Analytical Modeling and Nonlinear Analysis of Beam-Column Connection in Steel Moment Resisting Frame

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Abstract
Generally the analysis of steel moment resisting frames has been performed without considering the accurate connection behaviors. However, a number of beam-column connections of the steel moment resisting systems raised sudden fracture resulted in enormous economical loss in recent earthquakes. In this study, the analytical modeling of Korean beam-column connections in steel moment resisting frames was proposed based on the experimental studies of other researchers. The adopted computer program for inelastic analysis was DRAIN-2DX, and the standard element type was set to the Element Type 10 in DRAIN-2DX for panel zones and beam-connections. The inelastic analysis results of the example structure with connection modeling were compared to those of the structures without connection modeling.

Keywords: steel moment resisting frames; beam-column connection; analytical modeling; nonlinear analysis

1. Introduction
Steel moment resisting frame is a structural system whereby beam-column connections are made with fully restrained joints, thus allowing the flexural stiffness and flexural strength of the frame members to resist horizontal forces. This structural system gives the structure excellent ductility, making it ideal for applications with a significant response modification factor in seismic criteria, and is widely used in seismic areas. In the 1994 Northridge earthquake, however, early brittle fractures in beam-column connection areas occurred, causing significant damage. Thus, various related American institutions have been conducting experimental researches on the behavior of steel moment connection and developing analytical modeling techniques.

In Korea, the column-tree type and US connection type of bolted-web-welded-flanges are commonly used as beam-column connections in steel moment resisting frames. These connection types employ a work method similar to the welded moment connection method, which exposed serious problems in the 1994 Northridge earthquakes (Lee, 2002). Researches on inelastic behaviors of these connections have not been conducted sufficiently, however, and proper analytical modeling techniques for the design profession have not been developed to appropriately forecast inelastic behaviors. Thus, this study sought to analyze the inelastic behaviors of the above connection types and to propose analytical modeling techniques based on existing modeling techniques. Also, to compare the behavior of the proposed model with that of the existing model, nonlinear analyses of a steel moment resisting frame were conducted. DRAIN-2DX was used as a nonlinear analysis program.

2. Existing Analytical Modeling
Analytical modeling techniques for beam-column connections in steel frame structures are categorized as follows (Foutch and Yun, 2002).

(1) Linear Centerline Model
To design structures or evaluate the performance of existing buildings, there is a need to review two criteria, namely, the strength of members and the stiffness of buildings. The linear elastic model using the central line model is suitable for designing a steel moment resisting frame. Although the model shows appropriate results for designing, it cannot accurately forecast the distribution of the inelastic member forces created by the dynamic load.

(2) Elastic Model with Panel Zone
Fig. 1. shows the scissors model, which includes a panel zone. In this model, beams and columns are connected via rigid links in a panel zone, and the crossroad hinge is connected via a spring with the stiffness of the panel zone. Since this model contains dimension and stiffness of the panel zone, it forecasts
more accurately the distribution of shear forces, flexural moments, and axial forces than the above model.

(3) Nonlinear Centerline Model

The inelastic model is useful in assessing the behavior of existing buildings. To conduct nonlinear analysis, most commercial programs, as shown in Fig.2., connect springs with nonlinear features with section properties of beams and columns.

(4) Nonlinear Model with Panel Zone

The nonlinear analytical models, which include panel zones, are categorized into three techniques. The first model is the scissors model in Fig.1. containing nonlinear property for a spring element. The second model, shown in Fig.3., uses two springs with a panel zone's stiffness and the average strength of the panel zone and the beams. Although this model can express stiffness and strength of a panel zone, it cannot accurately express its shear deformation without expression of the accurate dimension of the panel zone. The third model, developed by Krawinkler (2000), models a panel zone into 8 rigid bodies (Fig.4.). Actually, this model shows the least difference between the actual behavior of a structure and the behavior of the analytical model.

3. Characteristics of Analytical Modeling of Panel Zone and Beam Connection Elements

The following are the characteristics of the spring elements required in the analytical modeling.

