헌치로 보강된 철골 모멘트 접합부의 탄성 횡변위에 대한 영향

Effects of Haunch Reinforced Steel Moment Connection on Elastic Lateral Drift

이 철 호[1)]

Lee, Cheol Ho

요 약 : 철골 모멘트 접합부를 헌치로서 보강할 경우 내진거동이 크게 증진됨이 최근의 실물대 시험에서 입증되고 있다. 본 연구에서는 헌치로서 보강된 철골 모멘트 접합부가 골조의 탄성 횡변위 거동에 미치는 영향을 해석적으로 평가하는 방안을 제시하였다. 즉 내부의 보-기둥 부분골조를 대상으로 기둥, 보 및 이중패널존에서 기인하는 탄성 횡변위 성분을 해석적으로 유도하였다. 핵심이 되는 내용은 헌치 보강시 생성되는 이중패널존의 전단변형을 고려하는 것이었다. 제시된 방안에 의한 예측치는 3차원 유한요소해석에 의한 결과와 잘 부합하였다. 본 연구에서 수행한 사례연구에 의할 때 헌치의 도입으로 패널존의 강성증대가 가장 현저하여서 패널존의 전단변형에서 기인하는 탄성 횡변위가 50% 정도 감소되었다. 본 연구의 결과는 아직 잘 알려지지 않은 헌치 보강에 따른 부차효과(side effects)의 이해에 도움이 될 수 있을 것이다.

핵 심 용 어 : 철골모멘트골조, 모멘트 접합부, 헌치, 패널존, 보강, 내진거동, 횡변위

1. Introduction

The majority of damage observed in many steel moment-resisting frames (steel MRFs) after the 1994 Northridge earthquake has been local fractures at beam-to-column welded joints. Instead of assumed ductile response, brittle fracture was prevalent. In an effort to repair damaged steel moment frames as well as to strengthen existing and new steel construction after the Northridge earthquake, a variety of ideas have been proposed and verified experimentally. At the University of California, San Diego, four damaged full-scale size specimens were repaired by adding a haunch on the bottom side of the beam and tested either statically or dynamically (Uang and Bondad 1996a, 1996b). Fig. 1 shows the details of one specimen and the test results clearly showed

1) 정회원, 경남대학교 건축공학과 부교수

the effectiveness of such a repair scheme (Uang and Bondad 1996a).

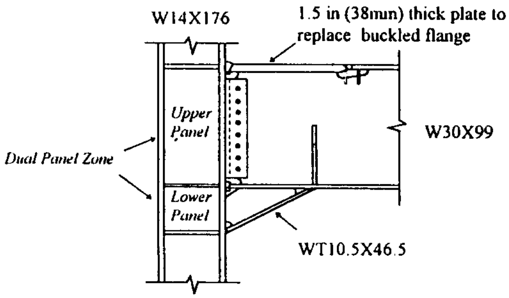


Fig. 1 A Haunch Repaired Steel Moment Connection (Uang and Bondad 1996a)

Although a haunch-strengthened moment connection performed well at the beam-column subassembly level, the overall seismic performance of repaired structures and possible side effects arising from the haunch reinforcement need to be investigated. The original aim of adding haunch is to move the plastic hinging away from the column face and to reduce the stress demand in the groove welds, thereby making the moment connection to be ductile. But this repair scheme will also accompany some increase of the elastic lateral stiffness of the structure as a side effect. When haunches are incorporated in a steel moment frame, the response prediction is complicated by the presence of "dual" panel zones ; the dual panel zone in a steel column is formed when the conventional beam-to-column connection in steel MRFs is enhanced for seismic performance by adding haunches (see Fig. 1). Conventional modeling for the panel zone (for example, Krawinkler 1978) cannot be applied in this case. Recently the author proposed a simplified analytical procedure to model the behavior of the dual panel zone (Lee and Uang 1997) and also conducted a case study to answer some concerns regarding the use of haunches to reinforce existing structures at damaged locations only (Lee 1997, Lee and Uang 1995a, b).

