

# FINITE ELEMENT ANALYSIS OF THE CONVENTIONAL AND UPGRADED STEEL MOMENT CONNECTIONS

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## ABSTRACT

Many steel moment frames were damaged during the Northridge Earthquake and the Kobe Earthquake in Japan. The common damages came from fractures in the beam-to-column moment connection region. This incident clearly suggested that the current design procedures of the moment connection were not adequate. In this study, first, the nonlinear finite element analysis of a typical beam-to-column moment connection was performed to investigate the behavior of the weak panel zone and identify some potential problems of such a connection. Second, the upgraded model, which is a combination of the uses of continuity plates, the unrestrained beam flange length, and the supplemental plate, was modeled and analyzed. The stress triaxiality ratio at the beam flange interface area of both conventional and upgraded finite element models was compared.

The results from the first part show that the state of stresses around the connection area is, in fact, very complex. The stress contours show the high stress concentration between the interface and the access hole area. The current practice design is inadequate to meet the required strength of the connection. The study also shows that the weak panel zone can be used to absorb energy from the connection area in order to reduce the high ductility demand of the beam flange. Finally, the upgraded model also demonstrates the improved behavior of the connection.

Keywords: Finite element (FE) analysis, steel moment connection, weak panel zone.

## INTRODUCTION

Moment-resisting structural steel frames have long been recognized as one of the best structure systems to resist seismic forces. The performance of such frames under seismic forces depends primarily on the strength and the ductility of their beam-to-column connections (Blodgett,

1993). These buildings are intentionally designed such that, under earthquakes, the energy can be dissipated by the mean of yielding of the material around the beam-to-column connection regions (Englehardt et al., 1993; Goel et al., 1996). In other words, the safety of a structure depends mainly on

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the capability of the structure to absorb energy in the inelastic range rather than in the elastic stiffness of the structure (Popov et al., 1989).

Unfortunately, a large number of premature failures of welded moment connections in moment-resisting steel frames were reported after the 1994 Northridge Earthquake (SAC Joint Venture, 1995-1998) and the 1995 Kobe Earthquake in Japan. Even though there was no total collapse of steel-framed buildings due to the moment connection failures during these two earthquakes, these premature brittle failure types have raised questions on the reliability of the current design codes and construction practices for steel moment-resisting frame.

Since then, there are many projects initiated around the world. In the U.S.A., leading by the Structural Engineers Association of California (SEAOC), Applied Technology Council (ATC), and California University for Research in Earthquake Engineering (CUREE) combined into a major research group (SAC Joint Venture) to investigate steel structure damages and to develop guidelines for repair, rehabilitation, and new design practice for steel moment frames.

Up to date, some questions have been answered, but there are still many of them left unanswered (SAC Joint Venture, 1995-1998). Therefore, the purpose of this project was to study the behavior of the welded steel beam-to-column moment connections and to suggest a possible upgrade scheme to the connections.

## OBJECTIVE

The objective of this study was to develop more understanding of the inelastic behavior of the welded steel moment connection with its weak panel zone condition, such as stress distribution around the connection area. The study also included the potential of using a weak panel zone design in order to absorb some levels of energy from connections.

## MATERIALS AND METHODS

### Nonlinear finite element (FE) analysis of steel moment connections

For this study, the inelastic behaviors of the steel moment connection were investigated using a commercial FE software, and the possible upgrading scheme was purposed in order to succeed the use of the weak panel zone concept without jeopardizing the connection region.