The elements that determine the stiffness and strength of a panel zone are the web and the flange of a column. The sum of these two elements determines the shear force-rotation curve of a panel zone, and shows trilinear behavior (Fig.5.). The curve expressions are as follows (Yun and Foutch 2000).

\[ \theta_y = \gamma_y = \frac{F_y}{3G} \]  

\[ \theta_m = \gamma_m = 4\gamma_y \]  

\[ V_y = 0.55F_yd_1d_3 \]  

\[ V_p = V_y + \frac{3b_2t^2}{d_1d_3t} \]
was made using the moment-rotation curve shown in Fig.6. Since the 1994 Northridge earthquake caused early brittle fractures in the welded parts of moment connections as shown in Fig.7., however, SAC Joint Venture has conducted experiments and researches on moment connections (Lee and Foutch, 2000, Yun and Foutch, 2000). As a result, it was observed that inelastic hysteretic behavior in a moment connection was classified into, as shown in Fig.8.(a), a yielding fracture in the welding area, reduced strength, and a pinching effect due to unloading. Accordingly, Foutch and Shi developed the new beam connection element shown in Fig.9. (element type 10) to optimize analysis of the moment-rotation behavior using the DRAIN-2DX program (Foutch and Shi, 1997).

Fig.8.(b) shows the moment-rotation curve created when the experimental curve in Fig.8.(a) was analyzed with this element type 10 of Fig.9.

4. Analysis of the Behavior of Connections in Korean Steel Moment Resisting Frames

The moment connection types used in Korean steel moment resisting frames are the column-tree type and the bolted-web-welded-flange type. Details of these connections are shown in Fig.10.

4.1 Analysis of Experimental Data on the Moment Connections

The moment connections shown in Fig.10. involve welded areas in beam flanges, which are similar to the connection types that experienced brittle fractures in the Northridge earthquake. Thus, the moment-rotation curves of these connections are shown in Fig.9., and the required variables to apply these curves in nonlinear analysis are as follows.

\[
\delta_u: \text{Fracture Rotation} \\
M_f: \text{Post-Failure Moment}
\]

| Researches          | Steel type | Web Connection type | No. of specimens |
|---------------------|------------|---------------------|------------------|
| Lee and Park, 1998  | SS400      | SM490               | Welded           | 3                |
| Lee, 2002           | SS400      | SM490               | Bolted           | 2                |
| Lee and Kim, 2003   | SS400      | SM490               | Welded           | 4                |
| Han and Kwon, 2003  | SS400      | SM490               | Bolted           | 3                |
| Lee, et al, 2004    | SM490      | SM570               | Bolted           | 1                |
| Kims et al, 2001a   | SM490      | SM570               | Bolted           | 2                |
| Kims et al, 2001b   | SM490      | SM570               | Bolted           | 3                |
| Kim and Kim, 2000   | SM490      | SM570               | Bolted           | 2                |
$M_f$: Pinching Moment
$P_1, P_2$: Pinching Rotation
Thus, Korean experimental data in Table 1., which contains two connection types shown in Fig.10., were used to find these variables. Subsequently, items that have to be considered in finding these variables are as follows.

(1) Restricted Variables
The SAC experiment in Fig.8. shows the behavior after the fracture, which makes it possible to estimate all the required variables such as $M_f$, $M_g$, and $P_1$, $P_2$. However, since the Korean experiments stopped immediately after the fracture, as shown in Fig.11., only $\delta u$ could be estimated.

Fortunately, only $\delta u$ is considered to significantly affect the behavior of a frame, and other variables have little effect on the behavior of a frame. Therefore, this study determined that other variables, $M_f$, $M_g$ and $P_1$, $P_2$, follow the values of foreign experimental data. For reference, to identify how $M_f$ has an effect on the behavior of a frame, nonlinear analyses were conducted on a fifth story structure with $M_f/M_y$ divided into 10%, 20% which is the value that is suggested in FEMA-351, and 30%. The fifth story structure is a typical steel moment connection frame as shown in Fig.12.(a). The results of the analyses are shown in Figs.12.(b) and 12(c).

As shown in Fig.12.(b) and Fig.12.(c), since there was little difference in each plotting, $M_f$ is considered to have little effect on the whole frame behavior. Also, since other variables, namely, $M_g$, $P_1$, and $P_2$, were presumed to have little effect on the entire frame behavior, their values also were estimated by foreign data. As a result, $M_f/M_g$, $P_1$, and $P_2$ were made to be used at 5%, +0.001 and -0.003, respectively.