The objective of this study is to investigate effects of haunch reinforcement on the elastic lateral drift of the steel MRFs as a result of modifying the structure with haunch. To this end, approximate analytical expressions of the elastic lateral drift components caused by the beam, column, and dual panel zone were derived for a typical interior subassembly repaired with haunch.

2. Equivalent Rotational Stiffness of Dual Panel Zone

When a connection is reinforced with haunch, a new modeling technique is needed. Treating the dual panel zone as a two-spring serial system *in shear*, and defining a secant shear strain (Fig. 2), it can be shown that the equivalent rotational stiffness $K_{e,eq}$ of the dual panel zone can be established as follows (Lee-Uang 1997) :

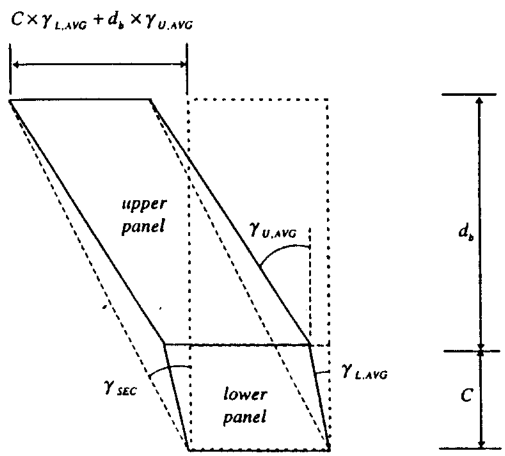


Fig. 2 Deformed Configuration of a Dual Panel Zone in Shear (Lee and Uang 1997)

$$K_{e,eq} = \frac{a_U}{\left\{\frac{d_b}{d_b+C}\right\}\left\{\frac{C}{d_b}\frac{a_U}{a_L}\frac{t_{cw,U}}{t_{cw,L}}+1\right\}} K_{0,U} \quad (1)$$

where

$$K_{0,U} = G(d_c-t_{cf})t_{cw,U}d_b \quad (2)$$

$$a_U = \frac{1}{Q_U d_b} \quad (3)$$

$$a_L = \frac{1}{Q_L d_b} \quad (4)$$

$$Q_U = \frac{A_{bf}+A_{bw}(5-R_1R_2)/24}{S_{btf}} - \frac{1}{H_C} \quad (5)$$

$$Q_L = \frac{R_2A_{hf}+A_{hw}(2R_2+R_1R_2)/6}{S_{btf}} - \frac{1}{H_C} \quad (6)$$

$$R_1 = \frac{d_b/2}{d_b/2+C} \quad (7)$$

$$R_2 = \frac{A_{bf}+A_{bw}/4}{R_1(A_{bf}+A_{bw}/4)+(R_1+1)A_{hw}/2+A_{hf}} \quad (8)$$

$$S_{btf} = \frac{d_b(1+R_1R_2)(6A_{bf}+A_{bw})}{12}+R_2A_{hf}\left(\frac{d_b}{2}+C\right) + \frac{A_{hw}(R_1R_2+R_2)}{2}\left\{\frac{d_b}{2}+\frac{(R_1R_2+2R_2)}{3(R_1R_2+R_2)}C\right\} \quad (9)$$

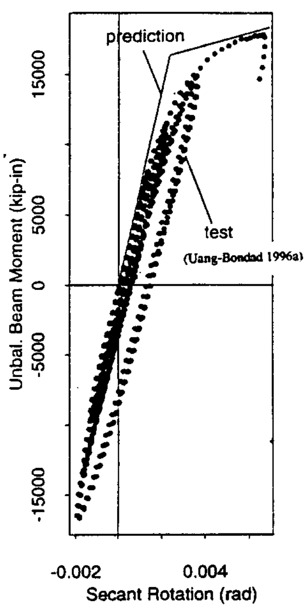


Fig. 3 Predicted versus Test Results

See notation for definitions of the symbols. Because of space limitations, a brief summary of the elastic behavior modeling, which is just relevant to this study, is given in the avove. The proposed modeling procedure predicted stiffness and strength which correlated well with available full-scale cyclic test results (Fig. 3).

