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NUMERICAL MODEL FOR PARTIAL FRACTURE FAILURE OF STEEL BEAM-TO-COLUMN JOINTS

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SUMMARY

The fracture mechanism is introduced into an elastic-plastic model, which is composed of multi non-linear springs at the end of the members. The failure assessment diagram of fracture based on the J integral theory is adopted for the calibration of the model parameters. By comparison with the experimental results, it is shown that the established model and the computational procedure can fully reflect the main characters of the hysterical behavior after partial fracture. A simplified non-linear model for connection fracture, which can be employed for the whole structural analysis, is then proposed based on numerical simulation results.

INTRODUCTION

Particularly since the 1994 Northridge earthquake and the 1995 Kobe earthquake, there are great concerns about the steel moment-resisting frame (SMRF) buildings with fractured beam-column connections. The research effort mainly focused on several key issues [Chen et al,1]: (1) Fracture mechanism of beam-column connections; (2) Repair details for existing buildings and revisions to moment-frame connection details; (3) Assessment of the structural response after connection failure. The third one is not only important for structural diagnosis and reliability evaluation of the damaged steel structure, but also critical for decision making in the future design concerning the implication of the partial fracture failure. To achieve this objective, a general methodology and numerical model are presented in this paper, considering the partial fracture at the beam-column connection.

In order to narrow down the discussion and simplify the computation, several assumptions and illustrations are made as following:

(1) The beam and column are made of H shape steel. The column is continuous and the beam is welded against the column flange. The connection is assumed to be a rigid type.

(2) The type of connection follows the configuration of the pre-Northridge design. An initial crack is introduced into the beam flange due to the un-removed backing bar, during the welding process [Roeder et al,2]. The revisions and improvements of the connection design can probably eliminate this kind of defect. However, because of the nature of the welding process, the initial defects (like inclusions,

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voids, etc) are most likely in the weld structure. Generally speaking, these initial defects can be treated as initial cracks in the structure.

(3) The fracture of the connection occurs at the beam flange [Engelhardt et al,3], which ignores the phenomena that the crack propagates into the column. Typically, the connection fracture mentioned in this paper is limited at the beam end.

(4) The fracture initiates at the beam flange and can propagate into the web of the beam, which can cause the complete failure of the connections. However this kind of failure is rarely found in the investigation after the Northridge and Kobe earthquake [SAC,4]. Therefore, only partial fracture is considered in this paper, which means the fracture is limited to the beam flange.

(5) The crack propagation and the fracture occur after the relatively large plastic deformation of the connection, which is proved by the investigation after the Kobe earthquake [Nakagomi et al,5]. For the structure experiencing strong seismic load, a large deformation is allowed according to the standard. This assumption seems reasonable for the fracture occurred during the strong earthquake.

NUMERICAL MODEL FOR THE BEAM-COLUMN FRACTURE SIMULATION

The numerical model for the partial fracture of the beam-column connection is developed based on the elasto-plastic fiber-like model [Chen et al,6] of the connections. The fracture mechanism is introduced into the original model. In the original elasto-plastic model, the steel member, beam or column, is divided into elastic element and plastic elements. The plastic element includes several axial elasto-plastic springs, two elastic shear springs paralleling to the principle section axis and one elastic torsion spring. The plastic element is placed at the location where the plasticity can develop during the loading process. All the elements are connected with nodes between them. The constitution relation of the elasto-plastic springs is shown in Fig.1. The elasto-plastic connection model can consider the material yielding and hardening, the Bauchinger effect, the degradation of the connection stiffness and strength due to the local elasto-plastic buckling.

