) Out-of-Plane: Consider out-of-plane movements in combination with in plane demands.

f) Geotechnical Consultant: Particularly for multi-story buildings, the services of a Geotechnical Consultant should be utilized to include, in addition to traditional recommendations, estimates on vertical acceleration, amplificant of soft stories, soil structure interaction (see item 7 (i) below), etc.

g) Strain Rates: Establish approximate range of strain rates. As stated above these can be significant if pulse effects can occur. It should be understood that high strain rates can shift the brittle to ductile transition temperature resulting in lower fracture toughness in steel materials. [(Ref. Barsom and Rolphe (1999)].

h) Drift Check: This is likely to be the most critical evaluation. Maximum drift and consequential beam to column rotations can be affected by many items including those listed above. A joint constitutive model derived from nonlinear joint analysis can help assess interstory drift(s) more reliably.

i) Soil Structure Interaction: If sufficient soil information is available and soil is suspected of having and influence, soil structure interaction should be considered.

j) Anchorage to Foundations: This should include; (i) determining fixity conditions, (ii) uplift resistance including base plate anchor bolts foundation, piles if occurs, (iii) shear transfer to foundations, (v) soil resistance.

k) Foundations: This should include assessing uplift on foundations (spread footings may uplift, piles will tend to resist uplift),soil resistance, uplift resistance of footings/pile caps(may fracture due to inadequate resistance to tension and/or punching shear), grade beam action including ductile considerations.

ii) Fracture Mechanics Analysis

It is our experience that most structural engineers have limited knowledge on Fracture Mechanics primarily because of education and codes and standards having minimal reference to this subject. Basic procedures are outlined in several publications including Barsom and Rolphe (1991), McEvily (2001) and Dowling (1999).

Briefly, fracture mechanics involves consideration of the resistance of a material to fracture due to the presence of defects quantified with the concept of fracture toughness. Two approaches have been developed:

a) Linear Elastic Fracture Mechanics (LEFM): LEFM assumes that the plastic stress field at the crack tip is sufficiently small that the principles of linear elasticity still apply. It is more readily applicable to high cycle fatigue conditions.

b) Strain-Based Approach: The strain-based approach in considering larger plastic strains addresses low cyclic fatigue.

c) Fracture Mechanics considerations applied to Steel Moment Frames: With regard to Fracture Mechanics on steel moment frame connections, checks that should be considered include the following:

- Check on existing welds with anticipated defects based upon AWS standards for allowable defects and/or defects established from testing.
- Check on through thickness subject to plane strain conditions based on possible defects. The check could use Von Mise's criterion assuming Plain Strain Condition and Poisson's ratio. However,

consideration of stress / strain concentrations needs to be included. One approach to consider is Neuber's Rule to estimate local stress and strain concentrations [Dowling (1999)].

- Consider historical cyclic effects including low cycle fatigue during past seismic events. [(Ref. Partridge et all (2000), Nastar et al (2010), Bertero & Popov (1967)].
- Consider strain rates including pulse effects discussed in 3(i) above.
- Consider potential lowest service temperature.
- Quality Control (QC) and Quality Assurance (QA)in addition to AWS D1.1, AWS D1.7, AWS D1.8 and AISC 341, should consider fitness for purpose recommendations. [(Ref. Barsom and Rolphe (1999), Williams (1998)]. Heavy members with thicker sizes should require greater QC and QA considerations including partial re testing.

Further development to address the specific fracture demands on seismic resisting systems, utilizing extensive research available including the references previously stated and Krawinkler, et al (1983), is warranted.

Regarding the above Figure 7.1 represents the Fracture Mechanics considerations at a defect causing brittle failure, Figure 7.2 indicates how triaxiality and high strain rates can cause brittle failure. Figure 7.3 indicates brittle fracture potential due to the ductile to brittle transition temperature changes. It also indicates the shift in transition temperature due to significant strain rate.