3. Formulation of Elastic Lateral Drift Components

Only lateral loading is assumed for a simplified derivation. Based on the classical portal method assumptions, a frame can be resolved into beam-column subassemblies having inflection points at mid-spans of beams and mid-heights of columns. Fig. 4 shows such a typical interior beam-column subassembly repaired with haunch. The elastic lateral deflection in a haunch repaired steel MRF is the sum of the following three lateral deflection components ; 1) lateral deflection caused by shear deformations in the dual panel zone, 2) lateral deflection caused by flexural deformations in the columns, and 3) lateral deflection caused by flexural deformations in the beams. Approximate analytical expressions for the above three deflection components are derived in the following.

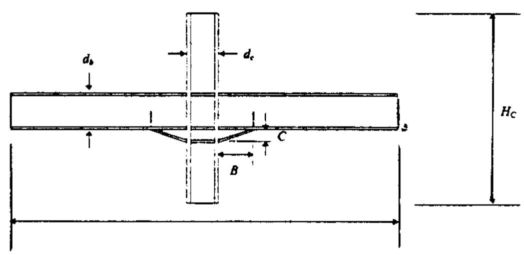


Fig. 4 A Typical Interior Subassembly Repaired with Haunch

Lateral Deflection Caused by Shear Deformations in the Dual Panel Zone δ_p

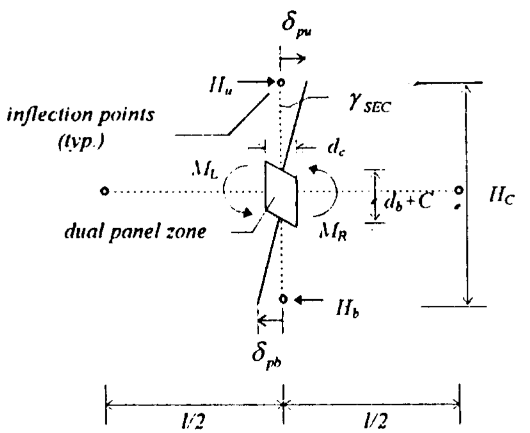


Fig. 5 Lateral Deflection Due to Shear Deformation of a Dual Panel Zone

The D-value method (AIJ 1988, Paulay and Priestley 1992) may be applied to approximately determine the share of each column in a particular story in resisting the story shear force. Once the column shear forces H_u and H_b acting on the subassembly are known, beam shear can be obtained by applying an overall moment equilibrium condition to the subassembly (see Fig. 5). Then the unbalanced beam moment ΔM which causes the shear deformation of the dual panel zone can be approximated as

$$\Delta M = M_L + M_R = \frac{(H_u H_C + H_b H_C)}{2}\left\{1 - \frac{d_c}{l}\right\}$$
$$\cong (H_u H_C/2 + H_b H_C/2) \qquad (10)$$

The secant shear strain of the dual panel zone γ_{SEC} can be calculated by dividing ΔM with the equivalent rotation stiffiness of the dual panel zone $K_{e,eq}$, which was already defined in Equation (1). The resulting expression for γ_{SEC} (in radians) is given in equation (11).

$$\gamma_{SEC} = \frac{\Delta M}{K_{e,eq}} \qquad (11)$$

The secant shear strain γ_{SEC} in Equation (11) approximately corresponds to the elastic story drift ratio contributed by the dual panel zone flexibility. Therefore the elastic lateral drift coming from the dual panel zone is approximated as

$$\delta_p = \delta_{pu} + \delta_{pb} \cong \gamma_{SEC} H_C = \frac{\Delta M}{K_{e,eq}} H_c \qquad (12)$$

For a subassembly without haunch, the expression corresponding to Equation (12) is

$$\delta_p(\text{without haunch}) \cong \frac{\Delta M}{K_{0,U}} H_C \qquad (13)$$