(2) Deformation of Connection
The deformation of a specimen, as shown in Fig.13., is the sum of deformations of a panel zone, a column and a beam. $\delta u$, however, which is required in this study, represents only the deformation of a beam. Thus, the deformation of a panel zone and a column should be subtracted from the total deformation. The results of the specimens in Table 1. are shown in Table 2.

(3) Regression analysis of $\delta u$
After the 1994 Northridge earthquake, many studies for steel moment connections have been accomplished in America. According to the results of the studies, the rotational angle of a connection is proportional to the depth of a beam. Also, to provide $\delta u$ for various moment connection types and beam types, FEMA-355D suggests the following expressions involving the beam depth as a variable, based on experimental results. That is, $\delta u$ is linearly proportional to the beam depth ($d_b$, in)

Welded web: $\delta u = 0.041$ \hspace{1cm} (5)
Bolted web: $\delta u = 0.021 - 0.0003d_b$ \hspace{1cm} (6)

The above expressions are proposed for the Post-Northridge connection type, and will show different values from those of Korean moment connections. Thus, based on Eqs. 5 and 6, this study conducted a regression analysis of the beam depth (cm) and $\delta u$ shown in Table 2. for both the welded-web and the
3.2 Proposed Analytical Model

To conduct nonlinear analyses of moment connections in steel moment resisting frames, the following model is proposed.

(1) Analytical Model
The Krawinkler model (2000) shown in Fig.4.

(2) Panel zone and Beam-connection Elements
- Panel zone: The trilinear curve shown in Fig.5. is used for the moment-rotation curve of the panel zone, and the stiffness and the strength are calculated using Eqs. 1 to 4.
- Beam-connection: The plotting shown in Fig.9. is used for the moment-rotation curve of the beam-connection, and corresponding variables are obtained using Table 3.

Table 3. Variables of the Moment-rotation Curve of the Beam-connection

| Variables | Proposed value |
|-----------|----------------|
| $K_2/K_1$ | % of a beam elastic stiffness, 6EI/l |
| $\delta_u$ | Welded-web-Welded-flange : |
| $M_f$ | Bolted-web-Welded-flange : |
| $M_w$ | 10%, 20%, 30% |
| $P_a$, $P_b$ | 5% |

4. Nonlinear Analysis of an Example Structure

To identify how the proposed model, which takes into account the fracture effect of Korean moment connections, differs from the existing model that does not take into account the fracture effect, nonlinear analyses of the 30 story structure shown in Fig.15. were conducted. The analytical modeling techniques of the existing model and the proposed model are compared in Table 4.

Table 4. Analytical Modeling Techniques of Existing Model and Proposed Model

| Variables | Existing model | Proposed model |
|-----------|----------------|----------------|
| Analytical modeling | Krawinkler model 2000 | Krawinkler model 2000 |
| Panel zone | Trilinear curve | Trilinear curve |
| Beam-connection | Not considering the fracture of the welded connection | Considering the fracture of the welded connection |

The values of $\delta_u$ for the two Korean connection types, namely, the welded web and the bolted web, are compared with FEMA 355D in Table 5. As expected, the values of $\delta_u$ of the Korean welded web significantly differ from those of FEMA 355D, whereas the values of $\delta_u$ of the bolted web are almost the same as those of FEMA 355D. Of these two values, the welded web connection type, used more widely in Korea, was used in the nonlinear analyses of this study and the results were compared with those of the existing model.