Since a use of solid elements is expensive and inconvenient to create the FE model. Moreover, considering from the loading and boundary conditions, it can be appropriately taken advantages of the symmetrical property. Therefore, only a quarter of the typical connection was modeled corresponding to the symmetry arguments (Wongkaew, 2003; Wongkaew, 2005). As shown in Figure 1, the plane of symmetry was applied on the center-plane in order to represent the symmetrical condition between the left and the right hand sides of the model. On the other hand, anti-symmetric plane was also used to represent the top and the bottom symmetry with respect to the mid-plane. Both symmetric and anti-symmetric boundary conditions were imposed together along the intersection regions (shedding areas). Nonlinear material finite element analysis (small displacement-large strain) was imposed to investigate the nonlinear behavior of the model. Materials were assumed to have  $F_y = 50$  ksi and strain hardening modulus was assumed to be 5% of the elastic modulus up to 1% strain (AISC, 1994). The material was assumed to be perfectly plastic for larger strain. Figure 2 shows the stress-strain curve used in the nonlinear analysis. The elastic modulus was taken as 29,000 ksi. A maximum vertical displacement of 6 in (4% story drift) was applied to the tip of the beam using a displacement control scheme. It should be noted that geometrical nonlinearity was not considered in this study.

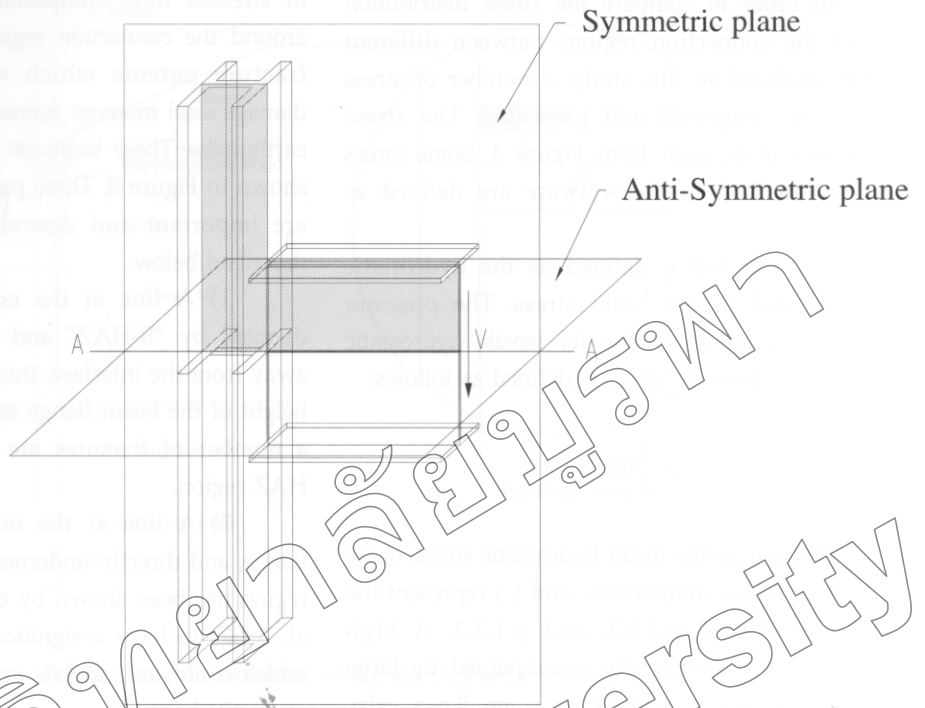


Figure 1. Boundary conditions for the finite element model using solid elements.

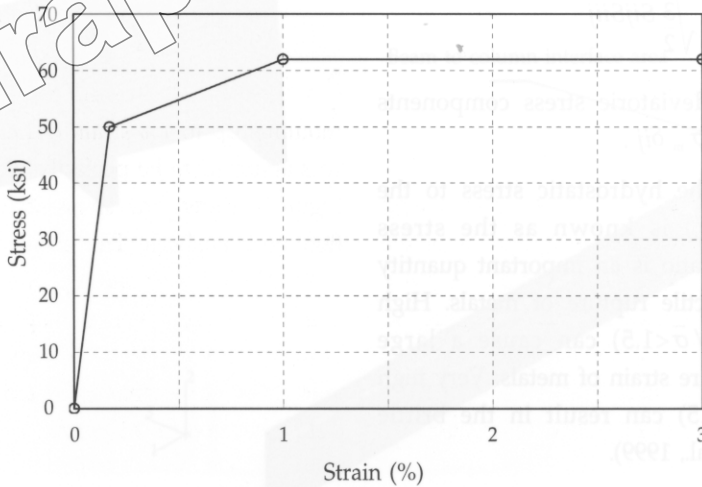


Figure 2. The stress-strain curve for the nonlinear analysis.

