# SAC Steel Project Building Models Development in OpenSees: Fracture Connection Model Validation

## Scope

The present document describes the model validation of the connection fracture model used to assess the influence of brittle connection fracture on the fragility of the three and 9-story SAC Steel Project buildings designed for Los Angeles according to pre-Northridge codes. This document provides a brief description of the validation of the connection model, which was developed by comparing the behaviour of the proposed model with experimental results obtained in the SAC Steel Project testing campaign. The models were developed in the Open System for Earthquake Engineering Simulation (OpenSees) platform.

## 1 Summary of the finite element modeling approach

The developed model is based on: (i) the Finite-Length Plastic Hinge (FLPH) formulation (Scott and Fenves 2006; Ribeiro et al. 2015) for simulating member response including nonlinear behavior, as well as strength and stiffness deterioration; and (ii) zero-length springs to model the connections. The zero-length springs are placed at both ends of the FLPH element and are assigned a rigid-plastic behavior, where the plastic strength is abruptly reduced to simulate connection fracture. This model allows for independently considering empirical models in simulating member response at the same time that considers the possibility of connection fracture.

The Modified Ibarra-Medina-Krawinkler (ModIMK, Lignos 2008), is used for its versatility. The ModIMK model is an energy-based deterioration model, which accounts for several deterioration mechanisms that are able to accurately reproduce the behavior of steel members and connections (Lignos and Krawinkler 2011), RC (Haselton and Deierlein 2007), and timber structures (Ibarra and Krawinkler 2005). Ribeiro et al. (2015) have developed a consistent implementation of this model in order to be used in both FLPH and in concentrated plasticity (zero-length springs) models. These models were implemented by the authors in the OpenSees framework and are used herein without any specific modification. Details of these models and the corresponding implementation are given in Ribeiro et al. (2015) and are omitted here for brevity. The Bilin and the Pinching models, defined in Ribeiro et al. (2015), are used herein to model members and connections, respectively.

The Bilin model is used to model plastic hinges of the member. The parameters of the model are obtained from empirical laws proposed by Lignos and Krawinkler (2011). The Pinching model is used to model the connections. This model includes strength and stiffness deterioration mechanisms. Connections may fracture at different rotation amplitudes in positive and negative bending. Different positive fracture rotations, $\theta^+$, and negative fracture rotations, $\theta^-$, are typically defined in positive and negative bending corresponding to bottom and top beam-flange fracture, respectively, which is mainly due to the influence of a slab, a backup bar, or an access hole. The moments at these rotation values depend on the model used to simulate member behavior. After fracture the behavior of the connection model includes a softening branch over a rotation of 0.002 rad. The softening branch models the flange or weld tears completely through, and thus the moment capacity decreases. Even after fracture in the top or bottom beam-flange the resistant capacity in the opposite flange of the beam remains intact. Thus the behavior of the connection has a pinched reloading branch, which is defined by three parameters ($F_{r,p}$, $F_{r,n}$, and $A_{pinch}$) that define the point at which reloading starts, and a residual resistant capacity defined as a fraction of the fracture moment. Figure 1 shows an illustration of the developed model.