Table 5. Fracture Rotation for Domestic Equation and FEMA355D

| Story Level | $\delta_u$ for Korean equation | $\delta_u$ for FEMA355D |
|-------------|-------------------------------|------------------------|
| Welded web  | 0.012 | 0.013 | 0.041 | 0.012 |
| Bolted web  | 0.014 | 0.015 | 0.041 | 0.014 |
| 1-3         | 0.013 | 0.014 | 0.041 | 0.013 |
| 4-6         | 0.013 | 0.014 | 0.041 | 0.013 |
| 7-9         | 0.013 | 0.014 | 0.041 | 0.013 |
| 10-12       | 0.013 | 0.014 | 0.041 | 0.013 |
| 13-15       | 0.013 | 0.014 | 0.041 | 0.013 |
| 16-18       | 0.013 | 0.014 | 0.041 | 0.013 |
| 19-21       | 0.013 | 0.014 | 0.041 | 0.013 |
| 22-24       | 0.013 | 0.014 | 0.041 | 0.013 |
| 25-27       | 0.013 | 0.014 | 0.041 | 0.013 |
| 28-30       | 0.013 | 0.014 | 0.041 | 0.014 |

4.1 Nonlinear Static Analysis

To conduct a nonlinear static analysis, this study followed the lateral load distribution of FEMA273, as shown in Table 6. As a result of an eigenvalue analysis of the example structure using the MIDAS program, the participation mass of the fundamental mode was
less than 75% of the total participation mass and the
fundamental period was 4.37 sec. Thus, a lateral load was
applied in proportion to the story shear distribution
calculated by the SRSS method. The results of the
analysis are as follows.

Table 6. Lateral Load Distribution of FEMA273 for the
Nonlinear Static Analysis

\[ F_i = C_{ix} V \quad C_{ix} = \left( \sum_{i=1}^{n} w_i h_i \right) / m \]

(1) If the mass participation in the fundamental mode is
over 75% of the total mass participation, a lateral load
pattern is represented by values proportional to \( C_{ix} \). This
case is usually used in the structure having a fundamental
period of less than 1.0 sec.

(2) When not included in the above case, a lateral load
pattern proportional to the story shear distribution
calculated a combination of modal responses such as
SRSS and CQC is used. The combination of modal
response should be calculated by the modes including
more than 90% of the total mass participation. This case is
usually used in the structure having a fundamental period
more than 1.0 sec.

(1) Base shear-Roof drift angle (\( \Delta / h \))
As shown in Fig.16., before fractures in the
connection occurred, two analytical models showed
the same behaviors. However, with the fracture's
occurrence in a connection, the proposed model's
strength sharply declined.

On the performance-based seismic design which
ATC proposes as a seismic performance evaluation
method, a base shear-roof displacement curve of the
nonlinear static analysis is converted into a capacity
spectrum. If a performance evaluation of a structure
with fracture connections is conducted, the capacity
spectrum will have a sudden decline curve, thus
showing a totally different performance point. That
is, since a nonlinear static analysis of the proposed
model can produce the result of seismic performance
evaluation that is different from that of the existing
model, serious analysis of the proposed model is
required.

(2) Moment-rotation curve of a connection
In the case of the proposed model, as shown in
Fig.17., a fracture in the connection occurs early and a
significant strength loss occurs. In evaluating ductility
of the connection, the area beneath the moment-
rotation curve may be used. The bigger the area is, the
better the energy absorption ability of the connection
is, and this means that there is a possibility of sufficient
ductile behavior. In the case of the proposed model
shown in Fig.17., the area beneath the moment-rotation
curve was observed to remarkably decline, which
indicates significant lower energy absorption ability.
If the ductile ability of a certain connection declines
remarkably, load redistribution of the entire frame will
occur and a significant change in the entire structure's
behavior will eventually occur. As such, connection
fractures have a great impact on the behavior of
structures.

(3) Story Drift
Since ATC proposes story drift limitation of a
building at a performance point as a method evaluating
the structural behavior, the impact of connection
fractures on story drift was reviewed.

Fig.16. Base Shear-roof Drift Angle

Fig.17. Moment-rotation Curve of the Connection

Fig.18. Base Shear-story Drift for Several Stories

Fig.18. shows base shear-story drift curve by several
stories. In the case of the proposed model, yielding and
fractures occurred at the connections on the 1st and the
10th stories, thus showing story drifts different from
those of the existing model. On the other hand, since
the only elastic behavior occurred without yielding and
fractures at the connections on the 20th and 30th stories
of the proposed model, base shear-story drift curve of
the proposed model shows the same curve as that of
the existing model. As such, since the story drifts of the
low stories increase significantly after the connection
fractures occur, story drift for the proposed model must be evaluated.

