HIGH-RISE PRE-NORTHRIDGE PARTIAL JOINT PENETRATION COLUMN SPLICE REPAIR

M. Chisholm¹, R. Pekelnicky² and J. Malley³

ABSTRACT

Steel construction in the 1980’s commonly utilized partial joint penetration column splice welds. The groove depth specified varied from $\frac{t_f}{2} + 1/8''$ to $t_f - 1/8''$. The unwelded portion of the flange is an initial flaw in moment resistance that when coupled with non-notch tough weld metal creates the potential for splice fracture during earthquakes. Although fracture toughness is directly measured through energy, ATC 114 Appendix A formulates from the latest pre-Northridge weld research a succinct method to calculate an effective stress to determine if fracture is the governing failure mode. A high-rise moment frame San Franciscan hotel was evaluated under ASCE 41-17’s emerging nonlinear time-history procedures and splice demands in the tower’s mid-height consistently exceeded their fracture stress. The demand is derived from P-M2-M3 forces across the section and taken as the peak combined tension stress at the extreme fiber. To minimize retrofit locations, the combined stress was calculated at every time-step because axial, strong- and weak-way bending do not occur simultaneously. The maximum combined stress was compared against the calculated fracture stress to determine each ground motion’s DCR. Both demand and capacity formulation were validated with fracture mechanic finite element modeling of representative columns.

The hotel required a unique repair strategy due to precast façade surrounding wide flange columns on three sides. Access was limited to between the flanges and 4-inch clear between the flange and façade. The retrofit gouged portions of the flange from the unwelded side into the tip of the existing weld and filled the groove with compatible weld forming a two-sided groove. Access to weld across the flange width was obtained by cutting a window through the web. Fin plates were welded to flanges in the 4” structure-cladding gap to maintain a load path while the web and partial flange were removed. Great care, sequenced calculations and strict procedure enforcement ensured stability throughout the gouge and weld processes.

¹Design Engineer, Degenkolb Engineers, San Francisco, CA 94105  
²Principal, Degenkolb Engineers, San Francisco, CA 94105  
³Senior Principal, Degenkolb Engineers, San Francisco, CA 94105

HIGH-RISE PRE-NORTH RIDGE PARTIAL JOINT PENETRATION COLUMN SPlice REPAIR

M. Chisholm¹, R. Pekelnicky² and J. Malley³

ABSTRACT

Steel construction in the 1980’s commonly utilized partial joint penetration column splice welds. The groove depth specified varied from $t_f/2 + 1/8"$ to $t_f - 1/8"$. The unwelded portion of the flange is an initial flaw in moment resistance that when coupled with non-notch tough weld metal creates the potential for splice fracture during earthquakes. Although fracture toughness is directly measured through energy, ATC 114 Appendix A formulates from the latest pre-Northridge weld research a succinct method to calculate an effective stress to determine if fracture is the governing failure mode. A high-rise moment frame San Franciscan hotel was evaluated under ASCE 41-17’s emerging non-linear time-history procedures and splice demands in the tower’s mid-height consistently exceeded their fracture stress. The demand is derived from P-M2-M3 forces across the section and taken as the peak combined tension stress at the extreme fiber. To minimize retrofit locations, the combined stress was calculated at every time-step because axial, strong- and weak-way bending do not occur simultaneously. The maximum combined stress was compared against the calculated fracture stress to determine each ground motion’s DCR. Both demand and capacity formulation were validated with fracture mechanic finite element modeling of representative columns. The hotel required a unique repair strategy due to precast façade surrounding wide flange columns on three sides. Access was limited to between the flanges and 4-inch clear between the flange and façade. The retrofit gouged portions of the flange from the unwelded side into the tip of the existing weld and filled the groove with compatible weld forming a two-sided groove. Access to weld across the flange width was obtained by cutting a window through the web. Fin plates were welded to flanges in the 4” structure-cladding gap to maintain a load path while the web and partial flange were removed. Great care, sequenced calculations and strict procedure enforcement ensured stability throughout the gouge and weld processes.