### Stress distributions and stress indices

In order to compare the stress distribution around the connection regions between different models analyzed in this study, a number of stress types were computed and presented. The stress directions can be seen from Figure 4. Some stress indices described in the software are defined as follows:

**Pressure Index:** defined as the hydrostatic stress divided by the yield stress. The pressure index is a negative number for tensile hydrostatic stress. The hydrostatic stress is defined as follows:

$$\sigma_m = \frac{1}{3} \text{trace}(\sigma_{ij})$$

Where  $\sigma_m$  is the mean hydrostatic stress,  $\sigma_{ij}$  are the Cauchy stress components, and  $i, j$  represent the global directions,  $i=1,2,3$ , and  $j=1,2,3$ . A high hydrostatic stress is usually accompanied by large principal stresses. If a crack or some flaws exist, these high principal stresses can result in large stress intensity factors at the crack tips, which increase the potential for the brittle fracture.

**Mises Index:** defined as the Von Mises stress divided by the yield stress. The Mises stress,  $\bar{\sigma}$ , is defined as follows:

$$\bar{\sigma} = \sqrt{\frac{3}{2} S_{ij} S_{ij}}$$

Where  $S_{ij}$  are the deviatoric stress components such that  $S_{ij} = \sigma_{ij} - \sigma_m \delta_{ij}$ .

The ratio of the hydrostatic stress to the Mises stress ( $\sigma_m/\bar{\sigma}$ ) is known as the stress triaxiality ratio. This ratio is an important quantity when considering ductile rupture of metals. High triaxiality ( $0.75 < \sigma_m/\bar{\sigma} < 1.5$ ) can cause a large reduction in the rupture strain of metals. Very high triaxiality ( $\sigma_m/\bar{\sigma} > 1.5$ ) can result in the brittle behavior (El-Tawil et al., 1999).

The indices described above and other types of stresses were computed at different locations around the connection regions associated with the fracture patents which were observed in the damage steel moment frames during the Northridge earthquake. These locations and line considered are shown in Figure 3. Three particular locations, which are important and desirable for this study, are described below:

1) A line at the edge of the weld zone denoted by "B-HAZ" and taken nominally  $1/2$  in away from the interface; this line is located at a mid height of the beam flange and is of interest because a number of fractures are observed in the beam HAZ region.

2) A line at the mid-height of the beam flange, and directly underneath the access hole. This region has been shown by other researchers (Lee et al., 1997) to have a significant strain concentration, which could lead to low cycle fatigue in the event of ductile behavior.

3) Finally, a line between those two points is also added because many fractures observed during the test at the University of Michigan are occurred pass through this region (Goel et al., 1996; Lee et al., 1997; Wongkaew et al., 2002).