8. Considerations for Retrofit/Repair

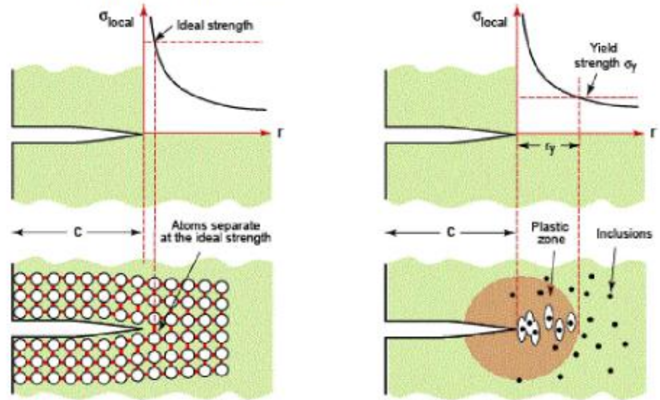
i) Repairs

If significant defects are found such that repairs need to be made to welds, particularly beam flange to column flange, great care needs to be taken with complete penetration welds such as not to cause cracking during partial removal of welds. This was experienced on some repairs carried out after the 1994 Northridge Earthquake (see Figure 8.1). Consideration should be given to the use of the weld overlay repair method which, in addition to other benefits, minimizes the potential for propagation of cracks. For further discussion, please see below.

ii) Retrofit

It is well understood by most engineers that the primary objective of a retrofit is to: 1) provide

Ref: Fracture Mechanics Texts



(a) Cleavage crack propagation (b) Ductile crack propagation

(a) The local stress rises as $1/\sqrt{r}$ towards the crack tip. If it exceeds that required to break inter-atomic bonds (the "ideal strength") they separate, giving a cleavage fracture.

(b) If the material is ductile a plastic zone forms at the crack tip. Within it voids nucleate, grow and link, advancing the crack in a ductile mode.

Brittle cracking in Steel
Figure 7.1

adequate lateral resistance; and 2) limit drift such that the existing steel moment frame connections are not damaged. An issue that the writers consider may have been overlooked is with regard to the pulse effects that can occur due to earthquakes such as can occur particularly in the Los Angeles Basin as discussed in 3(i) above. Considerations for retrofits can include the following:

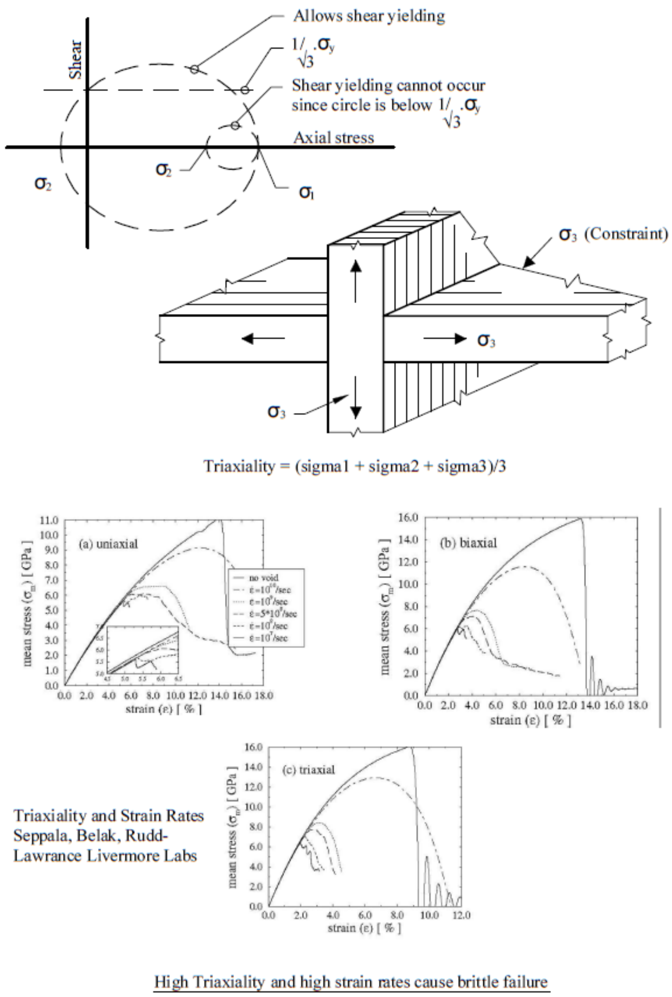
a) <u>Upgrading existing moment connections with new connections:</u> This may include pre-qualified connections specified in AISC 341 which include non-proprietary and proprietary connections.