¹Design Engineer, Degenkolb Engineers, San Francisco, CA 94105  
²Principal, Degenkolb Engineers, San Francisco, CA 94105  
³Senior Principal, Degenkolb Engineers, San Francisco, CA 94105

**Introduction**

The Hotel Nikko is owned and operated by subsidiaries of the Japanese engineering and construction firm Takenaka. Forward thinking in terms of seismic risk and mitigation, Takenaka opted to retrofit their 4-Star hotel as an accompaniment to an extensive modernization of the hotel rooms and amenities. Takenaka contracted Degenkolb Engineers to perform a seismic evaluation where we identified three main seismic deficiencies: pounding across the 5th floor seismic gap, transfer girders subject to loss of gravity load carrying capacity and partial joint penetration column splices with high fracture susceptibility. All deficiencies were addressed in multi-phase voluntary seismic upgrade – the most complex and delicate being the column splice repair.

**Seismic Deficiency**

The Hotel Nikko is a 25-story steel moment frame tower constructed in 1985. Pre-Northridge moment frame beam-column connections are a well-documented seismic deficiency. Flange welds used non-notch tough weld metal prevalent in the era. An initial exists crack between the applied weld and the base metal that propagates up from the unconnected back-up bar edge. Similarly, column splices of the corresponding period potentially exhibit poor performance during seismic activity, as it was common practice to only weld a portion of the column flange, as little as sixty percent, creating a larger initial flaw than the beam-column crack.

Column splice weld fracture is far less publicized than the beam-column connection fracture because documentation of its occurrence in actual earthquakes is scarce. Deep beams, common in moment frames, are more probable to surpass the strain threshold of a weld than shallow columns. Frequent beam flange fracture likely explains the lack of column splice fracture because the beam-column connection failure limits the deliverable force to the frame column’s mid-height. In a system without this unintentional fuse it is more likely a column will see the demand that can generate column splice fracture. The Hotel Nikko’s lateral systems is composed of a redundant perimeter moment frame with up to twelve short bays of resistance each direction, each side. A tradeoff for the frame’s redundancy and short-spans is frame beams as shallow as W14s.

Response-history analysis shows that the primary non-linear behavior was yielding in the panel zone region. Without beam-column flange failure and the steep strain-hardening backbone curve of panel zones, the moment demand at the column splice plane for most ground motions was as high as theorized. The challenge is to determine the consequence of that demand.

**Formulation**

Column splice fracture analysis and repair for an existing structure at the time of the retrofit construction and to date is not a mandated procedure in the United States. Neither a requirement for addressing the hazard nor a means of calculating a capacity based on fracture mechanics is available in an adopted code. Nonetheless, fracture mechanics is a heavily studied phenomena and literature to guide calculation was available to use. Kaufman et. al., 1997 provides a formulation to calculate the effective splice stress at which the weld metal fractures based on a ratio of unwelded flange thickness to full flange thickness. ATC-114 updates this equation with
additional research. Both methods are highly dependent on the fracture toughness of the weld metal. Without a material testing program on the project, the weld metal notch toughness was bounded at the lower and median estimates. The ATC formulation for splice capacity follows:[2]

\[
\frac{K_{IC}}{F\left(\frac{a_0}{t_{f, u}}\right)\sqrt{\pi a_0}} \leq F_{u, exp}\left(1 - \frac{a_0}{t_{f, u}}\right) \leq F_{y, exp}
\]

\[
F\left(\frac{a_0}{t_{f, u}}\right) = \left(2.3 - 1.6 \frac{a_0}{t_{f, u}}\right) \times \left(4.6 \frac{a_0}{t_{f, u}}\right)
\]

Toughness is a measurement frequently associated with the Charpy V-notch test. Today’s welds measure toughness in exceedance of 200 ksi-√in. Based on the year of construction and the known weld metal used the lower-bound estimate for the column splices is 40 ksi-√in and the median is 100 ksi-√in.

With the fracture toughness estimated and bounded, the remainder of the equation requires the ratio of unwelded flange, notch, to total flange thickness to generate the effective fracture stress. The hotel’s existing drawings scheduled flange weld size based on flange thickness. Most flanges were welded roughly the same percentage as the then typical \( t_f/2 + 1/8" \). At the end of the moment frames where the original engineer assumed uplift and tension could occur they were more heavily detailed as \( t_f - 1/8" \).