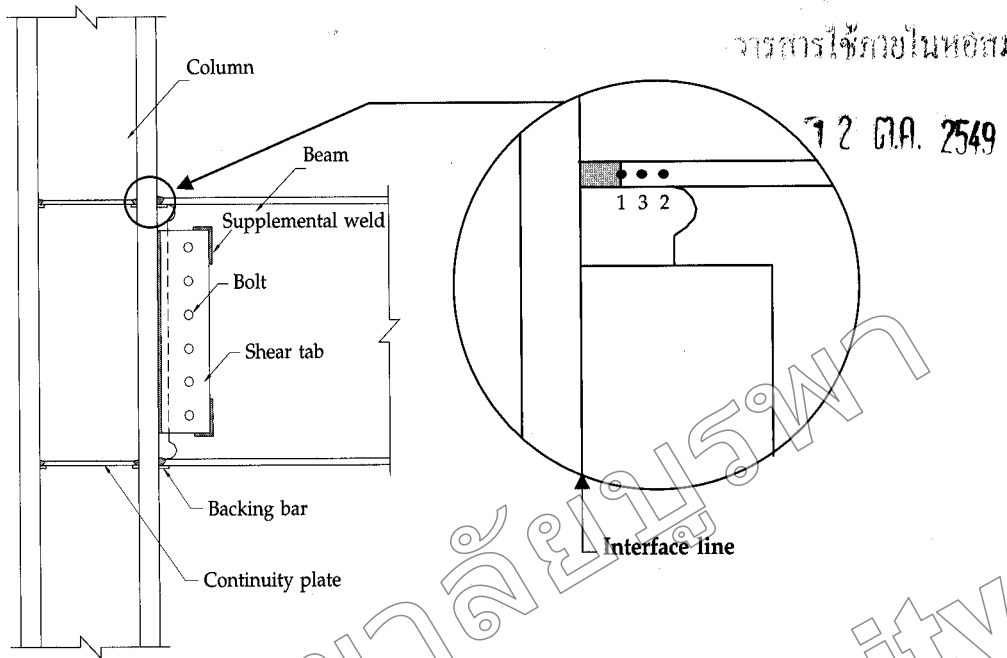


Figure 3. Some locations where stresses are extracted.

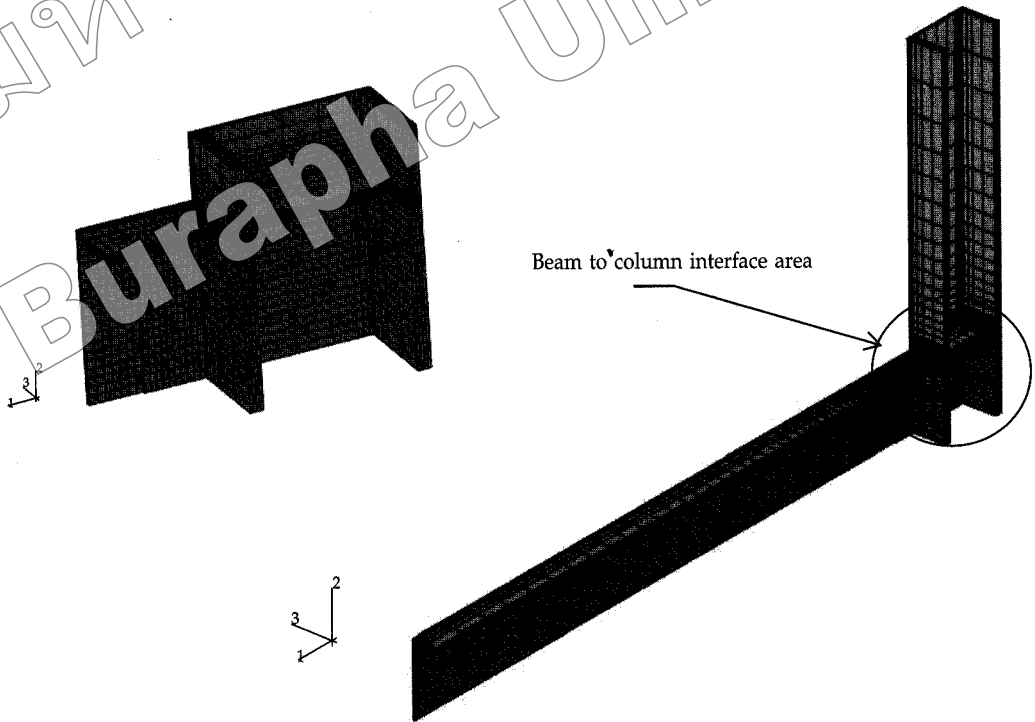


Figure 4. A quarter finite element model of the connection area using solid elements.

## RESULTS AND DISCUSSIONS

### The FE analysis of the conventional steel moment connection

A FE model of the connection is shown in Figure 4. The von Mises stress contour and the shear stress, S12, contour images around the beam flange region are presented in Figures 5 and 6, respectively. All stress contours showed the high stress concentration between the interface and the access hole area. As can be seen, the von Mises stress was highest at the middle of the beam flange. One important observation at the interface area is

that both stress values at the top surface of the beam flange were much higher than at the bottom surface. However, the reverse manner is noticed at the access hole cross-section. This implies that the negative local bending may be employed in the area between these two areas. This local bending of the beam flange can create a very large strain concentration, which may possibly lead to the fracture of the connection. Moreover, the shear stress contour also showed the sliding behavior passing from the access hole to the top of interface area through the thickness of the beam flange.