b) <u>The addition of supplemental lateral resisting systems</u>. These include the following:

   - Concentric Braced Frames (CBF)
   - Buckling Restrained Braces (BRBs)
   - Supplement Steel Moment Frames (SMFs)
   - Damper Systems. This may be in addition to the 3 systems listed above.
   - Other Lateral Resisting Systems (e.g. Eccentric Braced Frames)

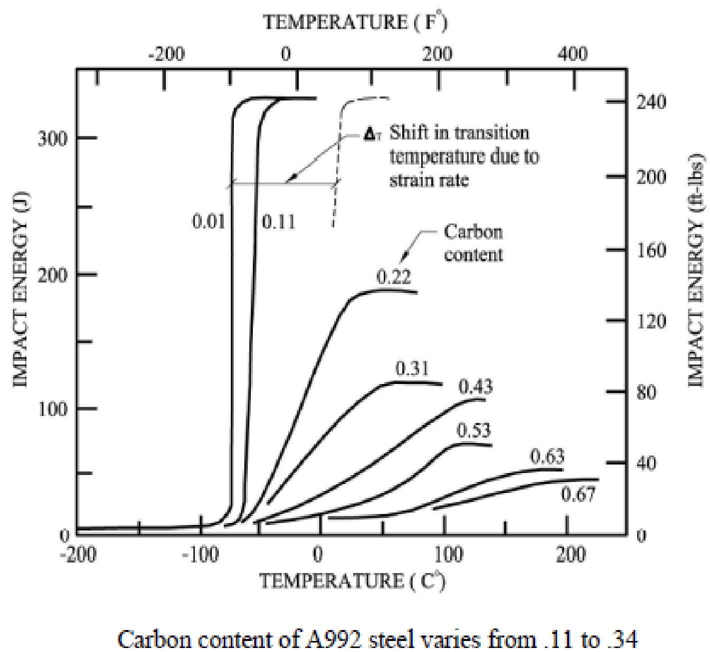
   BRBs have been demonstrated to accommodate appreciable drift. CBFs generally have limited drift capability except that significant improvements have been developed by Richard et al (2012) with the use of semi rigid connections and self-centering systems as described by Roke et al (2015) .

c) <u>Combination of a supplemental lateral resisting systems and upgrading existing moment</u>