Lateral Deflection Caused by Flexural Deformations in the Column δ_c

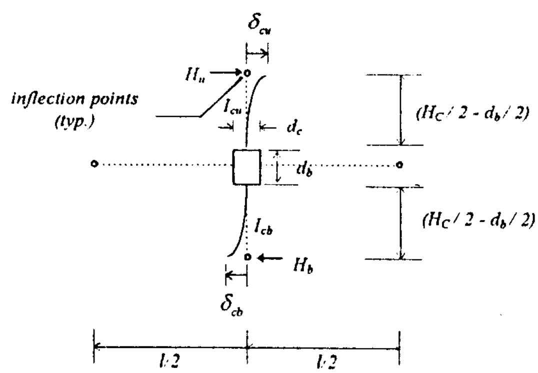


Fig. 6 Lateral Deflection Due to Column Flexural Deformations

δ_c is approximated as the cantilever bending deformations of the columns excluding the portion of the beam depth d_b and can be simply written as(see Fig. 6)

$$\delta_c = \delta_{cu} + \delta_{cb} \cong \frac{(H_C/2 - d_b/2)^3 H_u}{3EI_{cu}}$$

$$+\frac{(H_C/2-d_b/2)^3 H_b}{3EI_{cb}} \quad (14)$$

For a simple and conservative calculation, minor stiffening effects of haunch on the flexural deformation of the column were neglected in the above approximation.

Assuming $I_{ca}=I_{cb}=I_c$ and $H_u \cong H_b \cong \Delta M/H_C$, then Equation (14) can be simplified as

$$\delta_c = \left\{ \frac{(H_C/\ 2-d_b/\ 2)^3}{3EI_C} \frac{2\Delta M}{H_C} \right\} \quad (15)$$

By dividing δ_c in Equations (14) or (15) with the distance between the inflection points of the columns in a given subassembly, the elastic story drift ratio component θ_c due to the column flexural deformations is obtained as

$$\theta_c = \frac{\delta_c}{H_C} \quad (16)$$

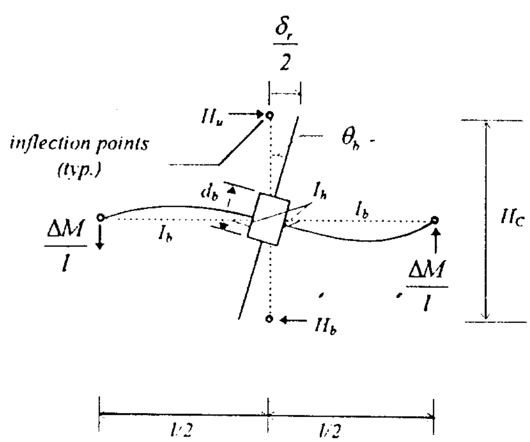
<u>Lateral Deflection Caused by Flexural Deformations in the Beams δ_r</u>

The beam shear determined from the overall moment equilibrium of the subassembly completely defines the bending moment distributions in the beams. Therefore the classical conjugate beam method can be applied to calculate the beam rotation at the column face (see Fig. 7). Note that the elastic loading is not imposed on the panel zone width due to the infinite flexural rigidity of the panel zone. The second moment inertia of the beam section in haunch region was found to increase almost linearly from the shallow end of haunch toward the column face. By applying the conjugate beam method, it can be shown that the beam rotation at the column face θ_b is

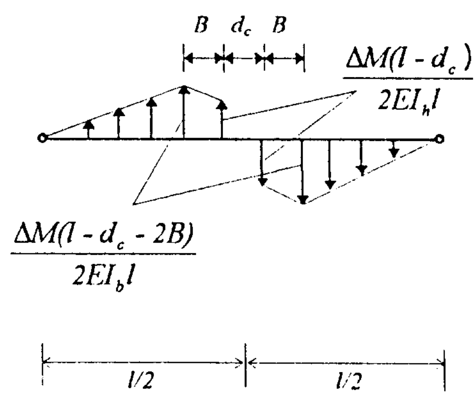
$$\theta_b = \frac{\Delta M}{12El^2} \times$$

$$\left[\frac{(l-d_c-2B)^3 + 3B(l-d_c-2B)(l-d_c-4B/3)}{I_b} + \frac{3B(l-d_c)(l-d_c-2B/3)}{I_h} \right] \quad (17)$$