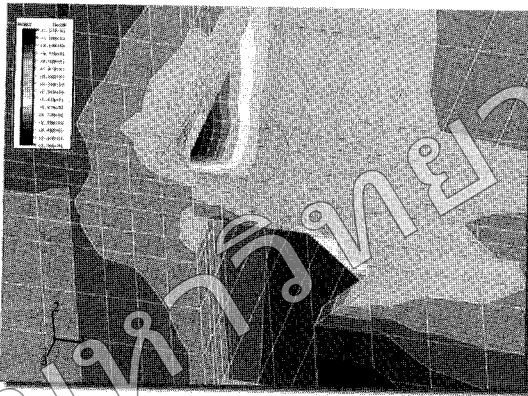


Figure 5. von Mises stress contour.

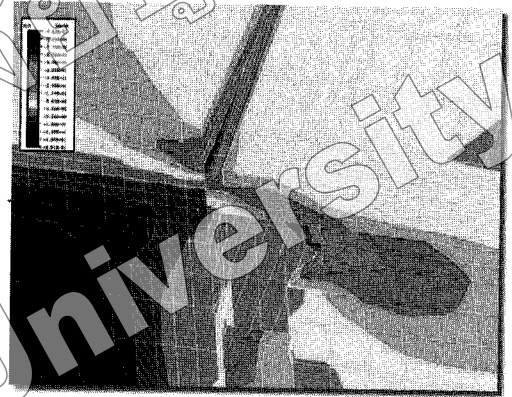


Figure 6. Shear stress contour.

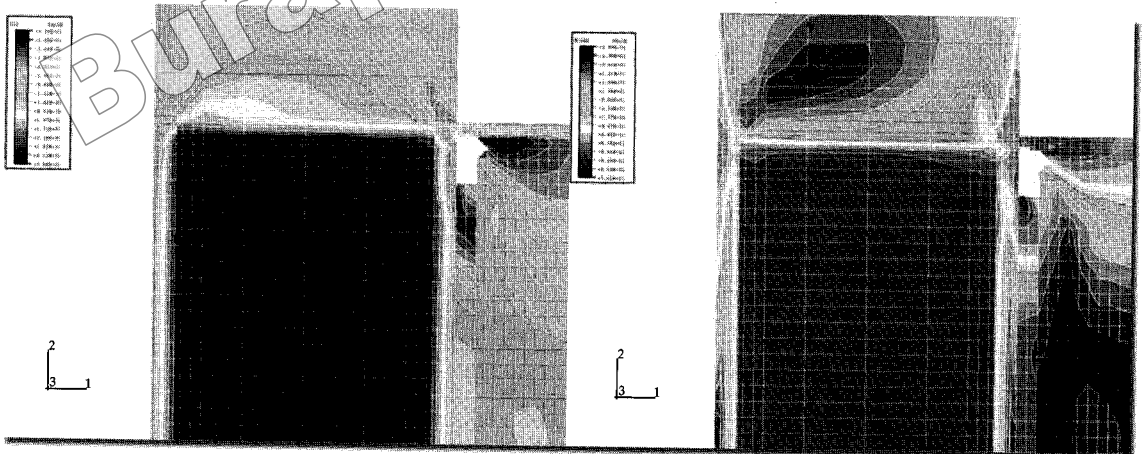


Figure 7. The shear stress and von Mises stress contours for the conventional model.