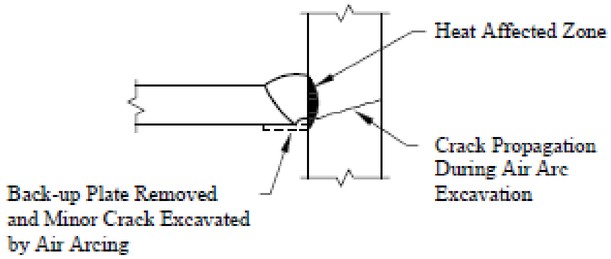
Triaxiality
Figure 7.2

connections: A combination of supplemental lateral resisting systems and upgrading existing moment frame connections is likely to be required. With regard to item 8)(ii)(a) above, when upgrading existing moment frame connections, there potentially can be several issues which include inadequate continuity plates, satisfying strong column/weak beam requirements, inadequate panel zone requiring added plates, inadequate welds, etc. Adding bracing for the columns may be required and possibly the need for additional lateral bracing to the bottom flange of beams (note, older buildings, circa 1960s, often did not have lateral bracing for the beams). Although several retrofit solutions may be feasible, invariably they may well involve significant additional components and welding. Based upon the experiences of the writers, with regard to moment frame repair projects following the 1994 Northridge Earthquake, access to facilitate the work can often be difficult. This is due to frame connections invariably being located adjacent to curtain walls such that access is only attainable on one side unless the curtain wall is removed. A further concern is that the additional welding, required to weld added components, may compromise existing material and the desired performance of the upgraded connection. This can be due to the addition of significant secondary stresses resulting from weld shrinkage, along with not well established welding procedures that can affect cooling rates increasing hardness, thus reducing ductility and fracture toughness. Also, inadvertent welding defects may occur such as undercutting, slag inclusions, poor profiles, etc., which again can reduce the performance of the connections. We understand that upgrades of existing moment frames utilizing pre-qualified connections (both non-proprietary and proprietary) and tested connections have been carried out on a number of buildings. Some connections may have involved significant modifications and welding; others, less so. With regard to adding supplemental lateral resisting systems, similar issues occur regarding welding components to the existing steel. Consideration should be given to making connections well away from existing moment frame connections.



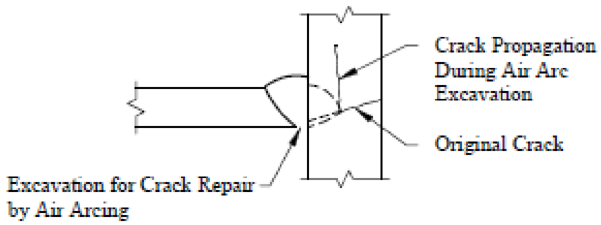
Brittle Fracture
Ductile to Brittle Transition Temperature
Figure 7.3

d) Accommodation of Drift: In our opinion there remains a concern of how much drift/rotation the existing moment frame connection can tolerate before fractures occur. Guidelines given in ASCE/SEI 41 and past testing by SAC