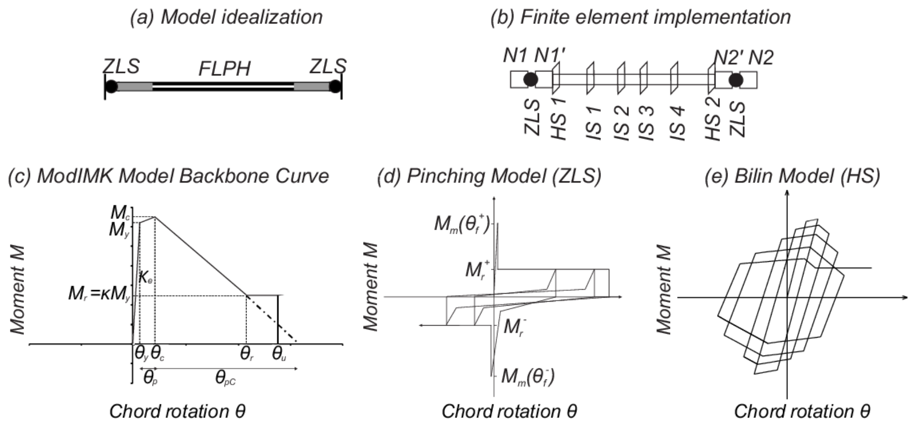


Figure 1 - Conceptual description of the proposed modeling approach

The modeling approach described above is exemplified through an example of a simply supported beam with anti-symmetric end moments. A finite-length plastic hinge element and a rigid-plastic zero-length spring are employed to simulate member and connection behavior, respectively. A cyclic pushover analysis is performed by controlling the rotation at the right-end node. Seven complete cycles of increasing rotation are executed and three responses are tracked: (a) global response (Figure 2(a)); (b) member response (Figure 2(b)); and (c) connection response (Figure 2(c)). In this example, results in the figures indicate that during the first two cycles the connection has not fractured, even though the beam element goes into a nonlinear range. During the third cycle, the connection fractures, which is associated with an abrupt decrease in strength in the positive moment region. Subsequently, after load reversal (fourth cycle) the fracture closes, which occurs during the segment with lower slope (below point 3). After fracture closure the initial strength remains intact for negative bending. As a consequence, the connection remains rigid. However, after significant member yielding, connection failed and a decrease in strength for negative moment is also recorded. During the following cycles (fifth, sixth, and seventh) the moment varies between the residual moment of the connection. When this value is reached connection rotation increases, while member rotation remains the same. The segments with lowest slopes correspond to the closing of the fractures. The described model behavior presents improvements to the behavior of the model proposed by Luco and Cornell (2000) (see Figure 2(a)), namely in what concerns differentiation between fracture closing and reloading, which is not possible in the former model. Moreover, this model fits well data driven from experimental tests of welded connections, as shown in the next section.

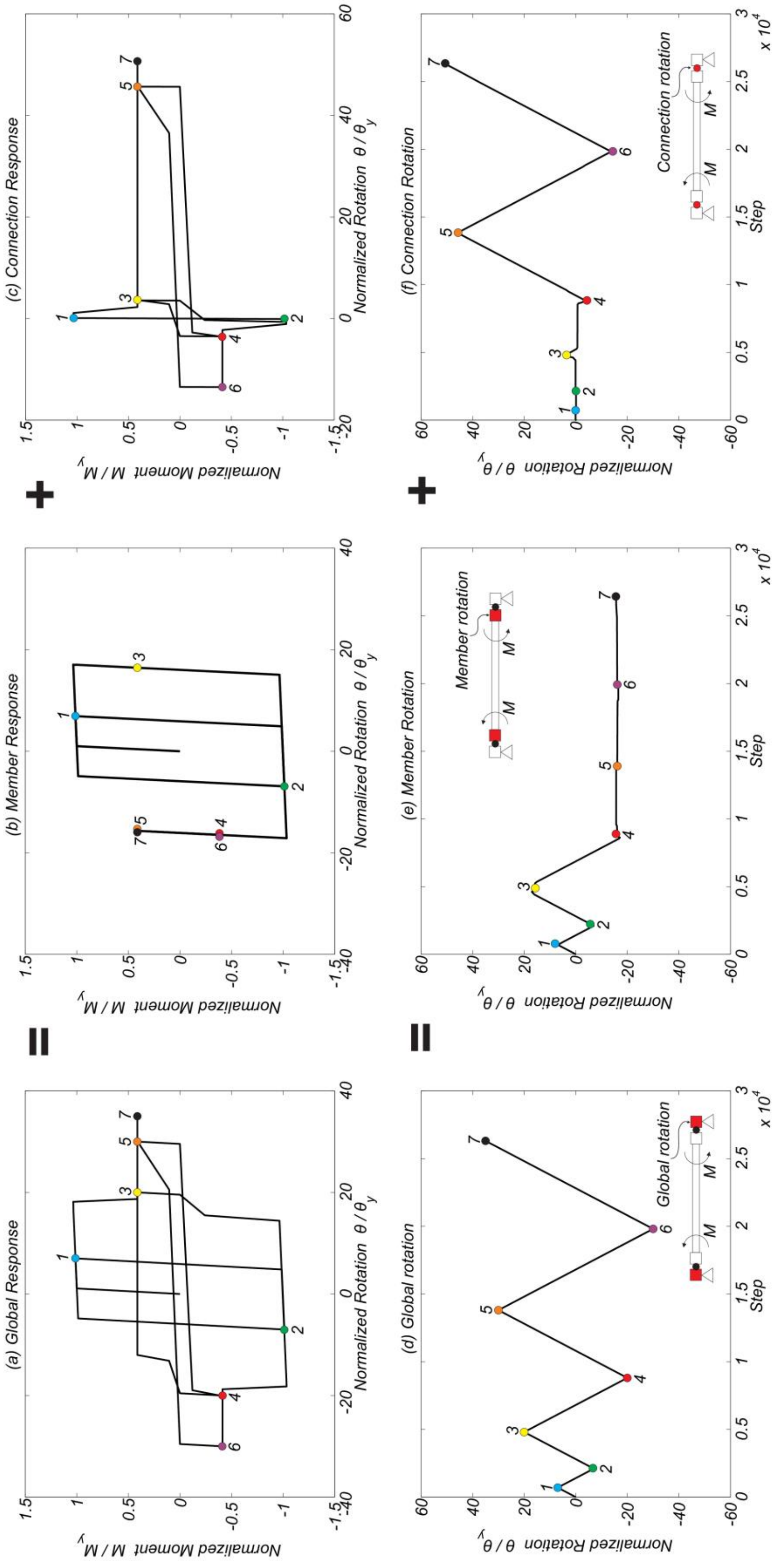


Figure 2 - Illustrative example of the modeling approach behavior

## 2 Model Validation

Four SAC tests were used to validate the proposed modeling approach. An additional test, performed by Bernuzzi et al. (1997) was used to validate post-fracture behavior.