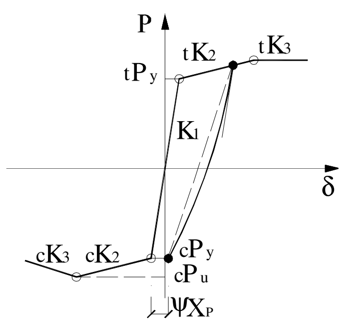


Fig 1 Loading-to-unloading Curve of Non-Fractured Spring

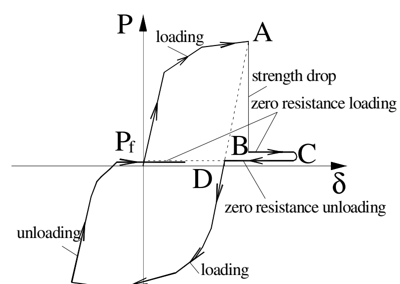


Fig 2 Loading-to-unloading Curve of Fractured Spring

After introducing the fracture mechanism into the original elasto-plastic spring model, necessary modification is needed for the constitution relation shown in Fig.1. The general constitution relation for the fractured spring is shown in Fig.2. The fracture occurs at the tension side after the load reaches the critical value (point A in Fig.2). Then, the strength of the spring suddenly drops to zero. For the convergence in the calculation, the residual strength can be chosen as a very small value, say, 1% of the critical fracture strength (point B in Fig.2). If the fractured spring is loaded while the crack is open, which is defined as zero resistance loading" state, the deformation can develop without the increase of the stress (B-C in Fig.2). If the fractured spring is unloaded before the crack closure, which is defined as zero resistance unloading", the deformation can decrease without the change of the stress (C-D in Fig.2). The fractured spring is assumed to have the same performance as the original spring after the crack closure.

During the calculation, the elasto-plastic springs follow the rules defined in Fig.2. When the stress in the spring reaches the critical value, unstable crack propagation will occur. After comparison of several parameters in the fracture mechanics, J-integral seems to be more appropriate for the fracture analysis for the present problem and thus is chosen as the controlling fracture parameter. The EPRI J estimation scheme [Kanninen et al,7] was used to calculate the J-integral value. The whole joint model is treated as several simple cracked bars, for which the J-integral is calculated by interpolating between the pure elastic and the pure plastic values. The fracture analysis is based on the obtained J-integral together with some experimental parameters.

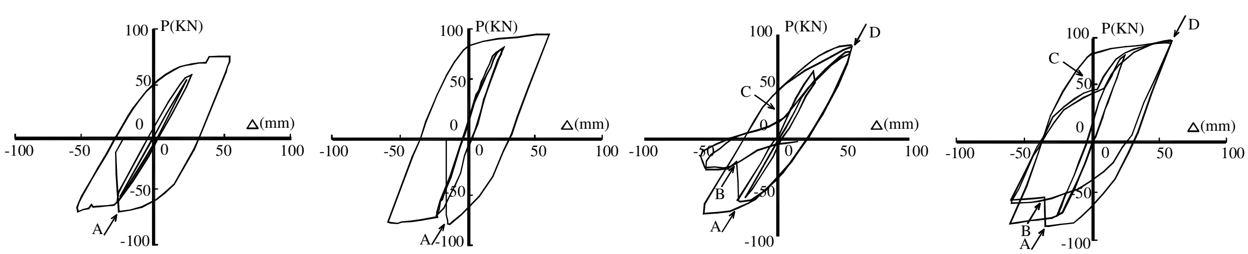
According to the experimental results [Liu et al,8] for two materials (Q235c and Q345c), the J resistance curve for the stable crack propagation and the failure assessment diagram for the unstable crack propagation are currently used in this study [Liu,9]. For practical calculation, the loading history is divided into many small steps. In each step, first the failure assessment diagram is used to assess the state of the spring. If no failure occurs, the stable crack propagation is calculated based on the J resistance curve and accumulated into the current crack length. Check for the convergence criteria and then proceed to the next step. If failure occurs, the constitution relation of the spring is then adjusted according to the rules in Fig.2. The geometric properties of the whole section are recalculated and the global stiffness matrix is updated. Recalculation in the current step is needed to evaluate the other springs' possibility of fracture. Check for the convergence criteria and then proceed to the next step. The procedure is repeated till the end of the computation.

COMPARISON BETWEEN NUMERICAL SIMULATION AND EXPERIMENTAL RESULTS

Uang, Yu et al. [Uang et al,10] did a set of full-scale connection tests based on a real pre-Northridge structure. The beam used is the American standard steel section W36x150, for which the geometric properties can be found elsewhere. The material properties are listed in Tab.1. The fracture parameters are unavailable and thus the experimental results for Q345c are used[Liu et al,8].

Tab.1 Material Properties for the Beam Steel

Component	Steel Type	Yielding Strength (MPa)	Ultimate Strength (MPa)	Elongation rate (%)
Flange	A36	338	476	25
Web	A36	328	452	34



a) **Experimental Results**

b) **Simulation Results**

c) **Experimental Results**

d **Simulation Results**

Fig 3 Comparison between Analytic Model and Test Data

Following the procedure described above, a program code named ROMEO-F was developed. The numerical simulation was performed according to the full-scale tests, where the plastic element was assumed to be 1/10 of the total length of the beam. The comparison of the numerical and the experimental results are shown in Fig.3.