The difference in capacities between the weld sizes as a portion of the flange and between the assumed toughness is staggering. Figure 1 shows the consequence of the formula based on the column section and weld detail.

![Figure 1: Critical Weld Stress vs. Notch Ratio.](image-url)
Validation

To provide a mathematical means of verification for the proposed formulation, Dr. Amit Kanvinde was engaged to build and analyze finite element models of representative column shapes found in the hotel. The most common size column members in the perimeter moment frame are W14x176 through W14x257. Figures 2 shows the FEA model and mesh used in the assessment. The finite element results more closely aligned with the latter formulation from the ATC document and thus design calculations used that method as the weld capacity template.

![Figure 2: Column Flange Finite Element Analysis.](image)

Evaluation

Fracture of the column splice weld requires a net tension in the fiber closest to the flaw. Because single bevel partial joint penetration welds are made from the outside face of the column flanges, the initial flaw is opposite the extreme fiber. This offset is accounted for in the ATC formulation.

It is at this fiber where tension is resisted by the toughness of the weld prying open the initial crack and preventing it from propagating. The column’s gravity compression force must first be...
overcome by either uplift in the column or more likely strong-way bending across the wide-flange section. Combining the maximum forces from uplift, strong-way moment and weak-way moment is far too conservative to provide refined retrofit strategy so time-step post-processing was required.

Time-step P-M2-M3 interaction forces were processed to determine the peak weld stresses in each column splice for all eleven ground motions. This was done for both the lower-bound and median estimates of the weld fracture toughness. The time step analysis identified about 70 of the more than 200 splice locations exceeded their fracture stress and were considered for retrofit.

**Design Constraints**

The hotel is clad with precast concrete covered with tile imported from Japan in the early 1980s and mimicking of the exact external appearance is improbable. In addition to the façade replacement aesthetics being in question, the waterproofing concern, tower crane and scaffolding coordination and extended schedule were an unacceptable liability and cost risk. Therefore, the owner selected to preserve the exterior shell and conduct the retrofit entirely from the inside of the structure.

More than half of the columns selected for retrofit are encapsulated on 3-sides with precast panels in which the external tiles are embedded. This enclosed the column flanges to a near inaccessible 4-inch gap between the flange outside face and the inside of the precast wall. This gap is too narrow to provide a quality repair weld and inspection. This condition is shown in Figure 3.

There were six additional column splice conditions less frequent than the condition shown but with their own challenging precast façade conflicts.
Retrofit Detailing

No room is available between the inside face of the façade and the column flange and the precast shell will not be removed. Therefore, the only location to create access is from between the column flanges by removing the column web.

Once a window of access is cut through the web of the column the inside face of the flange available to work on is the opposite side from which the original weld was applied. However, it is the side closest to the unwelded flange portion. The repair strategy is to gouge and grind out the unwelded portion of the flange and into the competent portion of the existing weld creating a single bevel groove from the inside of the column. Then this groove would be filled with modern and compatible weld material. The result resembles a two-sided groove weld – one side the original T-4 material, the other side gouges into the root of the first and welds with current material. The enlarged detail of the procedure is shown if Figure 4 Left.
To complete this repair a significant portion of the column cross-section is removed. The web access window at the plane of the splice completely removes the web. The gouge and grinded portion of the flange no longer maintains compression contact between the upper and lower column. Finally, the heating from the gouging and rewelding was considered for a strength reduction of the remaining portion of the flange.

The hotel was either operational or partially filled with various construction teams during the retrofit timeline and all columns still needed to maintain gravity load carrying capacity and stability. The lowest level of column splice repair was on the 5th floor of the twenty-five story tower. The remaining partial flange and full flange was not well suited to support twenty stories of dead and reduced live load from above without shoring.
Permanent shoring splints were designed to supplement the removed portions of the cross-section. The splints were termed fin plates and added in the 4-inch gap between column flanges and façade as shown in Figure 4 Right.

After both flanges repair welds are complete the full compression capacity of the columns to resist seismic axially and shear loading must be restored. A weld replacement plate was installed outboard from the web centerline completing the structural procedure summarized below.