Figures 3 and 4 show the experimental results obtained in the five tests plotted against the numerical results obtained with the implemented model in OpenSees. The model corresponds to two perpendicular frame elements, connected at the base of the vertical element as represented in the figure. The horizontal member is simply supported at both ends. Member sizes and geometrical properties of the test set up are represented at the figures. The beam and column nonlinear properties, namely plastic rotation capacity and deterioration parameters are obtained through the empirical expressions proposed by Lignos and Krawinkler (2011). A cyclic pushover is performed by applying displacements at the top of the vertical element. Material properties are considered following characterization provided in the test reports. The plastic hinge lengths are assumed to be equal to *Lp = L/16*. The connection fracture rotation is obtained after calibration of the proposed modeling approach, which serve as sensitivity analysis and validation of the considered fracture rotations. In these examples, Krylov-Newton algorithm (Scott and Fenves 2010) is used to solve the nonlinear system of equations.

Figures 3(a), 3(b), and 3(d) correspond to tests carried out by Bertero et al. and Popov et al. (SAC 1996) as part of the SAC Project Phase I, whereas Figure 3(c) corresponds to a test carried out in SAC Project Phase II Task 7 (SAC 1998). In Figure 3(a) (Test 1) a very small rotation capacity and, consequently, a brittle fracture was observed at the first cycle of the highest amplitude, with rotation $\theta$+=0.010 rad. In fact, the failure here was a fatigue failure since the fracture occurred at a rotation lower than the maximum attained rotation in previous cycles. The modeling approach provided herein is not able to capture fatigue fractures and thus the rotation of 0.010 rad was considered to be the best proxy of this type of fracture with the proposed model. In the test, fracture of the top beam-flange weld did not occur.

Test 2 (Figure 3(b)) exhibits a similar behavior as fracture did not occur at the maximum attained rotation. Instead it occurred almost at the maximum point in positive bending direction. After fracture, which yielded a value of $\theta^+$ =0.010 rad, the test was continued for one more cycle. This is important to verify the post-fracture behavior of the model. The observed behavior is satisfactory as the numerical results are close to the experimental ones in the decay of the strength and also during reloading. During loading in the negative direction, the top flange also exhibits a decrease in strength, which is presumably related to tearing in the welded connection. Although a complete fracture did not occur, a value of $\theta^+$ =0.018 rad is assumed. After unloading, the numerical results fit well the experimental ones. In spite of the absence of a slab, the larger value of the fracture rotation in the negative loading direction is normal due to the typical lower quality of the bottom flange weld (Popov et al. 1998). Test 3 (Figure 3(c)) is the one with more particularities, mainly because a pre-yield fracture took place at the bottom beam-flange weld. At the top flange, no fracture was recorded. The pre-yield fracture occurred at a rotation of approximately $\theta^+$ =0.003 rad. The maximum negative rotation was 0.018 rad, thus $\theta^+$ >0.018 rad. The simulated response is very close to the observed behavior, which proves a good accuracy of the proposed model in simulating post fracture behavior. It is worth noting that the reloading branches are very close, as are the residual segments. The residual strength capacity is estimated to be 25% of the full member capacity. The pinching parameters were considered to be $F_{r,p}$ = $F_{r,n}$ = 0.3 and $A_{pinch}$ = 0.8. In Test 4 (Figure 3(d)) similar results are obtained and a close match is also found after fracture at both loading directions. In this test, the top beam-flange weld fractured first at a rotation of $\theta^+$ =0.019 rad, followed by fracture of the bottom flange weld at an estimated rotation of $\theta^+$ =0.016 rad. The test was stopped after fracture of the bottom flange connection. However, it is worth noting that both fractures occurred in reality due to fatigue phenomena.

Figure 4 shows the test described by Bernuzzi et al. (1997) in Europe. Since European sections are used, fracture rotations obtained in this test do not match the rotations obtained in the SAC tests. Nonetheless, this test is important to verify the post-fracture behavior as several cycles were conducted after fracture. Bottom beam-flange weld fracture occurred at a rotation $\theta^{+}$ =0.043 rad, after which a pinched hysteretic behavior is observable. In the test results, shown in Figure 4, this behavior is well simulated by the proposed model. The top beam-flange weld did not fracture. The residual strength capacity is taken as 35% of the full member capacity. Finally, the calibrated pinching parameters were considered to be $F_{r,p}$ = $F_{r,n}$ = 0.6 and $A_{pinch}$ = 0.5. In conclusion, the proposed modeling approach captures well the post-fracture behavior of the damaged connections. However, it depends on the definition of the fracture rotation for the accurate assessment of the global response. It is worth noting that the proposed model is not able to capture fatigue-induced fractures, which is a significant drawback. Nonetheless, the model shows a great ability to reproduce post-fracture behavior.