For a subassembly without haunch, the expression corresponding to Equation (17) is



(a) Deformed Configuration



(b) Elastic Loading on Beam

Fig. 7 Lateral Deflection Due to Beam Flexural Deformations

$$\theta_b(without\ haunch) = \frac{\Delta M(l-d_c)^3}{12El^2I_b} \qquad (18)$$

The beam rotation in Equation (17) is also the elastic story drift ratio (in radians) of a given subassembly due to the beam flexural deformations. By multiplying θ_b with the distance between the inflection points of the columns, the corresponding lateral deflection δ_r is computed to be

$$\delta_r = \theta_b H_C \qquad (19)$$

Combining Equations (11), (16), and (17), the total elastic story drift ratio θ_t can be written as

$$\theta_t = \gamma_{SEC} + \theta_c + \theta_b \qquad (20)$$

4. Illustrative Application and comparison

Fig. 8 shows the frame subassembly selected for a sample analysis. The member sizes were W30X99 and W12X252 for the beam and column, respectively. The repair design was performed per SAC Interim Guidelines (1995).

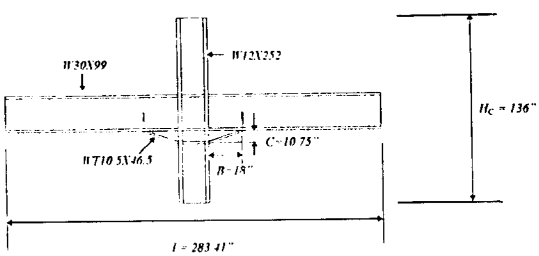


Fig. 8 A Subassembly Repaired with Haunch per SAC Interim Guidelines

Finite element analysis was also carried out for the subassembly. The four-node shell elements in the general purpose linear finite element analysis

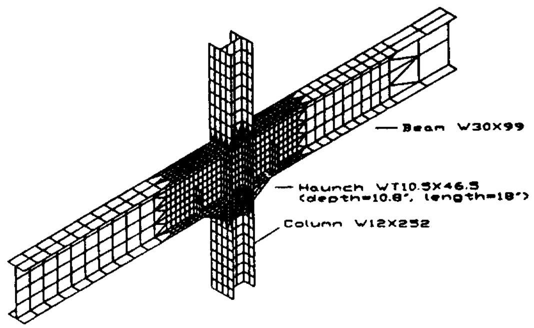


Fig. 9 A Haunch Repaired Subassembly for Finite Element Analysis

program SAP 90 (1992) were used to analyze the three dimensional subassembly. The beam tips were modeled as the inflection points and concentrated forces were applied to the top and bottom ends of the column to simulate column shear forces produced by the lateral earthquake force. Fig. 9 shows finite element meshes. In the case of finite element analysis, the average shear deformation, in either upper or lower panel zone, was obtained by calculating the extension (Δ_1) and contraction (Δ_2) of the diagonals of each panel zone with a width and height of d_1 and d_2, respectively, as follows :

$$\gamma_{L\,AVG}\ (or\ \gamma_{U,\,AVG}) = \frac{\sqrt{d_1^2+d_2^2}}{2d_1d_2}(\Delta_1-\Delta_2) \qquad (21)$$

Nodal displacements available at the four corners of the panel zone were reduced to obtain the relative displacements required in Equation (21). By treating the dual panel zone as a two-spring serial system in shear (see Fig. 2), the secant shear rotation of the dual panel zone was calculated as follows :

$$\gamma_{SEC} = \frac{C\gamma_{L,\,AVG}+d_b\gamma_{U,\,AVG}}{d_b+C} \qquad (22)$$

Beam rotation component was calculated using the nodal horizontal displacements available at beam-to-column joints. Finally, lateral deflection caused by the column was obtained by subtracting the contributions of the dual panel zone and beam from the total relative deflection between the top and bottom of the column.