By comparison with the experimental results, it is shown that the established model and the computational procedure can fully reflect the main characteristics of the hysterical behavior after partial fracture. During the experiments, there were sudden huge sounds and deformations when the fracture occurred. For safety issues, generally the test was stopped right after the fracture (Fig.3-(a)) or loaded for another 1~2 cycles (Fig.3-(c)).

From both the numerical and the experimental results, a significant change of the hysterical behavior was observed after the partial fracture of the beam flange. A brief conclusion is drawn as following:

(1) There is a sudden drop of the strength when the fracture occurs (Point A in Fig.3) and the joint can be loaded at a stable manner (Point B in Fig.3);

(2) The unloading stiffness from the tension side deteriorates enormously and the hysterical loop exhibits "shrinking" behavior, which indicates the poor energy dissipating capability of the partial fractured connections.

(3) When the compression loading reaches a specific value, the stiffness increases suddenly. The hysterical loop exhibits "pinch" behavior (Point C in Fig.3). It is due to crack closure and regeneration of stiffness.

(4) The unloading stiffness from the compression side is similar to the un-fractured elastic stiffness (Point D in Fig.3). The stiffness also deteriorates as the deformation decreases. The final target point is the stable loading strength after the partial fracture (Point B in Fig.3). This kind of revolution indicates the process from the crack closure to the re-opening of the crack.

Although there is a small difference between the simplified model and the real situation, the comparison is more than reasonable. The numerical model presented in this paper is qualitatively and quantitatively appropriate for application in the further analysis of the partial fracture of the connections.

SIMPLIFIED BEAM FRACTURE MODEL FOR GLOBAL SYSTEM

The present model is a more detailed connection model, which is difficult to apply directly to big structural systems due to expensive computational costs. In order for the whole structural analysis, a simplified beam fracture model is also recommended. A fracture element is suggested based on the previous simulation results and the experimental results. The fracture element is actually a rotational spring which is placed at each end of an elastic beam in order to emulate plastic hinging (point plasticity) and fracture. The model ignores the length of the plasticity propagation and assumes the angle increase depends only on the moment increase at the end of the beam (Fig.4). The moment-rotation hysteretic behavior (M-θ curve) of the fracture element mimics that seen in two sets of numerical simulations of beam-column connections which experience top and/or bottom beam flange fracture (Fig.5). Three basic curves are explained in detail later.

The parameters in the model are chosen as:

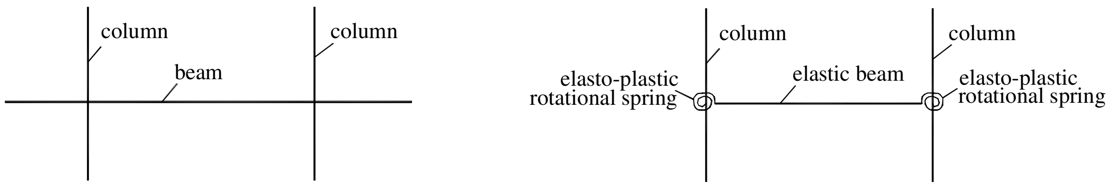
The yielding moment: $M_{y+} = M_{y-} = \sigma_y W$ (1)

The ultimate moment: $M_{u+} = M_{u-} = \sigma_u W$ (2)

The stiffness: $K_1 = EI/(0.1L)$ £¬ $K_2 = \frac{1}{100}K_1$ £¬

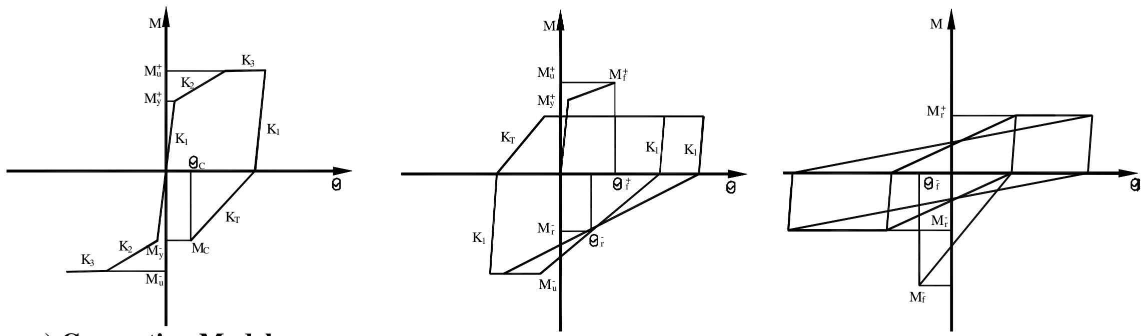
$$K_3 = (-\frac{1}{500} \sim \frac{1}{1000})K_1 \text{ £¬ } K_T = (\frac{1}{50} \sim \frac{1}{100})K_1 \quad (3)$$