Figure 7 shows the contour lines of the shear and von Mises stresses in the plane of the beam and column webs at the last increment of analysis (6.0 in displacement). The maximum shear and von Mises stresses initiated from the center of panel zone and propagated toward the corner of the panel zone and the column flanges. The maximum shear stress in the panel zone was 36 ksi for almost the whole area of the column web. However, the shear stress rapidly decreased almost to zero when approaching the beam flange level (directly beneath continuity plate). In other words, the shear stress was contained inside of the column web. The sign of the shear stresses in the column web also changed at the middle of the beam flange level. This shear stress distribution was considerably similar to the one that was computed from the static equilibrium equations. As can be seen, the maximum von Mises stress was approximately 65 ksi, where the yield stress of this FE model was defined as 50 ksi. Therefore, the whole panel zone completely yielded, which was exactly observed in the experiments (Krawinkler et

al., 1975; Popov et al., 1987; Tsai et al., 1995). The results suggest that the panel zone element can be possibly designed to absorb energy from the earthquake in order to reduce some levels of energy from the beam flange at the connection area.

### FE analysis of the upgraded steel connection

From the previous part, even though it has been shown that the weak panel zone can absorb energy by undergo large strain, the high stress concentration at the interface area still create the problem. Therefore, the upgraded scheme is introduced to the FE model. The FE model was modified by increasing the unrestrained beam flange length. In addition, a steel plate, which has a thickness of 0.5 in, a height of 4 in, and a width of 12.57 in, was added to the model at the beam flange level. The picture of the upgraded model can be seen in Figure 8. Nonlinear material property, loading scheme, and boundary conditions were imposed the same as the nonlinear analysis model of the previous FE model.

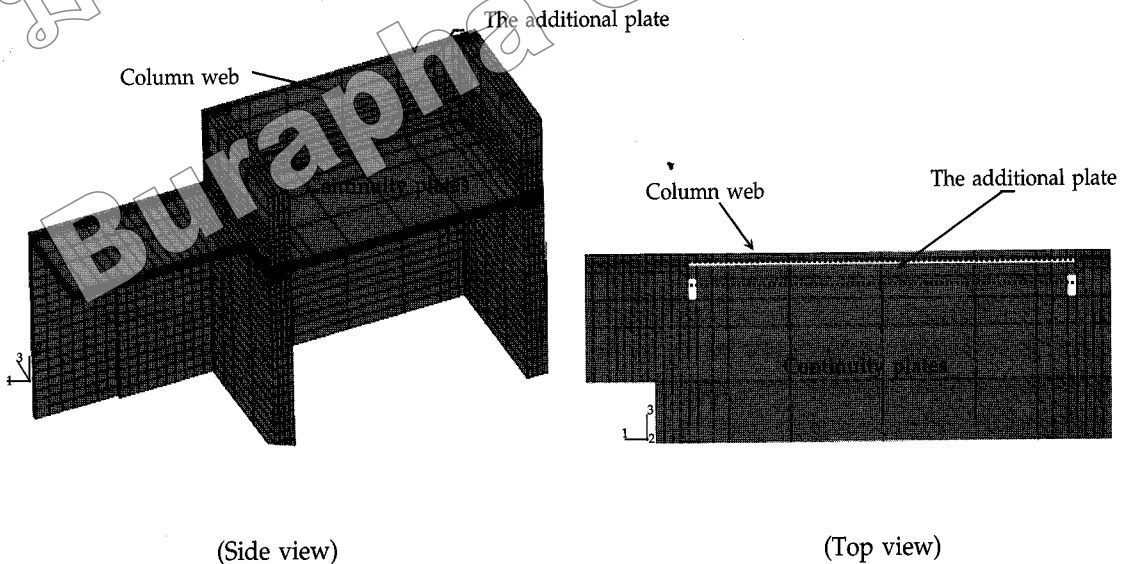


Figure 8. Side and top view of the upgraded model.