The results based on proposed procedure and finite element analysis are summarized in Table 1. The results were normalized by the beam unbalanced moment. The loading terms in the relevant equations of this paper were intentionally expressed with the unbalanced beam moment for a convenient comparison. The proposed formulations predicted the elastic lateral drift components which correlated well with finite element results. The prediction of total elastic lateral drift underestimated finite element result by about 5 percent. The effects of haunch repair on the elastic lateral drift is clearly shown in Table 1. With the presence of haunch, the increase of panel zone stiffness was most pronouncing ; the elastic lateral drift component due to the panel zone flexibility was decreased by more than 50 percent. The "stiffening" effects of haunch reinforcement on the beam were relatively minor; the decrease of δ_r was less than 10 percent. Overall, the total elastic drift of the haunch repaired structure was reduced by more than 20 percent relative to the original structure. In other words, the haunch repair accompanied 25 percent increase of the lateral stiffness.

Table 1 Comparison of Elastic Lateral Drift Components

		$\frac{\delta_c}{\Delta M}$ (kip^{-1}) (1)	$\frac{\delta_p}{\Delta M}$ (kip^{-1}) (2)	$\frac{\delta_r}{\Delta M}$ (kip^{-1}) (3)	$\frac{\delta_t}{\Delta M}$ (kip^{-1}) (4)
With Haunch	Proposed Procedure	9.52E-06 (22.6%)*	11.61E-06 (27.6%)*	21.00E-06 (49.8%)*	42.13E-06 (100%)*
	SAP90	10.03E-06 (22.6%)*	11.67E-06 (26.5%)*	22.40E-06 (50.8%)*	44.10E-06 (100%)*
Without Haunch		9.52E-06 (17.0%)*	22.40E-06 (40.6%)*	23.47E-06 (42.5%)*	55.21E-06 (100%)*
$\left(\frac{\text{With Haunch}_{\text{proposed}}}{\text{Without Haunch}} \right)$		1.00	0.52	0.89	0.76

* percentage of contribution of each drift component to total drift

5. Summary and Conclusion

In this paper, approximate analytical expressions were derived to quantify the effects of haunch reinforcement on the elastic lateral drift of steel MRFs. All the elastic lateral drift components of a haunch repaired interior subassembly were approximately derived. Incorporating the dual panel zone flexibility was among the most significant consideration in the derivation. The proposed formulations predicted the elastic lateral drift components which correlated well with finite element result. A sample analysis conducted for a subassembly designed per the SAC Interim Guidelines showed that (i) with the presence of haunch, the increase of panel zone stiffness was most pronouncing ; the elastic lateral drift due to panel zone flexibility was reduced by about 50 percent, (ii) the stiffening effects of haunch on the beam were relatively minor, and (iii) overall, the haunch repair accompanied about 25 percent increase of the lateral stiffness as a side effect. This side effect will be beneficial to reducing often too excessive story drift of steel moment-resisting frame with "pre-Northridge type" moment connection.

Acknowledgments

Funding for this research provided by

Kyungnam University in 1996 is gratefully acknowledged.