Fig. 19 shows story level-story drift curve for the analytical step. Since the horizontal load was small at the initial stage of the analysis, the story drift distribution of all stories was even. With the progress of the analysis, however, the P-Δ effect increased, and thus, the story drift at the low stories was sharply increased. The maximum story drifts of the existing model and the proposed model were 0.081 and 0.101, respectively, showing an approximately 25% difference.

4.2 Nonlinear Dynamic Analysis

The earthquake data used to conduct a nonlinear dynamic analysis is from Santa Monica, City Hall Grounds, 0 Deg, 1994, and the maximum ground acceleration is 0.369g (Fig. 20.).

(1) Time History

As shown in Fig. 21., since at the initial stage, the behavior of the connections occurred only in the region of elasticity and yielding, the time history of the existing model and the proposed model showed the same behavior. Sudden significant ground acceleration, however, caused fractures in the proposed model's connections, and thereafter, a totally different time history from that of the existing model was observed.

(2) Moment-rotation curve of a connection

The connection behavior of the proposed model and the existing model is shown in Fig. 22. In the case of the proposed model, when the rotation of the connection exceeds the fracture rotation (δ_u), the connection loses significant strength and generates a significant rotation. When the maximum rotation of the connections were compared—i.e. 0.0144rad for the existing model and 0.0189rad for the proposed model—an approximately 32% difference was seen.

ATC proposes plastic hinge rotation limitation as the members' reviewing method for seismic performance evaluation. Likewise, in the case of the proposed model, the occurrence of a fracture in a connection creates a significant plastic hinge rotation; and thus, to conduct an accurate seismic performance evaluation, the proposed model is very much needed.

(3) Story Drift

Fig. 23. shows a comparison of the story drift between the proposed model and the existing model. In most of the stories, there was no story drift difference between the existing model and the proposed model.
except from the 15th to the 20th stories. This is because brittle fractures occurred intensively at the welded moment connections of these stories. As a result of comparison, the maximum story drift of the existing model was 0.01572 and that of the proposed model was 0.02122, with the latter approximately 35% bigger.

5. Conclusion

In the Northridge earthquake, the welded moment connections in steel moment resisting frames did not produce sufficient ductile behaviors, but instead created brittle fractures that caused significant economic loss. Thus, this study reviewed analytical modeling techniques focusing on the brittle behavior of connections, and proposed an analytical model appropriate to Korean moment connections. Also, to compare the behavior of an entire structure between the existing model and the proposed model, the nonlinear static and dynamic analysis were performed. The conclusion is as follows.

(1) Korean moment connections are composed of the column-tree type and bolted-web-welded-flanges type. These are similar to the welded moment connections that caused significant problems in the Northridge earthquake. Thus, the spring element of Shi & Foutch (1997), which considers connections' fractures, was used in the Krawinkler (2000) model, and the following expressions were proposed as the value of the most important variable, $\delta_U$.

Welded web: $\delta_U = 0.0183 - 0.000078d_b$
Bolted web: $\delta_U = 0.0229 - 0.000126d_b$

(2) Other variables, $M_p, P_r$, and $P_s$, required for the spring element could not be analyzed using Korean experimental data. However, because it was confirmed that those variables have little impact on the nonlinear behavior of a structure, they were estimated based on the values of FEMA-351.

(3) To identify the impact of brittle fractures of beam-connections on the nonlinear behavior of a steel moment resisting frame, a nonlinear static analysis of a 30-story structure was conducted. In conclusion, there was a great decline in strength in the proposed model after a fracture occurred. There was also a great difference in the moment-rotation curve of the connections, and in the story drift.

(4) To identify the impact of brittle fractures of beam-connections on the nonlinear behavior of a steel moment resisting frame, a dynamic analysis of a 30-story structure was conducted. As a result, when connection fractures occurred in the proposed model, the time history curve and a connection's moment-rotation curve of the two models showed totally different shapes. Also, as a result of comparison of the story drifts, the maximum story drift of the proposed model was a 35% bigger than that of the existing model. As such, because these results show totally different seismic performances between the proposed model and the existing model, use of the proposed model is more seriously required.

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