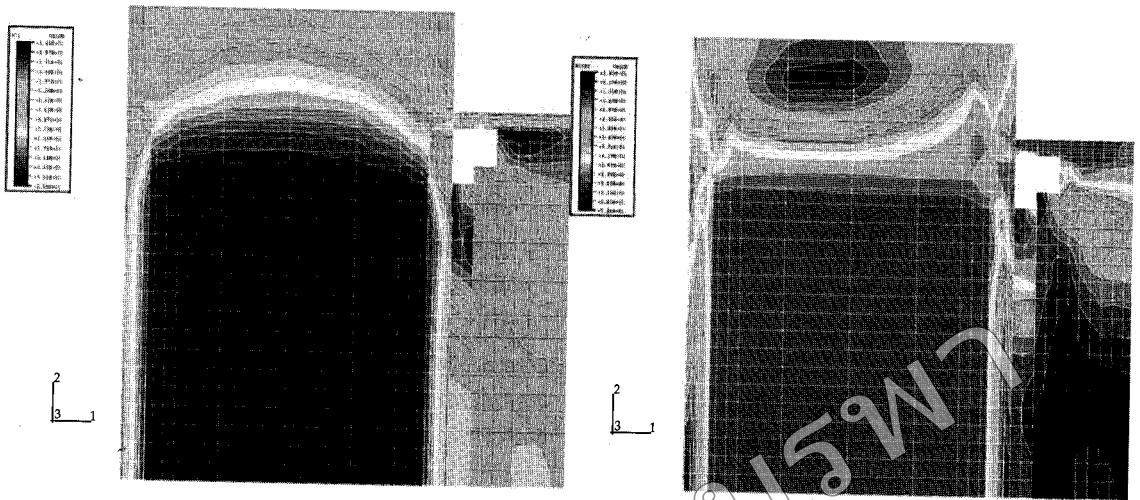


Figure 9. The shear stress and von Mises stress contours of the upgraded model.

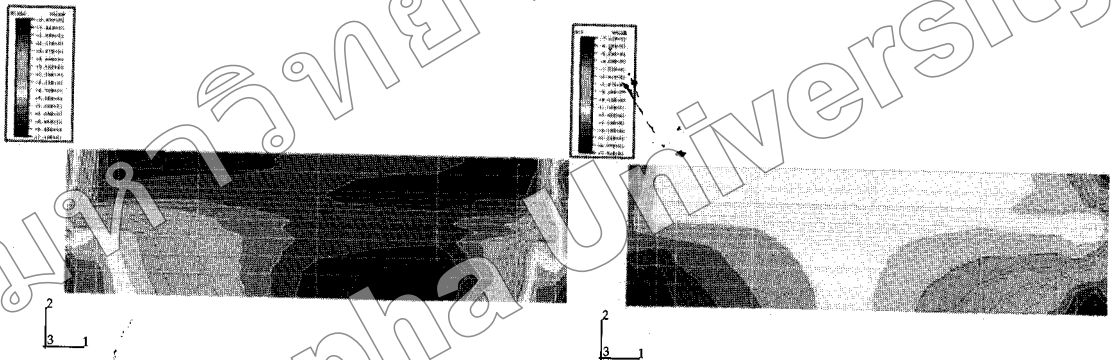


Figure 10. The von Mises stress and S11 stress contours of the additional plate.

The contour plots of shear and von Mises stress of the last increment (6.0 in displacement) in the plane of beam and column webs are shown in Figure 9. Similarly, the maximum of shear and von Mises originated at the center of the panel zone, and propagated outward. However, this model, the maximum contour line was contained at the level of the additional plate. The maximum stresses did not push up to the beam flange level as observed in the conventional model. It implies that this additional plate was effective enough to control the inelastic activity at the center of the panel zone up to the same

level as the shear tab edge. It was because the additional plate was added to the model from this level up. The maximum shear stress in panel zone was 36 ksi, and maximum von Mises stress was 62 ksi, which were almost in the same level of strength as the previous model. It means that no strength drop in the panel zone was observed in this case. Figure 10 shows the von Mises and S11 stress distributions in the additional plate itself. It can be seen that the average of these two stresses was as approximate as 30 ksi. It suggests that this plate is active and substantially receiving forces from the beam flange.