Where, E is the Young's modulus, I is the moment of inertia and W is the section modulus.



a) Beam-to-Column Frame **b) Simplified Computation Model**

Fig 4 Rotation Spring Model at the End of Bar



a) Connection Model for Non-Fracture Type **b) Connection Model for Fractur of Single-Flange Only** **c) Connection Model for Fracture of Both-Flange**

Fig 5 Connection Simplified Fracture Model: M-θ

Before the fracture occurs, use the hysteretic loop shown in Fig.5-(a). The factor K_3 is chosen as a small negative value to consider the stiffness deterioration due to the local buckling of the plates if any. For compact section, however, it is reasonable to choose zero and ignore the buckling issues. The unloading path is defined as: first unload to zero stress according to the elastic stiffness K_1, and then load to the nearest reversal deformation according to the secant modulus K_t. After that, follow the skeleton curve. If no reversal deformation occurred before, load to the yielding moment M_y according to the secant modulus K_t. After that, follow the skeleton curve.

When the load reaches the critical value (M_f^+, θ_f^+), fracture occurs at one beam flange. The hysteretic loop is changed to single flange fracture model (Fig.5-(b)). Immediately after the flange fracture, there is a sudden drop in the strength to M_r which is the residual strength of the beam. Then the connection enters the "zero resistance loading" state. The deformation develops without the increase of the stress. The unloading path from the fracture side is defined as: first unload to zero stress according to the elastic stiffness K_1, then load toward the point (M_u^-, θ_u^-), where θ_u^- is the maximum reversal deformation experienced before. If no reversal deformation occurred before, load toward the point (M_r^-, θ_r^-) are defined in Eq.(4)~(5). The unloading path from the non-fractured side is defined as: first unload to zero stress according to the elastic modulus K_1, and then load to M_r^+ according to secant modulus K_t. After that, enter the "zero resistance loading" state.

$$M_r^- = -M_r^+ = -M_r \tag{4}$$

$$\theta_r^- = \theta_f - \frac{M_f - M_r^-}{K_1} \tag{5}$$

If the load is increased further, until the other flange fractures, the hysteretic loop is changed to the double flange fracture model (Fig.5-(c)). On both sides, load according to "zero resistance state" at the stress level $M_r^+ (M_r^-)$. The unloading path is defined as: first unload to zero stress according to the elastic modulus K_1, and then load toward the point M_r^-, θ_u^- (M_r^+, θ_u^+), where θ_u^- is the maxima reversal deformation experienced before.

There are two critical issues in the procedure described above. The first one is the assessment of the fracture. Under the cyclic loading the final fracture load is time dependent, which means it depends on the accumulated damage in the previous loading history. According to the numerical simulation results, it can be simplified as:

$$M_f = [(1 - \frac{a}{t}) \times 0.7 + 0.3] M_u \tag{6}$$

Where, a is the initial crack length and t is the flange thickness.

The second issue is how to determine the residual strength. The simulation results for different sections indicate that the residual strength varies between 0.4~0.5 M_u, if the connections hold for the ideal rigidity. For the connections which use bolts at the beam web, the residual strength may be small due to the variation from the perfect rigid connection assumption. According to the investigations after the Northridge earthquake [Luco et al,11], the residual strength for pre-Northridge connection is about 0.3 M_u. The authors recommend that the residual strength is chosen as:

$$M_r = kM_u \text{ £¬} k = 0.4 \text{ (for whole section welded joints), or } 0.3 \text{ (for only flange welded joints)} \tag{7}$$

This fracture element model can be used for the global structural response analysis considering the partial connection fracture. It simulates the pre-fracture and post-fracture performance of the joints and includes both the strength and stiffness deterioration mechanisms.

CONCLUSION

A fiber-like numerical model is presented in this paper to consider the partial fracture of the steel frame connections, which includes the crack closure and the re-opening effect. By comparison with the experimental results, it is concluded that the model can simulate the partial connection fracture very well, both qualitatively and quantitatively.

Based on the simulation results, a simplified fracture element model is also presented for the global structural analysis. This is useful for the assessment of the influence of the partial connection fracture to the whole structural system.

The fracture considered in this study is limited to beam flange fracture. For other types of fracture failure, the applicability of the current model needs further research.

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