References

1. AIJ (1988). *Standard for structural calculations of reinforced concrete structures*, Architectural Institute of Japan.
2. "Interim guidelines : evaluation, repair, modification and design of welded steel moment frame structures." (1995). *Report No. SAC-95-02*, Federal Emergency Management Agency, Sacramento, Calif.
3. Krawinkler, H. (1978). "Shear in Beam-Column Joints in Seismic Design of Steel Frames," *Engineering Journal*, AISC, Vol. 15, No. 3, pp. 82-91.
4. Lee, C. H. (1997). "System seismic performance of haunch repaired steel MRFs : dual panel zone modeling and a case study." *Structural Engineering and Mechanics, An International Journal* (in press)
5. Lee, C. H., and Uang, C. M. (1995a). "Seismic response of haunch repaired steel SMRFs : analytical modeling and a case study." *Report NO. SSRP 95/11*, Univ. of California, San Diego, Calif.
6. Lee, C. H., and Uang, C. M. (1995b). "Analytical and field investigations of buildings affected by the Northridge earthquake of January 17, 1994." *Report NO. SAC-95-04, Part 2, pp. 8-1-8-49*, ATC, Redwood City, Calif.
7. Lee, C. H., and Uang, C. M. (1997). "Analytical modeling of a dual panel zone in haunch repaired steel moment-resisting frames." *J. of Struct. Engrg.*, ASCE, No. 1 Vol. 123, pp. 20-29.
8. Paulay, T. and Priestley, M. J. N. (1992), *Seismic Design of Reinforced Concrete and Masonry Buildings*, John Wiley & Sons, INC., New York.
9. SAP90 (1992). *A Series of Computer Program for Finite Element Analysis of Structures*. Computer & Structures Inc., Berkeley, Calif.
10. Uang, C. M. and Bondad, D. (1996a). "Static cyclic testing of pre-Northridge and haunch repaired steel moment connections." *Report No. SSRP-96/02* (Final Report to SAC), Univ. of California, San Diego, Calif.
11. Uang, C. M., and Bondad, D. (1996b). "Dynamic testing of pre-Northridge and haunch repaired steel moment connections." *Report No. SSRP-96/03* (Final Report to NSF), Univ. of California, San Diego, Calif.

Notation

The following symbols are used in this paper:

A_{bf} = beam flange area;
A_{bw} = beam web area;
A_{hf} = haunch flange area;
A_{hw} = haunch web area ;
B = horizontal length of haunch ;
C = depth of haunch;
d_1, d_2 = width and height of panel zone ;
d_b = depth of beam;
d_c = depth of column;
E = modulus of elasticity of steel;
G = shear modulus of steel;
H_C = story height, or distance between inflection points of columns;
H_u, H_b = column shear force;
I_b = second moment of inertia of beam;
I_c , I_{cb}, I_{cu} = second moment of inertia of column;
$K_{0,U}$ = conventional rotational stiffness of single panel zone;
$K_{e,eq}$ = equivalent rotational stiffness of dual panel zone;
l = beam span length;
$1/Q_U$, $1/Q_L$ = average depth factors for upper and lower panel zone, respectively;

R_1 = ratio of beam bottom flange stress to haunch flange stress;

R_2 = ratio of haunch flange stress to beam top flange stress;

S_{bf} = repaired section modulus for beam top flange at column face;

t_{cf} = thickness of column flange;

$t_{cw,L}$ = web thickness of lower panel zone including, if any, doubler plates;

$t_{cw,U}$ = web thickness of upper panel zone including, if any, doubler plates;

t_{hw} = thickness of haunch web;

α_L = stiffness and strength modification factor for lower panel zone;

α_U = stiffness and strength modification factor for upper panel zone;

Δ_1, Δ_2 = extension and contraction of the diagonals of panel zone;

ΔM, M_L, M_R = unbalanced beam moment;

δ_{cb}, δ_{cu}, δ_C = lateral deflection caused by column flexural deformations;

δ_{pb}, δ_{pu}, δ_p = lateral deflection caused by panel zone shear deformation;

δ_r = lateral deflection caused by beam flexural rotations;

$\gamma_{L,AVG}$ = average shear strain in lower panel zone;

$\gamma_{U,AVG}$ = average shear strain in upper panel zone;

γ_{SEC} = secant rotation of dual panel zone;

θ_r = story drift ratio due to beam flexural rotations;

θ_c = story drift ratio due to column flexural deformations;

θ_t = total story drift ratio.