### Stress triaxiality ratio at the beam flange interface area

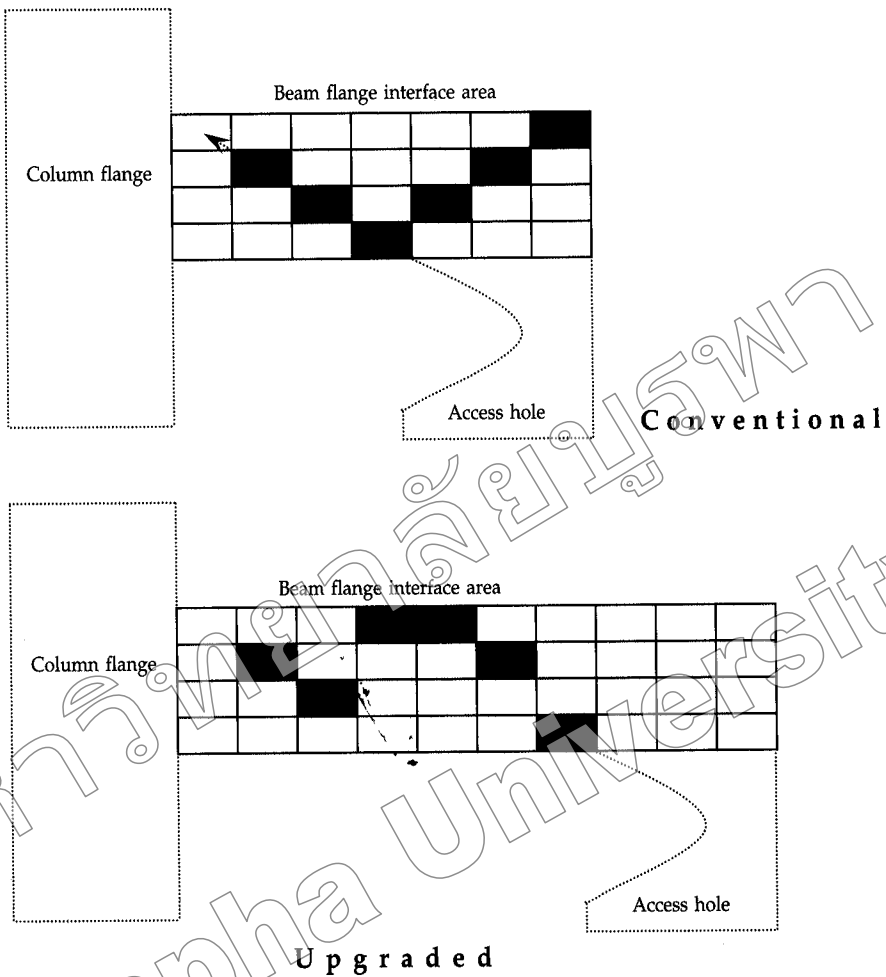


Figure 11. Locations of maximum stress triaxiality ratio at the 6 in beam deflection.

Figure 11 shows a drawing of the locations of maximum stress triaxiality ratio (red areas). The results were extracted from the FE analysis of conventional and the upgraded model. As defined from the beginning that higher stress triaxiality ratio can cause higher chance in the fracture. It can be seen clearly from Figure 11 that the fracture plane of the conventional model was similar as seen from the test photographs (Wongkaew, 2002) and Figure 6. The maximum stress ratio was initiated around the access hole element and propagated through the thickness of the beam

flange roughly about 45-degree angle in both forward and backward directions. This was confirmed by the test results that a number of fracture planes also observed in backward direction. However, with the upgraded model, even though the stress ratio was maximized at the access hole region, the pattern was not as obvious as before. Moreover, the stress ratios from the upgraded model showed more uniform values of about 0.40-0.50 over the beam flange interface area. The upgraded model also resulted in less stress triaxiality ratio which can be computed as much as

40% smaller. Thus, based on this analysis, it can be concluded that the upgraded model may be able to perform in the better behavior than the original model.

## CONCLUSIONS

The finite element results from this study show that the stress distributions of the steel moment connection are very complex. Next, the panel zone can absorb energy and show an incredible stability before collapse. Therefore, it may be possible to use the strong column (with the weak panel zone) and weak girder concept in designs of the seismic connection. Finally, the upgraded model also demonstrates the improved behavior of the connection, and could be used successfully with further study.

## FUTURE STUDY

This study is mainly conducted on finite element analysis. The experimental study is still needed to confirm the results from FE analysis before the design recommendations can be suggested.

## ACKNOWLEDGEMENT

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