Section

Crack Propagation Into Column Flange During Minor Crack Repair



Section

Crack Propagation Into Column Flange During Column Flange Excavation

Figure 8.1

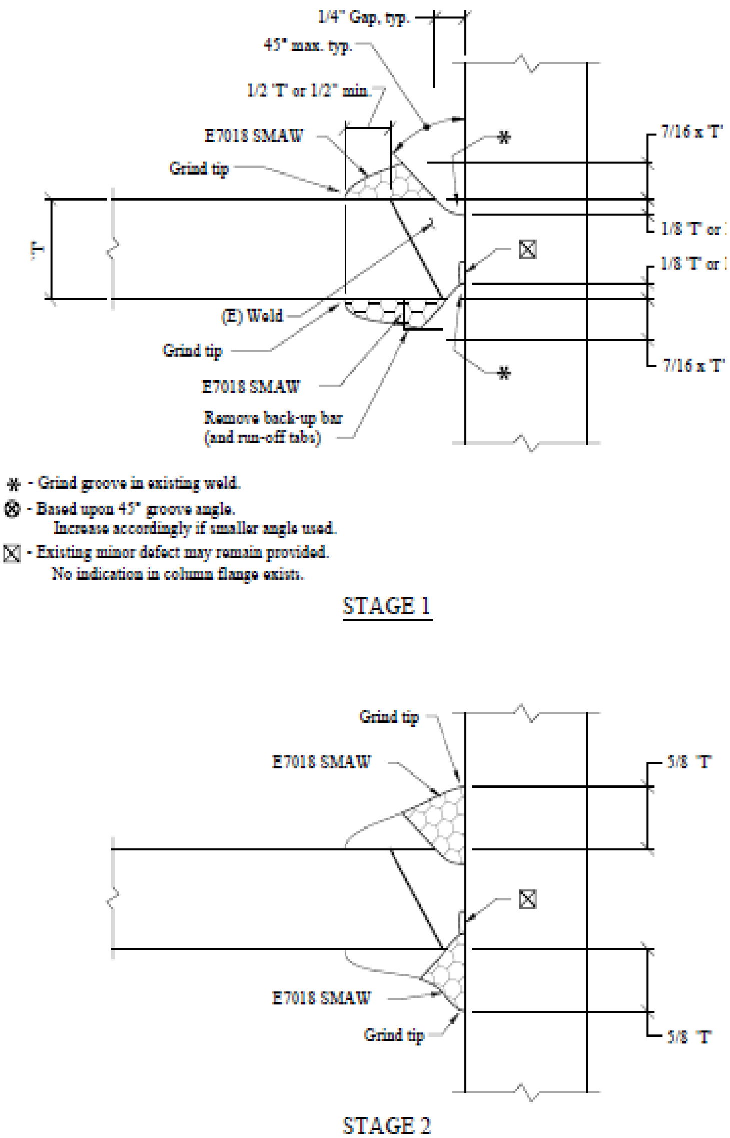
on existing steel moment connections have given some level of the order of drift/rotation magnitudes. However, we wish to mention that based upon the inspection and repair experiences of the writers, there were no connections that indicated any minor local flange buckling where fractures occurred. It appeared that fractures typically occurred below yield, at yield or just above yield. A significant concern is size effects which have been well documented by engineers and researchers [(Burdekin (1999); Torroja (1958)]. Simple first order analysis can show that strain is approximately a function of depth. Plain strain conditions, which can prevent ductility, are more prominent in thicker members. Weld shrinkage stresses are greater and material properties less desirable since thicker members are rolled less in the mills. A beam to column test, carried out post-Northridge with a W36x359 beam to a W36x670 column with cover plates, fractured at about 1% rotation per Maranian (2009) [see Figure 5.2)]. It was evident from at least one building inspected after the 1994 Northridge Earthquake that damage was much more significant with larger, heavier members than the smaller members. Thus, in our opinion, the amount of drift/rotation that can be confidently accommodated by steel moment frames and their connections is very questionable, highly dependent upon the many factors discussed and listed in the preceding discussions in this paper. Certainly, the addition of supplemental lateral resisting systems will enhance resistance and reduce drift and rotation. However, retrofit solutions preventing failures at joints and possible localized partial collapse, with all the inherent questions and unknowns, appears statistically not sufficient to attain a confidence level that is generally pursued in design.

In short, the challenges facing Structural Engineers, with good intent to provide solutions that will work with sufficient assurance, are very great. While it remains important to conduct the gathering of information discussed in Sections 4, 5 and 6, along with our recommendations given in Section 7, in our opinion, structural analysis will need some degree of pragmatism to render designs that can better provide confidence in the safety of these structures.

Consideration should be given to carrying out measures that, in addition to added supplemental lateral resisting systems, can minimize the potential for fracture and, should it occur, provide life safety measures to arrest localized partial collapse.

While we seek to remain neutral, with regard to repair and retrofit solutions, we wish to also mention additional options which may not have received that much attention. These are described below.

The weld overlay repair method for the repair of existing, complete penetration welds was developed in the 1990s by the later Dr. Warner Simon, Dr. James Anderson and Peter Maranian (see Figure 8.2). It involves accepting that there may be some defects in the existing weld and that the existing weld has low fracture toughness. Essentially, by encapsulating the existing weld with weld overlay with good fracture toughness, utilizing the shielded metal arc weld process (SMAW), the potential for fracture is substantially mitigated. Furthermore, the high stresses, applied by the beam flange, are spread over a greater area thus reducing the tri-axial stress conditions. Small component tests including cyclic tests, drop weight tests, tension and bend tests, were carried out along with full scale beam to column tests per the criteria required for the SAC program. This testing is reported in Simon et al (1991), Anderson et al (2000), Brandow & Maranian (2001) . Weld overlays were used for the repair of several steel moment frame buildings following the 1994 Northridge Earthquake. They can also be used in combination with upgrades to connections including improving access holes as developed by Ricles et al (2000) and incorporated in AISC 341.