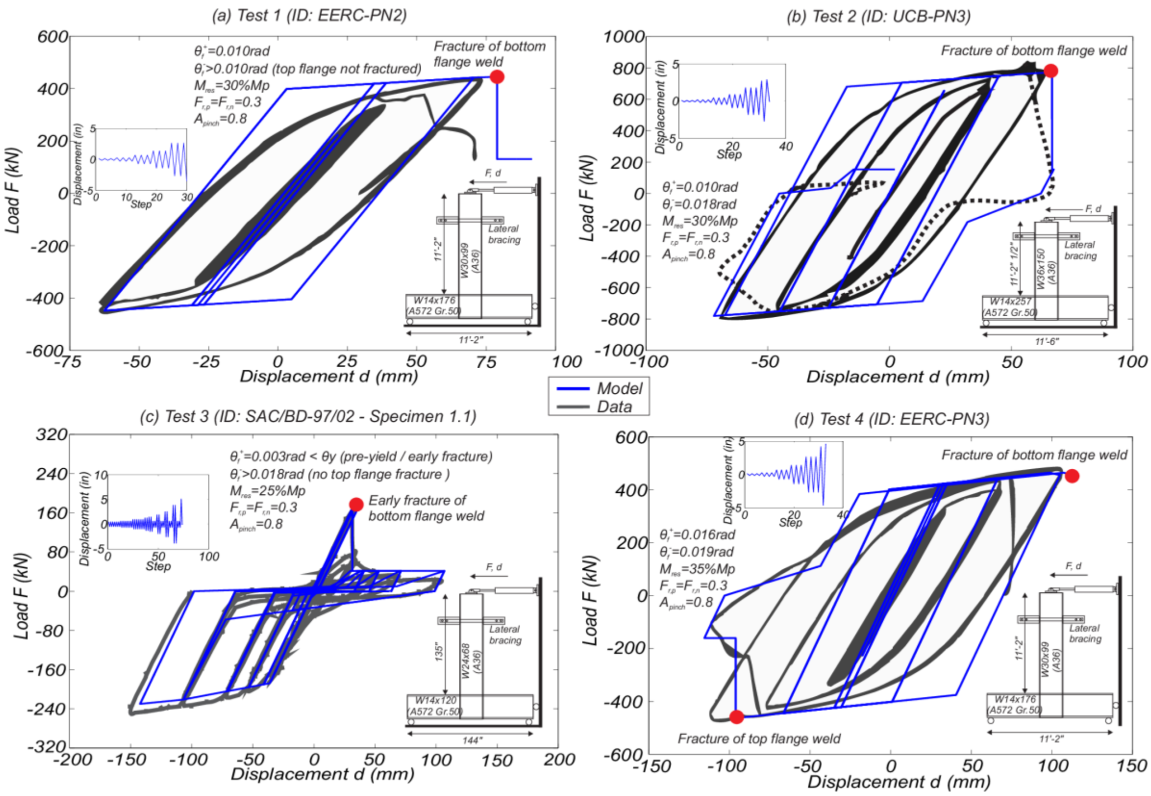


Figure 3 - Numerical analyses performed to validate the modeling approach using four SAC Project tests

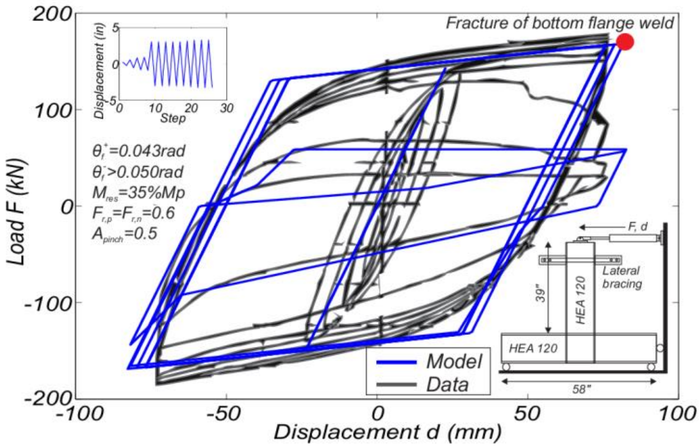


Figure 4 - Numerical analysis performed to validate the modeling approach using a test carried out by Bernuzzi et al. (1997)

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