Splice repair procedure:
1. Remove fireproofing where accessible and grind residue from surfaces to which weld will be applied
2. Install flange stiffener plates in the column pocket above and below the splice
3. Install fin plates on each column flange aligned with the web
4. Cut a preliminary web access whole smaller than dimensioned on elevation detail
5. Remove fireproofing from backside of web
6. Cut full web access hole
7. Gouge and grind first flange to specified groove profile
8. Weld created groove profile
9. Inspect first flange groove weld
10. Gouge and grind second flange to specified groove profile
11. Weld created groove profile
12. Inspect second flange groove weld
13. Install the web replacement plate to the column flanges and flange stiffener plates

Mock-Up Construction

Due to the complex and precise nature of the gouge, grind and reweld procedure coupled with the confined access between column flanges a series of mock-up splice retrofit were carried out at the request of the owner in the steel sub-contractors shop for each unique splice type. The mock-ups were spliced together with the old T-4 weld wire used on the structure and many others in the mid-1980s. The presence of this old metal is significant in the gouge and grind process. The old weld wire is thicker and required to burn hotter than modern wire. Consequently, the weld left more slag and cavities in the groove than would be expected using current materials. The welder performing the mock-up encountered these imperfections and adjusted accordingly to produce the most well-prepared surface for welding.

Surrounding the mock-up column splice was story-high sheet metal panels used to represent the physical and optical constraints imposed by the precast façade (Figure 5 Left). Blocking direct access forces the welder to reach away from his body to perform the various actions described, as well as do so at a tight angle between flanges (Figure 5 Right).
Weld Inspection

During the mock-up phase the weld inspectors who would perform testing on the hotel were contracted to do the same on the test specimens and gave us the opportunity to work in real-time with the inspector to develop a project specific testing protocol. Although the resultant two-sided bevel weld forms a continuous weld across the flange, it is not a complete joint penetration weld and cannot pass the same criteria. The old weld metal may have additional internal flaws and register as defects in ultrasonic testing.

The procedure required the inspector to test the weld at depths across the flange thickness and record the depth from the inside of the flange where the new weld was applied the first flaw if one was present. If the first flaw occurred outside of the depth of the newly applied weld, the new weld was of sound material and the flaw was present in the existing material. This was considered an acceptable pass. Although flaws in the existing weld are still prone to fracture, these flaws are far smaller than the initial notch of the unwelded flange and the formula for effective fracture stress exceeds the yield stress of the base material.
Construction

Sequencing for the construction was developed to balance the fully connected and in-progress columns around the perimeter and up the height of the hotel. The strategy was to develop an additional level of safety in the event of a construction error. If a column’s load carrying capacity were to be compromised from over-gouging or other destructive means the adjacent columns would remain fully intact and capable of taking additional load shed from the failed column. Each perimeter moment frame was analyzed as a Vierendeel truss to test its ability to redistribute load from up to twenty stories above.

Laser measurers were installed at splice locations throughout the building to measure deflection in the column during the gouge and grind procedure. Initial markers were selected on the column above the splice and the instrument was mounted on the floor below to detect any convergence. This measurement provided real-time information of the impact of the procedure on the column and acted as a first warning alarm to stop construction before excessive deflections occur. Through the retrofit procedure only negligible deflections were recorded.

Significance

There are more than 25 years’ worth of steel buildings in seismic prone regions erected under the assumption that partial joint penetration welds are sufficient to resist the loads imposed near the mid-height of a column. Time-history analysis shows the demand at this location is much higher than anticipated in static procedures. Although uncodified, column splice fracture can represent a more consequential failure mechanism than beam-column connection fracture because a single column splice failure, even in a redundant frame, can mean the collapse of multiple bays classifying its behavior as a critical force-controlled action. Fracture propagation from the welded flange through the web base metal and shear tab may result in unseating and loss of gravity load carrying capacity.

An added danger of moment frame retrofits focusing on the beam column connection is a robust joint between the members means the first failure may occur elsewhere, driving the fracture to occur at the splice. Prevalence of damage in previous earthquake has focused on the beam-column joint as discussed. But previously retrofitted structures, buildings with less prone beam fracture detailing and shallow member moment frame beam structures like the Hotel Nikko could be subject to this deficiency if not addressed.

References