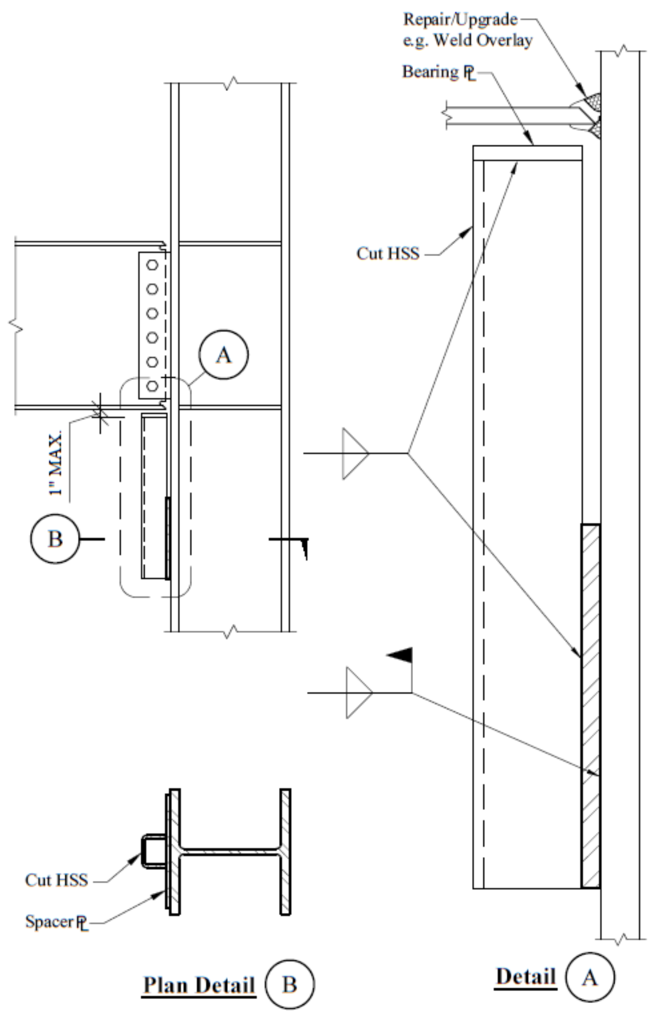
Weld Overlay
Repair Using SMAW
Figure 8.2

Regarding the possibility of localized partial collapse, due to insufficient confidence in achieving measures that address all potential issues, there may be many solutions that could provide a means of arresting localized partial collapse. One method is providing keeper details, such as shown in Figure 8.3 or proprietary bolted devices immediately below moment connections and other connections that could potentially fracture and lead to partial collapse during a seismic event.

9. Conclusions

The primary conclusions, derived from the above, indicate the following challenges:

i) There are numerous potential issues associated with unknown conditions at each existing building which, in order to establish, can require extensive investigations
ii) The seismicity requirements may not represent well actual events including pulse affects from Thrust Faults.
iii) There are a number of potential issues with regard to analysis and design due to the limitations of software, including dynamic behavior (low cycle fatigue, moment magnification, strain rates, out of plane movements, etc.)
iv) Limitation on establishing good predictions on the rotational capacity of existing connections.
v) The need to include Fracture Mechanics considerations.



Keeper Detail
(To address possible partial collapse)
Figure 8.3

In summary, even with the best intentions and degree of care, sufficient confidence, normally pursued in design, appears not readily attainable. Recommendations on retrofit/repair measures to address potential fractures and also address the possibility of partial collapse have been given with a view to providing sufficient confidence in enhancing the safety of steel moment frame buildings.

It is important to understand that, from an owner's perspective, the cost of repairs, following a seismic event, can substantially exceed the costs of proactive action to in act retrofit measures.

This paper has attempted to describe information needed and the numerous issues associated with existing steel moment frames and their connections. Many of the issues and recommendations provided may also be relevant to other lateral resisting systems.

It is hoped that, as procedures and systems are further developed on retrofit projects, and improvements can be made in the procedures and solutions to meet the difficult challenges to provide effectively safe existing steel moment frame buildings. Thus, the authors consider this paper to be work in progress.

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